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CONTEMPORARY PROBLEMS OF ARCHITECTURE AND CONSTRUCTION

PROCEDINGS OF 9 TH INTERNATIONAL CONFERENCE CONTEMPORARY PROBLEMS OF ARCHITECTURE AND CONSTRUCTION

Gela Kipiani

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Hydraulic Facilities Engineering, Environmental Engineering, Transport Problems Civil Engineering, sustainable construction, materials, structural mechanics and technologies, Architecture and Urban Planning;

Batumi - Georgia, September 13-18, 2017

Edited by
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PREFACE

I would like to thank all researchers, guests, partner universities for their participation. I appreciate the Scientific Committee, Organizing Committee members, Editorial Board and others for their contribution in publishing process of the above mentioned books.

Coordination of up-to-date issues in the field is one of the most important activities in the sphere of science, which unites representatives of different nationalities, exchange their best practices. 9lh International Conference on Contemporary Problems of Architecture and Construction is a vivid example of this activity.

Let me give you a brief update on the way we have overcome together and where we are today. In 2008 National University of Architecture and Construction of Armenia initiated and organized the international conference entitled "Contemporary Problems of Architecture and Construction" in Jermuk, Armenia from October 15 to 18 with participants from Russia, Georgia, Syria. In 2010 the 2nd conference was organized in Jermuk, Armenia from September 30 to October 3 with the participation of representatives from China, Russia and Georgia. The 3rd conference was organized in Beijing, China in collaboration with the Beijing University of Civil Engineering and Construction from November 20 to 24, 2011, with the participation of representatives from China, Armenia, Poland, USA, Russia, France, Italy, UK, Georgia, Moldova and Thailand. The 4th conference was held in the Polish city of Czestokhowa from September 24 to 27, 2012, as a joint effort of the Czestochowa University of Technology, National University of Architecture and Construction of Armenia, the Beijing University of Civil Engineering and Construction with the participation of representatives from Poland, Armenia, China, USA, Russia, Italy, Georgia, Czech Republic, Germany, Slovakia, Belarus, Ukraine and Hungary. The 5lh conference was organized in Saint Petersburg, Russia by the Saint Petersburg University of Architecture and Construction, National University of Architecture and Construction of Armenia, the Beijing University of Civil Engineering and Construction and Czestochowa University of Technology, which took place from June 25 to 28, 2013. The 6th conference was organized at the University of Ostrava in Ostrava, Czech Republic from June 24 to 27, 2014 with efforts of the VSB - Technical University of Ostrava, National University of Architecture and Construction of Armenia, the Beijing University of Civil Engineering and

Construction, Czestochowa University of Technology, Saint Petersburg University of Architecture and Construction. The 7th'in Florence, Italy at the University of Florence, November 19 to 21, 2015, with joint efforts of the University of Florence, National University of Architecture and Construction of Armenia, the Beijing University of Civil Engineering and Construction, Czestochowa University of Technology, Saint Petersburg University of Architecture and Construction, VSB - Technical University of Ostrava, with the participation of representatives from Italy, Armenia, China, Poland,Russia, Czech Republic, Kazakhstan, Iran, Georgia, Ukraine, Portugal, USA, Lithuania, Netherlands, Mexico, Philippines, Columbia. The 8th in Yerevan (Armenia) at the National University of Architecture and Construction of Armenia from October 26 to 28, 2016 in cooperation with the partner universities: National University of Architecture and Construction of Armenia; Beijing University of Civil Engineering and Architecture; Czestochowa University of Technology; Saint Petersburg State University of Architecture and Civil Engineering; VSB - Technical University of Ostrava; The University of Florence.

Hence we are proud to guest in Batumi (Georgia) the "9th International Conference on Contemporary Problems of Architecture and Construction", held from September 13 to 18, 2017 in cooperation with the partner universities:

- Batumi Shota Rustaveli State University
- National University of Architecture and Construction of Armenia
- Beijing University of Civil Engineering and Architecture
- Czestochowa University of Technology
- Saint Petersburg State University of Architecture and Civil Engineering
- VSB - Technical University of Ostrava
- The University of Florence
- NGO "Progress in Education and Science" Georgia

This conference reflects the values and missions of our universities in the best way. This is a platform where close, friendly traditions and relations between countries are getting stronger. It gives an opportunity to numerous scientists from Armenia, China, Poland, Russia, Italy, USA, Georgia, UAE, Australia, Canada, Belorussia, Ukraine, Slovakia etc. submit their contributions and open the space for discussions, scientific exchanges.

Hereby I'm sure that each of us would stay engaged, keep proactive, continue to meet the challenges of our fields and help us shape the future of International Conference on Contemporary Problems of Architecture and Construction more comprehensive and ongoing.

In addition I am very thankful for support of conference to Vice-Prime Minister & Minister of Energy of GeorgiaMr. Kakha Kaladze as well as for Chairman of Management Board of JSC Georgian State Electrosystem (GSE) Mr. Sulkhan Zumburidze

Gela Kipiani,
Guarantor of the Conference
Doctor of Technical Sciences, Professor
Georgian State Prize Winner in Science and Technology

Table of Content

Hydraulic Facilities Engineering, Environmental Engineering, Transport Problems Civil Engineering, sustainable construction

ANALYSIS OF CRACK FORMATION AND SEEPAGE INTO THE FOUNDATION OF THE ENGURI HPP DAM
Kalabegishvili M., Sadunishvili M. 3

TOTAL ENERGY Balance OF WINDOWS IN DEPENDENCE ON THEIR STRUCTURAL DESIGN
Iveta Skotnicova, Lenka Lausova 9

Legal Problems of Providing Transport Accessibility of Social Infrastructure Facilities for Emergency Response Services in the Transportation Security System
Dmitry Ivanov 13

NEW TECHNICAL POSSIBILITIES FOR ENHANCING THE QUALITY OF ARCHITECTURE – MATERIAL PROCESSING TECHNOLOGY BY WATER CUTTING
Jarosław Rajczyk 16

Initiatives towards improvement in energy performance in the Polish construction sector
Lucjan Kurzak 19

INNOVATIVE METHODS OF STRENGTHENING THE TIMBER CONSTRUCTIONS
Marlena Rajczyk, Damian Jończyk, Bartłomiej Stachecki 25

NEW CONSTRUCTION AND ARCHITECTURAL SOLUTIONS FOR CIRCULAR HOUSES FOR THE NEEDS OF HUMANITARIAN AND EMERGENCY CAPACITY BUILDING PROJECTS
Marlena Rajczyk, Paweł Rajczyk, Vedna Ramzaha, Artur Kiowski, Marcin Guzik 29

Experimental verification of hypothetical strength theory for gravel soils
Anatoly Mirnyy, Armen Ter-Martirosyan 34

Tendencies of the Development of Historical Cities
Taking Into Account Identification Characteristics of the Established Urban Environment 37
Fedor Perov,Aleksandr Panin .....

Study of Oscillations Damping Processes in Vibrating Construction Machines
Sergey Repin, SergeyEvtyukov 41

SOLVING PROBLEMS OF MAJOR REPAIR OF LARGE-PANEL BUILDING EXTERNAL WALLS 47
Evgeny Smirnov, Evgeny Shesterov, Irina Drozdova .....

Drivers of the Growth of the Russian Construction Sector
Galina Tokunova, Svetlana Golovina 52

ON ONE MIXED BOUNDARY PROBLEM FOR AN ELASTIC EXPONENTIALLY NONHOMOGENEOUS BY DEPHTH LAYER AT ANTIPLANE DEFORMATION
Siranush Avetisyan, Nune Grigoryan, Mger Mkrtchyan 58

PROSPECTS OF DEVELOPMENT OFNANOTECHNOLOGIES IN PRODUCTION OF CELLULAR GLASS BASED ON VOLCANIC GLASS WOOL AND OTHER SILICATE ROCKS OF THE REPUBLIC OF ARMENIA 63
Artavazd A. Arzumanyan, Emma R. Sahakyan .....

THE ROLE AND IMPORTANCE OF DRAINAGE SYSTEMS IN THE RECONSTRUCTION OF DAMAGED HEADWALLS
Ashot B. Grigoryan, Vardges I. Grigoryan, Vahan V. Grigoryan 66

UNIFORM STATIONARY LAMINAR FLOW OF VISCOUS FLUID IN OPENCHANNELS OF PARABOLIC CROSS-SECTION 69
Arestak A. Sarukhanyan .....

ONTACT PROBLEMS ON INTERACTION OF THIN-WALLED ELEMENTS WITH AN LASTIC LAYER UNDER TORSION
.A. Davtyan, A.V. Gasparyan, Sh.A. Knyazyan 73

PROBLEMS OF CONSERVATION AND USAGE OF WILD GREENS AND WAYS OF THEIR SOLUTION IN ARMENIA
RafikYedoyan, Tatevik Yedoyan…… 78
HYDRAULIC JUMP IN A PIPE CANAL
Margaryan Albert, Hakobyan Marianna , Gevorgyan Armine…… 82
Heterodyne method of linear measurements
Hovseph S. Petrosyan, Yeghisabeth H. Hayrapetyan…… 85
METHOD OF RECTANGULAR FINITE ELEMENTS WITH QUADRATIC LAW OF VARIATION OF COMPONENTS' DISPLACEMENTS ALONG THE CIRCUIT FOR SOLVING PLANE PROBLEMS OF MOMENT THEORY OF ELASTICITY
Narine Pirumyan …… 90
ABOUT WATER RESISTANCE OF PLASTERS Of TATEV MONASTERY COMPLEX AND GOVHAR AGHA MOSQUE IN SHUSHI
Gagik Sh.Galstyan, Avetiq A. Arzumanyan, Hovsep A. Hoveyan…… 92
ON A CLASS OF MIXED BOUNDARY-VALUE PROBLEMS OF CONTINUUM MECHANICS SOLVEABLY BY THE INTEGRAL EQUATIONS METHOD
Marine Grigoryan, Vardges Yedoyan…… 97
STUDY OF THE STABILITY OF ANNULAR ROUND PLATES BY THE USAGE OF HERMITE'S FUNCTION
Manvel Z.Pogosyan, Armine L. Grigoryan, Sargis H. Khalatyan…… 104
ON DETERMINATION OF VALUES OF THE VELOCITIES OF TRANSVERSE SEISMIC VIBRATIONS OF SOIL PARTICLES IN STRONG EARTHQUAKES
Eduard Khachiyan[1], Levon Levonyan[1]*…… 107
EXISTING RESIDENTIAL BUILDINGS' MANAGEMENT IN ARMENIA AND APPLICATION OF BIM
Marine L.Ghazaryan, Mher V.Markosyan…… 113
ON INTERACTION OF CRACK TYPE STRESS CONCENTRATORS IN AN ELASTIC PLATE AND AN ABSOLUTELY RIGID THIN INCLUSION INDENTED INTO THE PLATE TAKING INTO CONSIDERATION THEIR ADHESION
Eghine Kanetsyan, Musheg Mkrtchyan, Suren Mkhitaryan…… 117
THE STUDY OF CRACK PROPAGATION IN THE BODY OF CONSTRUCTION USING THE METHOD OF SECTIONS
Gevorg V. Ter-Petrosyan…… 123
Geometrical analysis of cavity defects behind lining in high-speed railway tunnel
Ren REN , Jun DONG, Dongqing LI, YonggangWANG, Dianyuan LIU …… 125
ASSESSMENT OF ENERGY SAVING POTENTIAL THROUGH IMPLEMENTATION OF THERMAL INSULATION OF RESIDENTIAL APARTMENTS
AnahitR. Harutyunyan, Armine B. Ghulyan…… 129
THE FORECAST OF GROUNDWATER LEVEL UNDER INFLUENCE OF NATURAL AND ARTIFICIAL FACTORS OF STRUCTURES AND AREAS
Emil Khachatryan, Artur Khachatryan, Gevorg Hakobyan…… 134
HEALTH MONITORING OF SEISMIC ISOLATED BUILDINGS
Artashes Sargsyan, Sargis Stepanyan…… 137
REDISTRIBUTION OF FORCES IN RUBBER BASE ISOLATIONS OF MULTISTOREY BUILDINGS DURING EARTHQUAKE ACTION
Tigran L. Dadayan..... 141
The peculiarities of building culture of Armenian historical and architectural heritage
Lusine G. Karapetyan..... 145
THE USE OF MODERN ENGINEERING DECISIONS IN BUILDINGS FOR DISASTER RISK REDUCTION
Guram Gabrichidze, Elguja Medzmariashvili, Gaioz Partskhaladze, Gocha Chavleshvili[2]…… 151

INTERNATIONAL REGULATION OF CONSTRUCTION DEVELOPMENT
Nino Zurashvili 158

THE DEVELOPMENT OF PRACTICAL METHODS FOR THE CALCULATION OF THIN-WALLED ROD SYSTEMS
Paata Geradze, Parmen Kipiani, Malkhaz Dograshvili..... 162

THE FIRST LIMIT STATE ATTHERMOMECHANICAL LOADING
Elguja Minadze[1], Omar Kikvidze[2]..... 165

GEOMETRICAL ANALYSIS OF CAVITY DEFECTS BEHIND LINING IN HIGH-SPEED RAILWAY TUNNEL
Ren REN , Jun DONG, Dongqing LI, YonggangWANG, Dianyuan LIU..... 167

EXPERIMENTAL STUDY ON MECHANICAL MODEL OF CLOSED CAVITY DEFECTS IN VAULT LINING OF HIGH SPEED RAILWAY TUNNEL
Dongqing Li Jun Dong, RenRen, Yonggang Wang..... 172

ANALYSIS OF DYNAMIC RESPONCE OF IN-SERVICE HIGH SPEED RAILWAY TUNNEL WITH CAVITY DEFECTS IN LINING
Jun Dong , Yonggang Wang, Dongqing Li, Ren Ren, Chao Wang..... 176

STUDY ON TEMPERATURE CONTROL MECHANISM OF CONCRETE CRACK UNDER COMPLEX STRESS FIELD IN THE CONSTRUCTION OF LARGE CABLE STAYED BRIDGE TOWER
Shun Chang MA, Chunguang REN, Jun DONG, Bo LI, Liyang ZHANG..... 181

STRESS ANALYSIS OF THE KEY PARTS IN THE CANTILEVER CONSTRUCTION PROCESSOF LARGE SCALE CABLE STAYED BRIDGETOWER
Liyang ZHANG, Jun DONG, Hongzhan SHI 189

STUDY ON CONSTRUCTION AND CONTROL OF LARGE VOLUME CONCRETE IN SUPER-LARGE CANTILEVER CONCRETE PYLON
Hongzhan SHI, Jun DONG, Xuejian XIANG, Dianyuan LIU, Chunguang REN..... 193

RESEARCH ON KEY TECHNOLOGY OF SUPER-LARGE CANTILEVER CONCRETE TOWER CONSTRUCTION OF LARGE CABLE STAYED BRIDGE
Hongzhan SHI, Jun DONG, Xuejian XIANG, Dianyuan LIU,Chunguang REN..... 200

STUDY ON SEISMIC DYNAMIC RESPONSE OF SUPER-LARGE CANTILEVER CONCRETE PYLON STRUCTURE OF LARGE CABLE-STAYED BRIDGEJUN
Jun DONG,Hongzhan SHI, Xuejian XIANG Dianyuan LIU,Bo LI..... 205

URBAN DEVELOPMENT A CASE STUDY OF BEIJING'S URBAN CONSTRUCTION AND CULTURAL RENEWAL
Hongbing Zhang..... 212

DETECTION AND EVALUATION OF POST-FIRE PC BOX GIRDER OF BEARING CAPACITY
Long Peiheng, Yu Wenfa, Zhang Guangda..... 216

REVIEW OF SEISMIC BEHAVIOR OF ASSEMBLED ANTI-LATERAL STEEL MEMBERS
Dongfu Zhao, Shuang Wu..... 222

PERFORMANCE OF SELF-CENTERING STEEL FRAMES WITH INTERMEDIATE COLUMNS CONTAINING FRICTION DAMPERS UNDER HIGH-MAGNITUDE SEISMIC ACTIONS 227
Yanxia Zhang, Hexin Zhang, Anran Liu, Yan Wang.....

VIEW OF APPLICATION OF BIM IN PROJECT MANAGEMENT OF URBAN UTILITY TUNNEL
Siqi Wang , Liang Wang, Weitong Yi , Youlu Li 243

REVIEW OF CLOUD AND BIM-BASED COLLABORATION IN CONSTRUCTION MANAGEMENT OF SUBWAY ENGINEERING
Weitong Yi, Liang Wang, Siqi Wang , Youlu Li..... 248

SAFETY AND RELIABILITY EVALUATION OF ANCIENT WOOD STRUCTURE BASED ON DYNAMIC METHOD 253
Xiufang Wang, Mingcong Gao，JunDong.....
STUDY ON THE VIBRATION RESPONSE OF THE EXISTING HIGH SPEED RAILWAY TUNNEL WITH CAVITY DEFECTS IN LINING
Chao Wang [1] Jun Dong [1a*] Dongqing Li [1] Ren Ren[1] Yonggang Wang [1]..... 258
EXPERIMENTAL STUDY ON THE EFFECTS OF CRACK WIDTH AND DEPTH ON THE CHLORIDE PENETRATION OF CONCRETE
Jianfu Yu; Jun Dong; Chunxiang Kang..... 265
STUDY ON SETTLEMENT AND DISPLACEMENT OF ADJACENT BUILDINGS OF EXCAVATIONS TO CORNER EFFECT
Weihao WANG, Jun DONG, Jianrong PANG..... 271
EXPERIMENTALE STUDY ON MECHANICAL BEHAVIOR OF NEW TYPE STEEL-TIMBER COMPOSITE BEAM
Guosong SHI[1], Jun DONG[2], Huifang CHEN[1]*..... 278
STRUCTURAL DESIGN AND ANALYSIS OFD BUSINESS AREA OF EVERGRANDE INTERNATIONAL CENTER
Xie Bing, Zhang Lu, Zhao Chiyun 283
APPLICATION OF LOW YIELD POINT STEEL IN STRUCTURAL SEISMIC DESIGN
Xiaowei Feng, Chiyun Zhao[1]..... 288
THEINFLUENCEOF CRD METHODONTHEEXISTINGSTATION
Songsong Hou , Fei Liu, ChunjieZhu..... 291
COLLISIONINSPECTIONANALYSISOFASSEMBLEDBUILDINGBASEDON BIM TECHNOLOGY
Tianlei Ma, Sihua Deng, Chenguang Li..... 298
THE RELATIONSHIP BETWEEN THE FRACTURE TOUGHNESS AND LOADING RATE OF ROCK-LIKE MATERIALS
Hongsen Wang, Chengzhi Qi, Qinglei Li, Huating Zhu 301
DYNAMIC ANALYSIS OF TUNNEL CROSSING SOIL-ROCK BOUNDARY UNDER DIFFERENT WATER PRESSURE
Li Qinglei, Qi Chengzhi, Wang Hongsen, Liu Yingqi 306
CRETE TOWER CONSTRUCTION OF LARGE CABLE STAYED BRIDGE
Hongzhan SHI, Jun DONG, Xuejian XIANG, Dianyuan LIU,Chunguang REN..... 312
THE STEEL-CONCRETE COMPOSITE BEAM STRESS ANALYSIS OF SINGLE PYLON CABLE STAYED BRIDGE PYLON PIER CONSOLIDATION AREA
LONG Peiheng, LU Zechao, ZHANG Guangda, HOU Suwei, JIAO Chiyu..... 317
STUDY ON POST-FIRE SEISMIC PERFORMANCE OF SRC COLUMN-RC BEAM JOINTS
Guohua Li, Long Xie, Dongdong Liu 321
DAMAGE STUDY ON NUMERICAL MODEL OF REINFORCED CONCRETE BEAM BASED ON ABAQUS
Xulong SI, Jun Dong, Dongqing Li, Liyang ZHANG, Hongzhan SHI, Shunchang MA 326
ROCK CUT VERSIONS OF ARMENIAN CHURCHES AND MYTHOLOGICAL AND SOCIAL GROUNDS OF THE ORIGIN OF THEIR ARCHITECTURAL COMPOSITION
Samvel M. Shahinyan..... 329
ELEMENTS OF CAPITALS' ORDER SYSTEM IN THE ARCHITECTURE OF THE ANCIENT ARTASHAT
Aminia Kanetsyan..... 336
THE CHURCHE IN PAINTER SLAVIK PARONYAN`S LANDSCAPES
Lilit A. Arsenyan..... 342
THE ROLE OF ARCHITECTURAL TERMS AND THE PROBLEMS OF THEIR APPLICATION
Artavazd S. Nazaretyan..... 345

THE CONCEPT OF SYSTEMS WITH THE MULTITIERED CIRCULAR ORGANIZATION OF FUNCTIONAL FLOWS
Ashot Yu. Safaryan, Yuri A. Safaryan..... 348

TRENDS OF DEVELOPMENT OF RESIDENTIAL BUILDINGS IN ARMENIA IN THE NEW SOCIAL AND ECONOMIC FORMATION
Emilia M. Sargsyan, Viktoria P. Voskanyan..... 351

ARCHITECTURAL-PLANNING ISSUES OF ORGANIZATION OF OPEN SPACES IN HISTORICAL CITY OF GYUMRI
Aloyan Artyom, Tatev Kartashyan..... 356

EXAMPLE OF ARCHITECTURAL PROJECT OF REHABILITATION OF STONE-DOOR CAVES AND ITS USE FOR TOURISM PURPOSES
Anush Khudoyan, Davit Nahapetyan, Vruyr Enoqyan..... 359

FORMING OF CONTEMPORARY SOCIALHOUSING ESTATES – CHOSEN ISSUES
Mariusz Zadworny..... 362

GEOMETRY OF SACRAL BUILDINGS
Aleksandra Repelewicz 369

ILLUSTRATED CONSTRUCTION EXPLANATORYDICTIONARY AND ITS INFLUENCE ONTHE DEVELOPMENT OF GEORGIANCONSTRUCTION CULTURE
Levan Klimiashvili, Tamaz Khmelidze, David Gurgenidze, Kakhaber Khmelidze..... 375

THE ROLE OF URBAN LOGISTICS IN THE DEVELOPMENT OF SOUTH GEORGIAN TOURISM AND RECREATION CLUSTER
T.K. Khoshtaria, N.T. Chachava..... 378

MONITORING OF BUILDING STRUCTURES BY FIBER-OPTICAL SENSORS
L. Klimiashvili, d. Gurgenidze, U. Dzodzuashvili, M. Tsikarishvili, O Kharshiladze, S.Bilidze..... 385

SEVERAL FEATURES OF BUILDING RECONSTRUCTION
L. Balanchivadze, I. Chanturia..... 388

NUMERICAL MODELING OF VENTILATION FLOW DISTRIBUTION IN SUBWAY TUNNELS WITH CONSIDERING OF THE PISTON EFFECT
Omar Lanchava, Giorgi Nozadze, Zaza Khokerashvili..... 390

EXPLORATION THE POSSIBILITY OF USING OIL PRODUCTION WASTE IN CONSTRUCTION
Nana Dondoladze 393

CURRENT PROBLEMS AND APPLICATION PROSPECTS OF DIFFERENT TYPES OF COMPOSITE REINFORCEMENT BASALT-PLASTIC REBARS FOR CONCRETE AND FOR STRENGTHENING OF STRUCTURES
Joni Gigineishvili, Igor Timchenko, David Gigineishvili..... 397

STRUCTURAL AND ARCHITECTURAL DESIGN IN CONDITIONS OF THE COMPLEX TOPOGRAPHY OF GEORGIA
Johni Gigineishvili[1], Igor Timchenko..... 404

INFLUENCE OF LATTICE SUBSTRATE LONGITUDE ACOUSTIC OSCILLATION ENERGY LOCALISATION ON DEEP CENTRES ON ISLAND GROWTH INTO EPITAXIAL PROCESS
O. Janelidze..... 412

THE CALCULATION OF THE RETAINING WALL LYING ON THE PRESSURE BASE
Teimuraz Kikava 416

UNIVERSITY AS A DRIVING FORCE FOR IMPLEMENTATION OF THE TERRITORIES' DEVELOPMENT STRATEGY
Larionova Viola Anatolievna, Stepanova Natalia Romanovna 419

RATIONAL TECHNOLOGY FOR COLLECTING SPENT REGENERATION SOLUTION OF CATION EXCHANGERS AND ITS REUSE.
Zurab Megrelishvili 423

MONITORING OF BUILDING STRUCTURES BY FIBER-OPTICAL SENSORS
M.*Bediashvili*[*] 426

ANALYSIS OF SANDWICH ORTHOTROP PLATES ON LOCAL LOADINGS
Gela Kipiani..... 429

ERVAND KOCHAR'S RELATIONS WITH GEORGIAN ARTISTS AS REFLECTED IN THE CONTEXT OF ARCHITECTURAL MODERNIZATION OF ERVAND KOCHAR MUSEUM
Karine Kochar..... 433

TRADITIONS AND INNOVATION IN THE ARCHITECTURE OF VASILI MIRZOYAN'S RELIGIOUS STRUCTURES *Marietta A. Gasparyan*..... 436

DEVELOPMENT OF ENGINEERING METHODS FOR CALCULATING COMBINED PILE-RAFT FOUNDATION (CPRF) *Vyacheslav Efimov, AnatoliyOsokin, Lidiya Kondratieva*.. 441

FABRICS STRUCTURE AND ITS REINFORCING EFFICIENCY IN COMPOSITE "TEXTILE-CONCRETE" *Tamar Moseshvili*..... 446

INTERACTIVE ACCULTURATION AND POST-COLONIAL CONSTRUCTION OF URBAN IMMIGRANTS' CULTURAL IDENTITY: VANUATU AS A CASE STUDY *Xuan Wu*..... 450

ON CONTEMPORARY CONNOTATION OF BEIJING TRADITIONAL ARCHITECTURAL CULTURE *LIANG XU*..... 456

STUDY ON SEISMIC RESPONSE OF CANTILEVER STRUCTURE IN CONSTRUCTION OF LARGE CABLE STAYED BRIDGE PYLON. *Bowen ZHANG, Jun DONG, Hongzhan SHI , Dianyuan LIU , Xuejian XIANG*..... 461

STUDY ON TRANSITION STAGE OF NONLINEAR VIBRATION OF UNDERGROUND STRUCRURE UNDER VERTICAL SEISMIC ACTION. *Siran Shen , Chengzhi Qi* , Yunpeng Guo*..... 466

BEARING CAPACITY CALCULATION OF SHEAR WALL BASED ON ARTIFICIAL **NEURAL NETWORK SYSTEM** *LUOJian* [1] ,*LIUJia* [1] ,*CHANGYuan1*,*JIUJINManzi*[1] 473

EXPERIMENTAL INVESTIGATION ON THE PUNCHING CAPACITY OF RECYCLED CONCRETE TWO-WAY SLABS STRENGTHENED WITH CFRP LAMINATES *Luo Jian* [1], *Du Chenxu*[1], *Liu Jia* [1]..... 477

1. Hydraulic Facilities Engineering, Environmental Engineering, Transport Problems Civil Engineering, sustainable construction

Analysis of crack formation and seepage into the foundation of the Enguri HPP dam

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Keywords: arch dam foundation, grout curtain cracking, reservoir drawdown/filling, Griffith criteria finite elements, seepage.

Abstract: Foundation of Enguri HPP dam (height of 271.5 m) has complex geological structure. The bedrock consists of carbonate rock strata of Lower Cretaceous stage, represented by non-uniform, strongly fissured limestone and dolomite. The rock massif is layered with clayey marl at every 40-50 m of depth, with the thickness of the layers ranging from 3-10 cm to 50 cm. This rock actually screens the seepage flow and strengthens the anisotropy of the foundation. There is a 120m-deep grout curtain and drainage arranged on the entire perimeter of the gorge, which play a role in the seepage regime in the dam foundation.

In the dam operation period, the seepage pressure significantly increased behind the grout curtain in the central section, and the pressure dynamics can be tracked back as follows: the first peak occurred in 1989, followed by a certain drop, and then a rise again. Therefore the area was assessed as an anomalous one. Since 1999 to date, when the cracks started to fill with mud, the seepage pressure behind the grout curtain has restored to normal regime parameters.

According to general evaluation, processes ongoing behind the grout curtain are linked to the existence of tensile stresses in the tectonic foundation of the dam, potentially followed by opening of the cracks, damaging of the curtain, and then mud filling of the cracks.

In 2006-2007, grout curtain and drainage rehabilitation works were conducted in the frame of a rehabilitation program, which resulted in reduction of seepage discharge in the dam galleries.

This study aimed to evaluate the anomalous seepage and its causes in the central part of the dam foundation. For this purpose, the study consisted of:

- Analysis of foundation's static stress state and current seepage processes, taking into account the grout curtain behavior, both independently, and together with the drainage, during reservoir filling/emptying (as part of unsteady regime);
- Evaluation of crack formation in view of decreased pressures and friction resulting from the crack mud filling process).

1. Introduction and Background

Foundation of Enguri HPP dam (height of 271.5 m) has complex geological structure. The bedrock consists of carbonate rock strata of Lower Cretaceous stage, represented by non-uniform, strongly fissured limestone and dolomite. The rock massif is layered with clayey marl at every 40-50 m of depth, with the thickness of the layers ranging from 3-10 cm to 50 cm. This rock actually screens the seepage flow and strengthens the anisotropy of the foundation. There is a 120m-deep grout curtain and drainage arranged on the entire perimeter of the gorge, which play a role in the seepage regime in the dam foundation.

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- Evaluation of crack development in view of decreased pressures and friction resulting from the crack mud filling process).

The static and seepage analyses have been conducted using a 3D Finite Element Model (FEM) of the dam-foundation, whereas crack development was evaluated using 2D models based on fracture mechanics approaches. This report presents some results of the study.

2. Geological Structure and Seepage Characteristics of the Dam Foundation

The dam foundation has rather complex geological structure (fig.2.1). There are anticline strata in the central part of the foundation that consist of least permeable soils. A low-permeability breccia stratum located 150m upstream from the dam continues along the existing layers in the foundation, and spreads for 200 meters from the dam base. Thus, the seepage flow sharply narrows at the lower border of the grout curtain, resulting in additional flooding of the rock massif located in front of the curtain, which causes increased readings in piezometers located there. There are tectonic faults in the base that are filled with clayey material and screen back the seepage flow, which explains the difference in readings between piezometers located at some distance from each other.

A comparatively comprehensive analysis of the seepage regime in the dam foundation is possible using measured heads (pressures) in piezometers and seepage discharge measured in grouting adits and galleries. Most of the piezometers are situated in grouting adits, and are mainly located in pairs: in front of and behind the grout curtain.

Monitoring in the foundation after reservoir filling showed progressing chemical and mechanical suffosion (piping). The overall background mineralization of the seepage flow dropped from 1.28 to 0.18 g/l over the monitoring period (1989-1991). Chemical composition of the seepage water is getting similar to that of surface water, which confirms that filling material of seepage paths and cracks existing in the foundation rock is washed out. In the same time, pH increased from 8 to 9.5-11, which generally indicates that concrete leaching is going on [1].

In the period of first reservoir filling (since 1986) when the water level in the reservoir reached 460 meters, there were spasmodic rises of pressures (heads) behind the grout curtain, a trend occurring with increasing frequency compared to previous years (fig. 2). In 1989-1990 (NOL=510 m), monitored pressures in piezometers located in that zone constituted about 80-90 per cent of pressures at the dam, which indicated that the grout curtain was damaged.

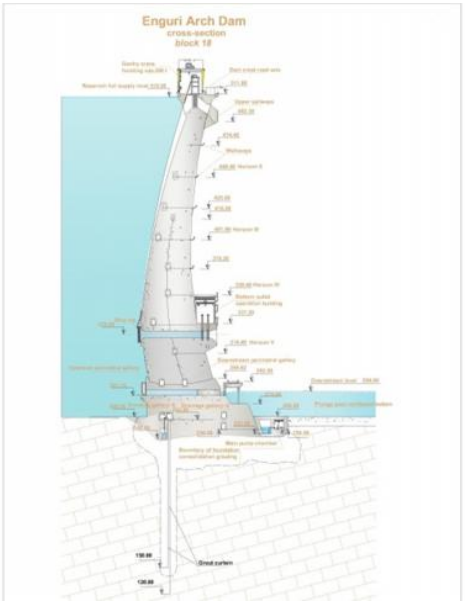


Fig. 2.1 Dam central section.

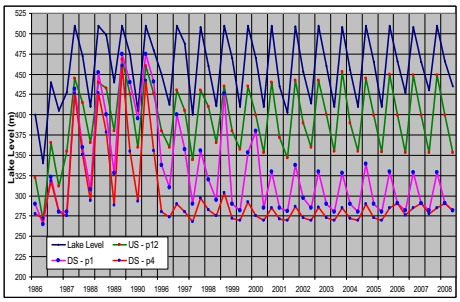


Figure 2.2 Piezometer pressure distribution: p1, p4 - behind the grout curtain; p12 - before the grout curtain.

3. Dam Foundation Analysis Models

This study reflects only processes going on in foundation in the dam centerline section. The crack development and current seepage analysis was conducted using a 2D model taking into consideration results of 3D stressed state model of the dam-foundation based on static-seepage diagram.

With respect to crack development in the foundation, there is a suggestion that when the reservoir is being emptied, the cracks filled with water under the dam pressure face behave under compressive stress and therefore close. At that moment the pressure in the cracks increases compared to that in the surrounding rock, which causes the crack to widen and develop. It is rather difficult to confirm this suggestion using an empirical model. This study demonstrates a crack development model that shows that in the operation period edges of tensile cracks under the pressure dam face are filled and then covered by clayey mud. This increases hydrostatic pressure on the crack surface and decreases friction. When the reservoir is full, the increased pressured inside the crack fosters further cracking, and when the water is at the lowest level, cracks that are subject to compressive stresses (because of low friction on the surface) may also develop further. The fact that the cracks get filled with mud is also reflected in the
piezometer readings (figure 2.2).

The crack analysis was conducted using fracture mechanics approaches. The crack depth was conditioned by the fact that under the influence of operational loads the value of the normal stress intensity at the end of the crack equaled zero.

The foundation seepage analysis for reservoir drawdown/filling was conducted for nonstationary regime. The foundation FEM reflected main defined seepage factors, including existing non-uniform structure of rock in the depth of the massif, the grout curtain, the screening tectonic faults, and the drainage.

The calculations were made under operation loads (dead load, hydrostatic seepage pressures), in the range between the highest and lowest water levels in the reservoir (510-410 meters).

4. Crack Development Analysis in the Dam Foundation

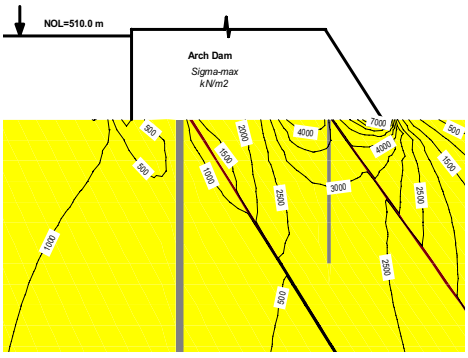
4.1 Crack Development during First Reservoir Filling

Generally, seepage causes loading of the foundation and crack edges, which is reflected in formula: $p=(1-\alpha_2)\gamma h$ (where α_2 is coefficient of effective backpressure area; γ-is water density, h- is water head pressure). Analysis of sediment close to the dam shows that the sediment mainly consists of mud and clayey particles (with high cohesive capacity). It should be noted that the seepage flow transits the sediment particles and causes covering of crack surface, thus decreasing permeability of the cracks α_2 and respectively increasing hydrostatic pressure on crack edges. Piezometer readings confirm that mud filling of cracks is really going on.

Crack analysis was conducted in view of the foundation geology (blocks and tectonic faults).

Calculation outcomes are as follows:

- According to the 3D dam-foundation model, there is a wide range of main stresses in the central cantilever section (Figure 4.1.1). Tensile stresses penetrate deep into the foundation (figure 4.1.2);
- At the water level of 450 m in the reservoir, opening of the dam-foundation contact zone was registered under the pressure face. In the same time the grout curtain was still strong enough (according to pizometer readings, figure 2.2).
- The depth of crack (0-1) reached 7.5 m and its orientation almost followed the dip of strata.



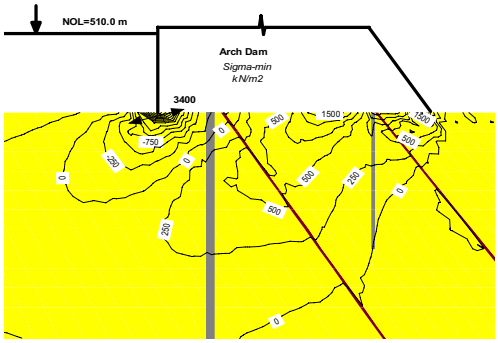


Figure 4.1.1 Main stress distribution in foundation of dam central cantilever (according to 3D model).

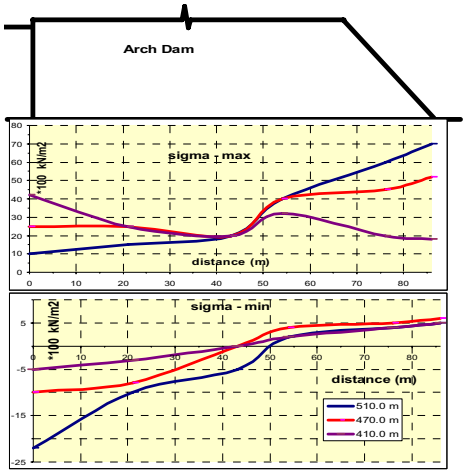


Figure 4.1.2 Main stress distribution in dam-foundation contact.

4.2 Crack Development during Reservoir Drawdown Low level.

The inclined crack located under the pressure face of the dam (0-1) is subject to high compressive stress during reservoir drawdown, so that the crack shear resistance needs to be investigated. According to Griffith criteria modified by McClintock and Walsh, the critical stress (figure.4.2.1), is defined as follows [2,3]:

$$\sigma_{1,cr} = \frac{-4T + \sigma_2(\sqrt{(1+\mu^2)}+\mu)}{\sqrt{(1+\mu^2)}-\mu}$$

where T - is ultimate tensile strength;

σ_2 - is the minimum compressive stress;

μ - is cohesion factor in the shear zone.

Cracks can be evaluated using safety factor:

$$\eta = \frac{\sigma_{1,cr}}{\sigma_{1,c}}$$

Where $\sigma_{1,c}$ is the calculated value of the maximum compressive stress. If $\eta \succ 1$, the shear crack retains its stability, but at $\eta \leq 1$ it starts developing.

Sources [2] and [5] show that the friction factor for concrete and rock ranges within $\mu = 0.6 \div 0.9$. During the mud-filling process the friction decreases on the 0-1 crack surface compared to normal friction conditions. Because of lack of clarity in this respect, the calculations have been made for the range of $\mu = 0.2 \div 0.9$.

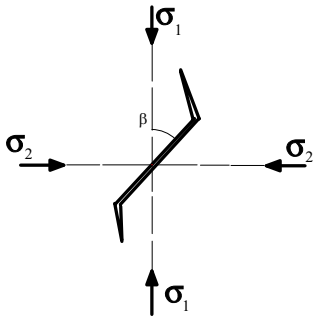


Figure 4.2.1 shear crack development scheme.

The calculations show that in the interval of $\mu = 0.2 \div 0.8$ the safety factor crack developed in foundation up to 4 meters (Fig.4.2.2).

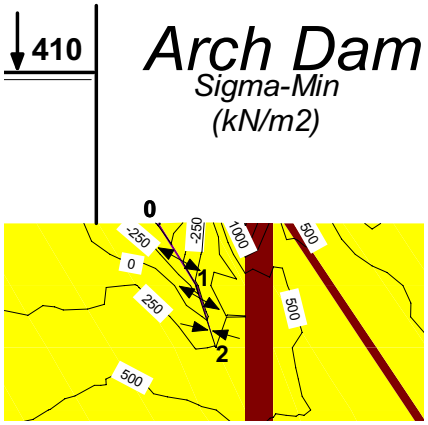


Figure 4.2.2 Crack development dynamics in the dam foundation (under compressive stress) during reservoir low level.

4.3 Crack Development in the Foundation during Subsequent Reservoir Filling

During subsequent reservoir filling, the crack became longer compared to the previous time (0-1-2), and the surfaces were partly covered with mud (as confirmed by peizometer data), which respectively caused decrease of α_2 (figure 4.3.1). According to standards, the recommended value of α_2 for rock massif is $\alpha_2 \approx 0.5$. Because of unclear α_2 related to the mud-filling process, a range of $0 \prec \alpha_2 \prec 0.5$ was considered for the crack analysis. In the same time, the hydrostatic pressure on the crack edges may double.

The calculations show that crack started to develop at $\alpha_2 = 0.1 \div 0.2$. The crack actually damaged the grout curtain. The overall length of the 0-1-2-3-4 crack was 12 meters (figure 4.3.2).

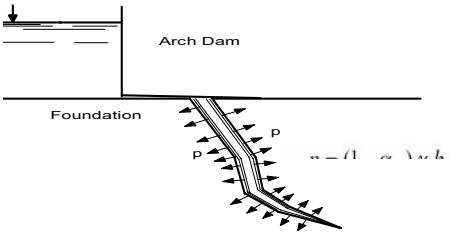


Figure 4.3.1 Crack modeling during mud filling.

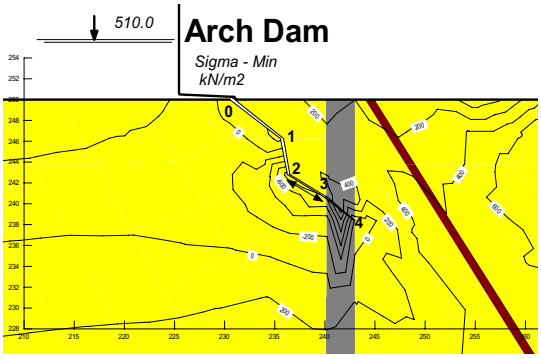


Figure 4.3 2 Crack development dynamics in the dam foundation during Subsequent Reservoir Filling.

0-1 – tensile developed during first filling of the reservoir (NOL=510m);

1-2 – shear crack developed during drawdown (MinOL=410m);

2-3-4 – tensile crack developed during re-filling (NOL=510m).

5. Seepage Regime in the Dam Foundation

5.1 Pressures in cases of Normal and Damaged Curtain

In view of the foundation geology, also behavior of the grout curtain and drainage, the obtained pressure distribution (Figure 5.1.1) well corresponds to measurements made in piezometers. Dam foundation non-uniformity, in particular, screening tectonic faults, have a significant impact of the seepage regime. Under the influence of drainage, pressure in the area in front of it decreased by up to 20 per cent (figure 5.1.2). After the contract crack of 0-1-2-3-4 in the foundation damaged the grout curtain, the seepage regime sharply changed (figure 5.1.3), which also corresponds to results of piezometer measurements.

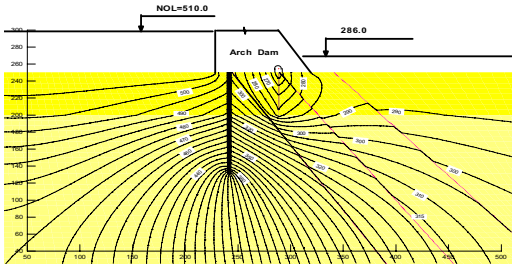


Figure 5.1.1. Pressure distribution in the dam foundation with the intact grout curtain and drainage in place.

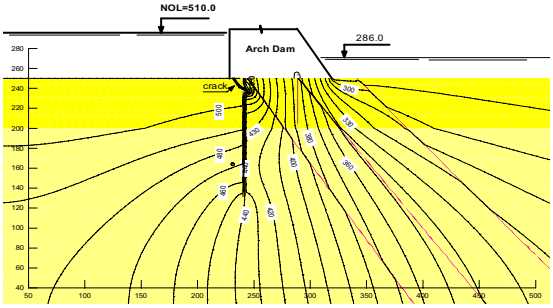


Figure 5.1.2. Pressure distribution in the dam foundation with the damaged grout curtain and drainage.

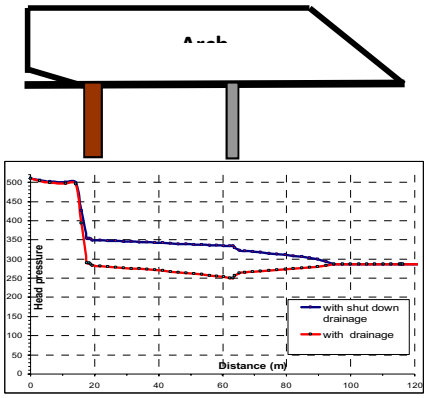


Figure 5.1.3. Pressure (head) distribution in the dam-foundation contact.

5.2. Seepage Processes during Reservoir Drawdown/Filling

During refilling of reservoir after drawdown (under nonstationary regime), seepage pressure

gradients increase on the foundation surface [6]. Thus the process prevents opening of the dam-foundation contact under the pressure face, which has been taken into account during static calculations.

During reservoir drawdown (in the area where the thickness is about 6m), back-seepage takes place: the flow moves bottom up (Figure 5.2.1). Pressure gradients at the foundation surface reach 2.4, which may cause washing of the open contact under the pressure face and the cracks.

There should be noted that in operating period, due to energy crisis in Georgia (1990 - 1998) and poor functioning of the dam bottom outlet, reservoir filling-drawdown rate was increased. The mentioned could have negative influence on upon distribution process in foundation.

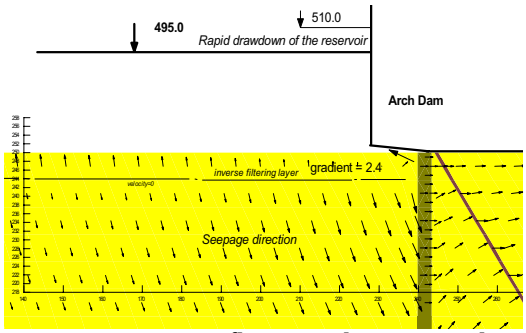


Figure 5.2.1 Seepage flow speed trajectory during drawdown.

Conclusions

- The damage of the dam-foundation contact and the grout curtain of the Enguri HPP dam (detected in 1987) was caused by joint dam-foundation behavior under static influence of operational loads. Development of cracks and associated damage of the curtain is indentified on different dams, so the issue is pressing indeed. The presented analysis model could be used as a general methodology for different dams.
- Mud-filling of cracks results in increased hydrostatic pressure and decreased friction on the crack surface. Thus until the mud-filling process was completed, high compressive and tensile stresses occurred at high and low water levels in the reservoir, which fostered further crack development. Crack modeling in the mud-filling process was based on fracture mechanics. The model was used to understand the dynamics of crack development and current seepage processes in the Enguri dam foundation during reservoir filling and emptying. The model was used to analyze opening of the dam-foundation contact and damaging of the grout curtain.
- The foundation anisotropy, especially the geological cracks (filled with clay), has a significant impact on seepage processes and crack development:

- Cracks screen back the seepage flow, respectively distributing pressures, and thus increasing the pressures – a fact that has been taken into consideration when analyzing piezometer data.
- Structural orthotropy of rock mass has increased tensile and shear crack development during reservoir filling/emptying.

- During rapid filling (under the influence of non-stationary seepage regime), there is an additional load on the foundation surface, which fosters contact opening under the pressure face of the dam.

During drawdown, the flow in the upper layer of the foundation (about 6m) moves backwards: from the foundation to the reservoir, which creates conditions for contact and crack washing.

- In 1997, the cracks started to fill with mud, which then became irrevocable. Rehabilitation (2006-2007) of the grout curtain minimized seepage in the foundation. At present the seepage regime in the foundation is stabilized.

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TOTAL ENERGY BALANCE OF WINDOWS IN DEPENDENCE ON THEIR STRUCTURAL DESIGN

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Keywords: Total energy balance, heat losses, thermal solar gains, window.

Abstract
The article compares the total energy balance of the windows that differ from each other by design solutions (frameless system, wooden and aluminium frame, facade system), the type of glazing chosen (double-glazed, triple-glazed, quadruple-glazed window) and area size (3 variants). The calculation of the total energy balance of compared types of window systems using three different calculation methods according to valid standards is evaluated in the paper.

1 Introduction
Windows are one of the important elements affecting the total energy balance of buildings. The use of passive solar gains in buildings belongs to the major factors influencing the low energy performance of buildings, especially in passive and zero standards. Appropriate choice of structural and material solution of windows can fundamentally affect the total energy balance of buildings[1].
The subject of the article is the calculation and comparison of the total energy balance of the windows, which differ from each other by the surface, material, constructional solution and the type of glazing. A comparison of the energy balance for four variants of the design of the windows (frameless system, wooden and aluminium frame, facade system), for three types of glazing (double-glazed, triple-glazed, quadruple-glazed window) and three variants of dimensions (1200 x 1200 mm, 3000 x 3000 mm, 3000 x 6000 mm) is carried out in the article. Altogether 11 types of windows are compared.
The overall energy balance is evaluated for heating and summer periods. Three computational methods are selected to compare the total energy balances of each window type.
The first method of calculation according to ČSN EN ISO 14438 [2] includes only the effect of the glazing in the assessment of the total energy balance and does not take into account the influence of the window frames. The energy balance value is based on 1 m^2 of glazing area, so it also does not take into account the influence of the size of the surfaces of the windows being compared.
The second method of calculation is based on the same methodology according to ČSN EN ISO 14438 [2], but adapted according to the literature [5]. The calculation of the total energy balance of the windows is determined from the sum of the heat losses and usable thermal solar gains. The calculation includes the influence of the area of the entire window, the effect of the window frame is included in the calculation by the correction factor.
The third method of calculation according to ČSN 730548 [4] solves in detail only the thermal gains of the windows on the set day in the summer period.

2 Properties of calculated windows
The basic technical properties of the glazing and windows frames are compared in Table 1.
Table 1 Properties of calculated windows

Numb.	Windows	Area S_w [m^2]	Area of glazing A_g [m^2]	U-value of glazing U_g [W/m^2K]	Area of frame A_f [m^2]	U-value of frame U_f [W/m^2K]	U-value of window U_W [W/m^2K]	Frame correction factor [-]	Solar gain factor g [-]
	Dimensions 1200 x 1200 mm								
1	Frameless quadruple-glazed window	1,44	1,44	0,30	0	0	0,57	1,00	0,49
2	Wooden double-glazed window	1,44	0,91	1,00	0,53	1,4	1,26	0,63	0,52
3	Wooden triple-glazed window	1,44	0,91	0,60	0,53	0,9	0,85	0,63	0,54
4	Aluminium double-glazed window	1,44	1,03	1,10	0,41	1,92	1,64	0,72	0,59
5	Aluminium triple-glazed window	1,44	1,03	0,60	0,41	1,88	1,27	0,72	0,51
	Dimensions 3000 x 3000 mm								
6	Frameless quadruple-glazed window	9,00	9,00	0,30	0	0	0,41	1,00	0,47
7	Facade system with double glazing	9,00	8,41	1,10	0,59	1,75	1,28	0,93	0,58
8	Facade system with triple glazing	9,00	8,41	0,60	0,59	1,68	0,81	0,93	0,52
	Dimensions 3000 x 6000 mm								
9	Frameless quadruple-glazed window	18,00	18,00	0,30	0	0	0,38	1,00	0,47
10	Facade system with double glazing	18,00	17,11	1,10	0,89	1,67	1,24	0,95	0,57
11	Facade system with triple glazing	18,00	17,11	0,60	0,89	1,67	0,76	0,95	0,51

Parameters of thewindows are considered in accordance with the technical documentation of individual manufacturers and suppliers. Other parameters and constants entering into the calculation are determined in accordance with the relevant standards and they are described in the following sections.

3 Calculation of the total energy balance of glazing during a specified period according to ČSN EN ISO 14438 [2]
The calculation is carried out in accordance with the methodology set out in ČSN EN ISO 14438 [2]. This method is designed to evaluate the heat loss balance and recoverable heat gains from solar radiation entering into the building for a specified period using

the average rate of heat losses (or gains), so-called energy balance values.

The energy balance E [W/m^{2}K] is determined for the given period according to the following equation:

$$E = U_g - \frac{\eta \times g \times f \times H_p}{D_p} \quad (1)$$

where:

U_g… is the heat transfer coefficient of the glazing [W/m^{2}K];

η ... heat gain factor [-], (used η= 0,6);

f ... glazing maintenance factor and shading effects [-], (used$f$ = 0,8);

g ... solar gain factor[-];

H_p… unshielded incident solar radiation during a given period [kWh / m^2], (values taken for southern window orientation from TNI 73 0331 [3];

D_p… sum of day degrees for the given period, (the values were taken for the southern orientation of the windows from TNI 73 0331 [3]).

The total energy balance of the glazing E [W/m^2.K] was calculated for the heating period (September to May) and the summer period (June to August). The results of the calculation of the energy balance of the glazing for the individual types of evaluated windows are shown in Table 2 and Table 3. Figure 1 shows the results of the calculation of the energy balance of glazing for the heating period. The results for individual types of rated windows are sorted in descending order according to the most favorable result.

Table 2 The total energy balance of the glazing for the heating season

Number of windows	1	2	3	4	5	6	7	8	9	10	11
U_g	0,3	1,0	0,6	1,1	0,6	0,3	1,1	0,6	0,3	1,1	0,6
η	0,6	0,6	0,6	0,6	0,6	0,6	0,6	0,6	0,6	0,6	0,6
g	0,49	0,52	0,54	0,59	0,51	0,47	0,58	0,52	0,47	0,57	0,5
f	0,8	0,8	0,8	0,8	0,8	0,8	0,8	0,8	0,8	0,8	0,8
H_p	559	559	559	559	559	559	559	559	559	559	559
D_p	3413	3413	3413	3413	3413	3413	3413	3413	3413	3413	341
E	-1,3	-0,7	-1,2	-0,8	-1,1	-1,2	-0,8	-1,1	-1,2	-0,8	-1,1

Table 3 Total energy balance of the glazing for the summer season

Number of windows	1	2	3	4	5	6	7	8	9	10	11
U_g	0,30	1,00	0,60	1,10	0,60	0,30	1,10	0,60	0,30	1,10	0,60
η	0,60	0,60	0,60	0,60	0,60	0,60	0,60	0,60	0,60	0,60	0,60
g	0,49	0,52	0,54	0,59	0,51	0,47	0,58	0,52	0,47	0,57	0,51
f	0,80	0,80	0,80	0,80	0,80	0,80	0,80	0,80	0,80	0,80	0,80
H_p	249,7	249,7	249,7	249,7	249,7	249,7	249,7	249,7	249,7	249,7	249,7
D_p	60,00	60,00	60,00	60,00	60,00	60,00	60,00	60,00	60,00	60,00	60,00
E	-40,5	-42,3	-44,4	-48,0	-41,9	-38,8	-47,2	-42,7	-38,8	-46,3	-41,9

Positive energy balance means thermal loss through glazing, negative energy balance means heat gain for the whole evaluated period. It is clear from the results that the most favorable energy balance for the heating season is given by the glazing with four glasses (1, 6, 9), followed by windows with triple glazing (3, 8, 5, 11) and double glass (4, 7, 10, 2). On the other hand, the energy balance of glass glazing in summer is better compared to other types of glazing - heat gains are the lowest.

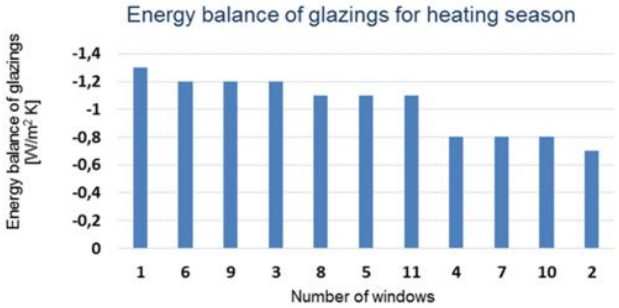

Fig. 1 The total energy balance of the glazing for the heating season

The decisive criterion for achieving the optimal energy balance of the glazing is a suitable combination of the heat transfer coefficient of the glazing and the solar factor. Figure 2 shows the course of the energy balance of the glazing in dependence on the combination of the value of the heat transfer coefficient of the glazing U_g and the solar factor of the glazing g for the heating period.

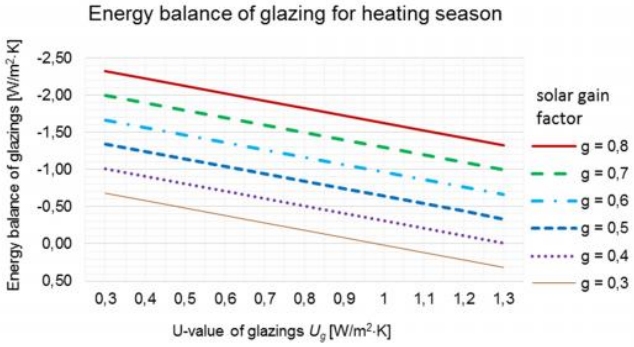

Fig. 2Effect of heat transmission coefficient $U_g$and solar factor g on the energy balance of glazing

4 Calculation of total energy balance of widows for a specified period according to the modified methodology ČSN EN ISO 14438 [2]

To take into account the influence of the size of the window area and the frame type in the calculation of the heat losses Q_l in W, the thermal solar gains Q_g in W and the total energy balance Q_{bil} in W, a methodology derived from ČSN EN ISO 14438 [2] was used, which was further adjusted according to the literature [5].

A relationship was used to calculate the total heat loss through window Q_l in W for a specified period

$$\boldsymbol{Q_l = U_w \times S_w \times \Delta t} \quad (2)$$

where:

U_w… is the coefficient of heat transfer through the window [W/m^{2}K];

S_w… window area including frame [m^2];

Δt ... difference between average indoor and outdoor temperatures [K].

To calculate the solar thermal gains through window Q_g in W for a specified period there is used the following relationship [4]:

$$Q_g = S_w \times k_r \times g \times Q_s \quad (3)$$

where:

S_w… is window area including frame [m^2];

k_r... frame correction factor [-];

g... solar gain factor [-];

Q_s... total intensity of incident solar radiation over a given period [W/m^2].

The total energy balance of windows in W is determined as the difference between heat losses and usable solar gains:

$$Q_{bil} = Q_l - k \times Q_g \quad (4)$$

where:

k ... *i*s solar recovery rate [-], (used k = 0,6 [2]).

Q_{bal}energy balance values in W are calculated for each size of evaluated windows for heating and summer periods, results are shown in Table 4. Fig. 3 shows a comparison of the total energy balances of individual windows of the size 1,2 x 1,2 m.

Table 4 Total energy balance of windows for heating and summer season

Number of windows	1	2	3	4	5	6	7	8	9	10	11
Q_{bil} [W] heating season	**-25,8**	-1,4	-9,7	-1,7	-4,2	**-170,3**	-104,1	-131,4	**-347,3**	-219,2	-274,
Q_{bil} [W] summer season	-15,6	**-9,6**	-10,4	-12,4	-10,9	**-94,3**	-103,3	-94,6	**-188,9**	-207,9	-190,

The calculation does not take into account the correction factor of the frames in the total window area, but only the sunny surface of the window S_{os} [m^2], which is reduced by the length of the shadows from the edges of the sunshades (including the window lining). The same window depths are considered for the side and top lining in the calculation.

The results of the total heat gainscalculationof the evaluated windows facing south for the time 12:00 on 21 July are shown in Table 5.

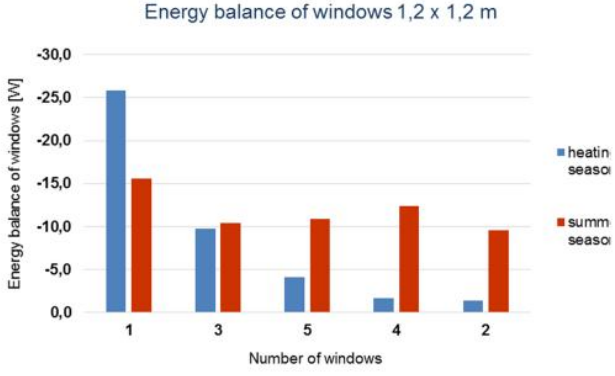

Fig. 3 The total energy balance of windows with dimensions of 1,2 x 1,2 m

In the heating period, the influence of the low heat transfer coefficient, the window glazing area size and the window frame correction factor are favorably reflected on the energy balance. It is clear from the above results that the best energy balance for the heating period is based on the area of the glazing: glazing with four glasses (1, 6, 9), windows with three glasses (3, 5, 8, 11) and double glazing (4, 2, 7, 10). On the contrary, the size of the glazing area is a disadvantage in summer, but it is also necessary to take into account the solar factor, which also plays a significant role in the energy balance.For the summer season can be written the following conclusions:

- For the windows 1,2 x 1,2 m, the energy balance for the windows with insulating double glazing and triple glazing in the wooden frame (2, 3) is the most favorable, followed by windows with insulating double glazing and triple glazing in a metal frame (5, 4) and frameless windows with insulating quadrilateral (1);
- For the windows 3,0 x 3,0 m, the situation is the opposite. The most advantageous energy balance is based on a frameless frame with insulating quadrilateral (6), followed by a window with insulating triple glazing (8) and insulating double glazing (7).
- For the windows 3,0 x 6,0 m, the energy balance results are similar. The most advantageous energy balance is based on a frameless frame with insulating quadruple-glazed window (9), followed by a window with insulating triple glazing (11) and insulating double glazing (10).

4 Calculation of the heat gains of the windows during the specified day in summer according to ČSN 730548 [4]

In ČSN 730548 [4] a detailed procedure for calculating the heat load and heat gains of the windows in the summer season is described. Calculation of heat gains from the external environment is done in most cases for a sunny day on July 21, when the greatest heat gains can be expected. In this article rated windows are considered oriented to the south and as the worst situation was chosen time at 12:00, when the intensity of solar radiation is greatest.

The heat flow of the window has two components in terms of calculation: convection heat transfer and heat transmission through solar radiation. The total heat gains of the windows Q_z in W are determined as the sum of the thermal gains through the heat transferQ_{ok} and the thermal gains through the window radiation Q_{or}.

Table 5 Total thermal gains of evaluated windows on 21.7. at 12:00

Number of windows		1	2	3	4	5	6	7	8	9	10	1
S	[m²]	1,44	1,44	1,44	1,44	1,44	9,00	9,00	9,00	18,00	18,00	18
S_{os}	[m²]	1,34	1,34	1,34	1,34	1,34	8,74	8,74	8,74	17,74	17,74	17
U_w	[W/m².K]	0,57	1,26	0,85	1,64	1,27	0,41	1,28	0,81	0,38	1,24	0,
g	[-]	0,49	0,52	0,54	0,59	0,51	0,47	0,58	0,52	0,47	0,57	0,
s	[-]	2,18	2,18	2,18	2,18	2,18	2,18	2,18	2,18	2,18	2,18	2,
$t_e - t_i$	[K]	1,90	1,90	1,90	1,90	1,90	1,90	1,90	1,90	1,90	1,90	1,
Q_{ok}	[W]	1,56	3,45	2,33	4,49	3,47	7,01	21,89	13,85	13,00	42,41	25
Q_{or}	[W]	337,1	357,8	371,1	407,1	353,2	2083,2	2570,7	2304,8	4207,9	5103,2	45
Q_z	[W]	338,7	361,2	373,5	411,5	356,7	2090,2	2592,6	2318,6	4220,9	5145,6	45

It is clear from the above results, thatthe total heat gains are most affected by the sunlit window area and the solar factor value at calculations of the heat load by solar radiation (and the heat transfer). The influence of the frame (window design) in the calculations does not play a role.

5 Conclusions

Three calculation methodologies have been chosen to compare the total energy balances of each window type, differing in the approach to calculate the heat gains of the windows. The conclusions of the energy balance of the windows can be summarized as follows:

A) Rating of the total energy balance of glazing according to ČSN EN ISO 14438 [2].

Within the methodology, only the effect of glazing is included in the calculation of the energy balance and the effect of window frames is not taken into account. The energy balance value is based on 1 m^2 of glazing area, so the impact of the windows is not taken into account. The energy balance of the glazing of the 11 types of windows for the heating period is best for insulating quadruple-glazed windows, than the insulating triple-glazed windows and the double-glazed windows. The decisive criterion for achieving the positive (negative) energy balance of the glazing for the heating season is a suitable combination of a low value of the heat transfer coefficient of the glazing and the solar factor.

B) Evaluation of the total energy balance of the windows according to ČSN EN ISO 14438 [2], adapted according to the literature [5].

Under the modified methodology, the calculation of the total energy balance of the windows is determined from the sum of the heat losses and usable thermal solar gains. The calculation includes the influence of the area of the entire window, the effect of the window frame is included in the calculation by the correction factor. The energy balance of the glazing for the heating period of the 11 evaluated types of windows is best for the frame without glazing with the insulating quadruple (the largest area), the insulating triple glazing and the double glazing. On the other hand, the impact of larger areas will affect the energy balance of the windows (1200 x 1200 mm) unfavourably in summer. The lowest heat gains occur in windows with a wooden frame, with a metal frame and frameless windows with an insulating quadruple is the least favourably. Another situation is at the windows of the facade system, here and in the summer period frameless quadruple-glazed windows show the lowest thermal gains due to the low solar factor.

C) Evaluation of the heat gains of the windows during the specified day in the summer season according to ČSN 730548 [4].

The calculation does not take into account the correction factor of the frames in the total window area, but only the sunny surface of the window S_{os}[m^2], which is reduced by the length of the shadows from the edges of sunshades(including window lining). The thermal gains of the 11 types of windowsare most favourable (for July 21 at 12:00 with the windows facing south) for insulated quadruple windows, with insulating triple glazing and double glazing. Low heat gain values are positively influenced by the low solar factor value and consequently the value of the shielding factor.

The evaluation results in this article have been applied only to the 11 types of windows with given technical parameters. In the case of other window technical parameters, the results of the energy balances can be different.

6 Acknowledgments

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Legal Problems of Providing Transport Accessibility of Social Infrastructure Facilities for Emergency Response Services in the Transportation Security System

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Keywords: legal support of transportation security, emergency response services, transport infrastructure

Abstract

The goal of the study is to determine the role and importance of providing transport accessibility of social infrastructure facilities for emergency response services in the transportation security system and justifying (on the basis of obtained results) the need to take legal and other measures in the transport sector.

To achieve the specified goal, the following tasks were formulated:

-to reveal the essence, specificity and content of the concept "transportation security system";

-to determine the content and give a description of the concept of "emergency response services";

-to reveal the content and give a description of the concept of "social infrastructure facilities";

-to determine the role and significance of providing transport accessibility of social infrastructure facilities for emergency response services in the transportation security system;

-to justify the need to take legal and other measures in the transport sector, aimed at ensuring transportation security;

-to determine main directions of international cooperation in the considered field.

Scientific novelty and originality of the study consist in the detection of destructive factors, processes and patterns that conditioned the importance of transport accessibility of social infrastructure facilities for emergency response services to ensure transportation security.

Relevance of the study is conditioned by the need to ensure transport accessibility of the specified facilities in order to ensure transportation security.

A distinctive feature of this work is use of the security theory methodology in the study of legal problems of ensuring transport accessibility, that allows taking into account the threats to transportation security, as well as the need to assess the vulnerability of transport infrastructure facilities and vehicles.

1 Introduction

In the modern world, under conditions of transport development and the need to improve the transport infrastructure, the problem of ensuring transportation security becomes particularly acute. The main reason of this, in particular, is the growth of megacities, increase in the total number of transport vehicles, the need of counteraction to terrorist threats, etc.To the utmost, problems of ensuring transportation security show themselves in activities of emergency response services, as well as in cases of the occurrence of the corresponding threats in crowded places.

In this regard, the possibility of using legal tools to solve the problem of providing transport accessibility of social infrastructure facilities is of scientific and practical interest.

2. Results

Scientific publications contain various opinions concerning the security as a phenomenon, as well as numerous approaches to the definition of the basic "security" concept. This circumstance is to some extent a cause of existence of various interpretations of the transportation security concept.

Considering the problem of ensuring transportation security from the perspective of legal regulation, legal definitions should be used as initial concepts. In this connection, the present study comprises statutes of the Russian legislation regulating legal relations in the considered field.

Legal definition of the transportation security concept is represented in Clause 10 of Article 1 of Federal Law No. 16-FZ dd. 09.02.2007 "On Transportation Security" [4], according to which it is a state of protection of transport infrastructure facilities and transport vehicles from unlawful interference acts.

In our opinion, this definition has the narrow industry-specific and pragmatic character, which makes it convenient in law enforcement. From the point of view of the axiological approach to the transportation security theory, main valuables here are the transport infrastructure and transport vehicles. At the same time, interests of transportation security subjects (interests of individuals, the society and the state in the field of the transportation industry) are determined as the objectives of ensuring transportation security (Clause 1 of Article 2 of the Federal Law "On Transportation Security"), and safeguarding of interests of individuals, the society and the state is one of the basic principles of ensuring transportation security (Article 3 of the Federal Law "On Transportation Security").

The Russian legislation proceeds from the need of integration in international security systems. However, the legislator (in particular, in the Federal Law "On Transportation Security") does not reveal the content of the concept of the transportation security system, without which such integration is impossible, since it cannot be implemented in modern conditions beyond the legal field.

In our opinion, the transportation security system should be considered as a functional system, in the framework of which the interaction of interests and threats is carried out. It should be noted that this system differs

significantly from the system of ensuring transportation security. The latter is an organizational system, which represents a set of forces and means, as well as measures of legal, organizational, economic, social, technical and other character aimed at implementing the state policy in the field of transportation security.

The transportation security system shall be formed around the transportation security objects, i.e. vital interests of individuals, the society and the state. At the same time, the individual, the society and the state are transportation security subjects. They have to confront various threats arising in the considered area.

Emergency response services play the important role in ensuring transportation security and security in general. They are intended to receive calls and messages about accidents and emergency situations from the population and organize the emergency response of appropriate forces and means, if necessary. The main task of the mentioned services can be formulated as saving people and property, providing first aid in emergency situations.

In virtue of Clause 1 of Article 52 of Federal Law No. 126-FZ dd. 07.07.2003 "On Communications" [2], emergency response services include, among others, fire response, police, emergency medical service, emergency gas service. Taking into account provisions of Decree No. 894 of the Government of the Russian Federation dd. 31.12.2004 "On Approval of the List of Round-the-clock and Free Emergency Response Services, Communication Operators of Which are Obliged to Response to Calls of Users of Communication Services, and on the Assignment of a Single Emergency Response Phone Number" [3], the list of Russian emergency response services should be supplemented by the emergency responding agency and the "Antiterror" service.

In accordance with the Federal Target Program "Establishment of a System for Provision of Emergency Calls Via Single Phone Number "112" in the Russian Federation for the Period of 2013–2017" (approved by Decree of the Government of the Russian Federation No. 223 dd. 16.03.2013" [6]), one of the most important factors in ensuring the national security of the Russian Federation under conditions of the high level of risks of anthropogenic and natural disasters, negative consequences of emergency situations for sustainable social and economic development of the country is increase of the protection of the population, territories and potentially dangerous facilities.

The life and health care of citizens, the protection of property, the provision of personal and public security, as well as the need to confront threats of anthropogenic and natural character, and to counteract acts of terrorism dictate the need to increase the response efficiency of emergency response services.

Against the background of fundamental changes in social and economic conditions of the population life-sustaining activity: the expansion of cargo and passenger traffic, the growth of population density in cities, increase of the number of crowded places, increase in the number of terrorist threats constantly enhance requirements for the promptness and efficiency of responding to incoming calls from the population (messages about accidents).

It is the common practice to refer the following objects to the social infrastructure: residential, public, sports and industrial buildings and structures, places of rest, etc. In our opinion, a number of distinctive features is characteristic for such facilities. First of all, they have a specific purpose — to meet various public needs in vital goods, works and services. Secondly, all of them are crowded places. Besides, social infrastructure facilities often directly adjoin transport infrastructure facilities. And in some cases, social infrastructure facilities can be considered as objects of the transport infrastructure (for example, railway stations or automobile stations).

According to the Federal Target Program "Establishment of a System for Provision of Emergency Calls Via Single Phone Number "112" in the Russian Federation for the Period of 2013–2017" the problem of prompt and effective response to incoming calls to emergency services has become particularly acute in the recent time due to non-compliance of the existing response system to the needs of the society and the state, as well as the inadequate effectiveness of its functioning, the inadequate level of the staff readiness to work upon interaction of several emergency response services, low awareness of the public about actions during accidents and emergency situations.

The experience of emergency response services shows that, for the effective rendering of help in case of accidents or emergency situations, engagement of more than one emergency response service is needed in 10 percent of cases.

The relevance of the problem is stipulated by maintenance of a significant number of dead and injured people, as well as by significant direct and indirect damages from accidents and emergency situations. For example, according to the Federal State Statistics Service, the number of deaths from external causes was 216,867 people in 2010.

It should be noted that accidents and emergency situations that require a comprehensive response have the most serious consequences.

The most important effectiveness indicator for actions of emergency response services is time of their response to them. Its reduction directly affects severity of consequences of an accident or emergency situation (reduction of the number of dead and injured, as well as reduction of overall material damage).

Insufficient level of interaction organization starting from the time of the call receipt (message about an accident) to rendering help to victims with the involvement of several emergency response services is one of the main causes of high mortality in accidents and emergency situations.

Solving problems of increase of the promptness and effectiveness of responding to calls from the population (messages about accidents) largely depends on the speed of emergency response, especially with regard to social infrastructure facilities, since they are considered crowded as it was noted above. Therefore, it is especially important to ensure transport accessibility of these facilities for emergency response services. It is, in our opinion, directly related to ensuring of transportation security.

Apparently, ensuring of transport accessibility should be considered as one of the most important parameters determining the level of ensuring of transport security. Obviously, the lack of transport accessibility to social infrastructure facilities has a negative impact on transportation security, since it increases vulnerability of transport infrastructure facilities and vehicles.

The assessment of vulnerability of transport infrastructure facilities and vehicles is carried out in accordance with Decree of the Ministry of Transport of the Russian Federation No. 87 dd. 12.04.2010 "On the Procedure for Assessment of Vulnerability of Transport Infrastructure Facilities and Vehicles" [1]. The vulnerability assessment includes, among others, the following:

- study of technical and engineering characteristics of the transport infrastructure facility or vehicle (including geological, hydrological and geographic features of location of the transport infrastructure facility), and organization of their operation (functioning);
- study of the system of measures approved at the transport infrastructure facility or vehicle to protect against acts of unlawful interference;
- study of methods of potential threats and acts of unlawful interference into the operation of the transport infrastructure facility or vehicle using aggregate data on the number, equipment, availability, awareness, and actions of potential violators, objectives pursued during commission of an act of unlawful interference into the operation of the transport infrastructure facility or vehicle (hereinafter referred to as the violator's model);
- making recommendations for the subject of the transport infrastructure with regard to measures which need to be additionally included in the system of measures to ensure transportation security of the transport infrastructure facility and/or the transport vehicle.

However, order No. 87 of the Ministry of Transport of the Russian Federation dd. 12.04.2010 does not envisage the study of transport accessibility ensuring. The necessary level of transport accessibility of social infrastructure facilities is not legislatively established.

The Federal Target Program "Establishment of a System for Provision of Emergency Calls Via Single Phone Number "112" in the Russian Federation for the Period of 2013–2017" states that organizational and technical measures are required, including organization of an integrated response system, designing and arrangement of information and telecommunications infrastructures, subsystems of call receiving from the population and their processing (accident reports), storage and updating of databases, decision-making support, advisory services for the population, monitoring of potentially dangerous stationary and moving objects, a geo-information subsystem in order to reduce the average time for a prompt response of emergency response services and effective organization of the assistance to victims.

Achievement of transport accessibility goals involves, as a rule, solving of problems of the development of transport corridors, the transport and engineering infrastructure, the renewal of vehicle fleets, etc. Often this implies the need to attract significant financial, labor and other resources. However, solving of these tasks does not guarantee the achievement of the stated goal.

For example, when solving problems of increasing of transport accessibility to the social infrastructure facility located in the so-called "historical center" of a megacity (with characteristic narrow streets and, as a consequence, frequent congestion of vehicles, significantly reducing the capacity of roads), the use of just technical and engineering methods can be insufficient. In such situation, it would be probably necessary to use legal, procedural and institutional methods (including, adoption of legal decisions about temporary restrictions or cessation of the traffic).

It should be noted that attempts with regard to certain types of social infrastructure facilities are made to provide legal groundworks for transport accessibility (for example, ensuring of transport accessibility to medical organizations for all groups of the population, including disabled people and other groups of people with the limited mobility is established in Clause 7 of Article 10 of Federal Law No. 323-FZ of 21.11.2011 "On Basics of the Health Care of People in the Russian Federation" [5]). However, it is clearly not enough, since the lack of transport accessibility to social infrastructure facilities creates weaknesses in the transportation security system.

Thus, the significant attention should be paid to the legal provision of transportation security. In this respect, it is required to carry out work for the improvement of the legislation in the considered field. This refers not only to legal and regulatory securing of the need to determine transport accessibility during assessment of vulnerability of transport infrastructure facilities and transport vehicles. It is necessary to monitor the considered areas on the regular basis, having preliminarily set indices of the level of transport accessibility.

3 Conclusions

Ensuring of transport accessibility should be considered as the strategic priority in the activity for transportation security provision. In particular, the high level of transport accessibility is a prerequisite for prompt and effective response of emergency response services.

Tasks of ensuring transport accessibility shall be solved using legal, procedural and organizational methods as well.

In this connection, the international experience exchange is of great interest, including the legal experience in solving the problem under consideration, as it can have a general favorable effect on ensuring transportation security.

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NEW TECHNICAL POSSIBILITIES FOR ENHANCING THE QUALITY OF ARCHITECTURE – MATERIAL PROCESSING TECHNOLOGY BY WATER CUTTING

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Keywords: waterjet, architecture, material processing technology.

Abstract: The new waterjet cutting technology is a technology created in the USA. The beginnings date back to the 50s of the 20th century. Cutting involves the use of a stream of water or water with a mixture of abrasive so called. "Garnet". The water-cutting agent accumulates up to 4000 bar with the pump so that it can be processed to a thickness of 30 cm. The advantage of the method is that there is no unevenness at the edges of the workpiece, which can be seen with the naked eye. The material treated with water is not subjected to high temperature, mechanical or chemical interaction in the cutting zone. Thanks to today's widely used software and CNC capabilities, this technology allows the shaping of any geometric form. Water cutting is currently used for the processing of metals, stone, ceramics, glass, wood and plastics. Waterjet (the so-called commonly used processing device), enhances the interior design details by combining architecture stories with contemporary development trends.

1. Introduction

Cutting using water is based on a necessity to compress it to 4'000 bar using a pump that speeds up the abrasive grain and then directs it to the working head, which's movement is controlled by a computer. In the working head, responsible for the so-called, blast cleaning, there is the second most important element of the tool design located outside the pump, the so-called, orifice, which is built into the working head and is made of a precious stone with a diameter of less than 1 mm. At this point a transformation of potential energy into kinetic energy occurs. The treatment results in a velocity of 1000 m / s, which is 3 times the speed of sound (speed of sound $\upsilon = (331,5 + 0,6 \cdot \Theta)$ m / s, where Θ, temperature in ° C.). The stream of water is directed to the working table on which the material is processed in the cutting technology. The high water pressure and its low temperature allow precise cutting of various shapes with edges after machining without unevenness in the macroscopic evaluation. In addition, cutting with water does not require a processing opening. There are two types of cutting with water; clean water and abrasive cutting. Clean water cutting is used for the processing of soft materials: sponges, rubber, wood, paper; abrasive cutting involves the addition of a special garnet that increases the efficiency of the process. Garnet is a powdered abrasive material that is added to the water at the end of the process when the water is at the required speed and enters the mixing chamber where the abrasive blends are mixed and directed.

2. Advantages of waterjet cutting

This technology gives us the ability to cut any shape precisely in any 300mm material. Waterjet replaces cutting and material processing with diamond cutters, laser cutters and milling cutters, which not only have limited machining capabilities, but also have lower cutting efficiency. During laser and plasma processing, harmful gases are emitted and hazardous dusts are generated. Waterjet technology does not emit toxic dust and minimizes waste. Cutting waste in the form of worn abrasive and fine particles of cut material is collected at the bottom of the water tank - after treatment is considered as ecological waste. Water can operate in closed circuit, with the amount of water used is small and is about 3L / min and usually goes to the sewer. Waterjet technology does not heat up the material because its temperature is around 40 ° C, which is important for the processing of components for different mechanisms. Waterjet technology is characterized by a short machining process which, combined with CNC software, enables the execution of unit orders in a short time with use of complex technological operations.

3. Advantages and disadvantages of water cutting technology

Water cutting can only be used for flat surfaces as a result of machine construction. The cost of cutting with water is becoming more affordable, for the selected tasks such as cutting of non-profiled building stone, traditional methods of processing are still profitable. However, in the case of custom-made or short-form production in the plane, combining different materials such as marble, granite, glass with other non-standard materials, productivity and finesse are losing importance and quality and versatility become the most important. Through cutting technology, we can cut all materials of thickness from 1mm to 300mm, of course, each variant of hardness and thickness of the material requires the appropriate parameters of the fluid velocity. The paper [2] has described it as possible to determine the efficiency of the Tt cutting process, which is determined by the following relation:

$$T_t = f(V_G), \quad (1)$$

where, V_G - head speed [cm / min]; this speed depends on the parameters specified accordingly:

$$V_G = f(\frac{V_S}{K_S} \cdot H_m \cdot HB_m \cdot T_S) \quad (2)$$

where, VS - the speed of the outgoing water from the head nozzle [m / s];
K_S - garnet concentration stream [%];
H_m - workpiece thickness [mm];
HB_m - hardness of the workpiece
T_S - garnet grain size
These parameters ensure optimum cutting conditions. Moreover, it is important to adjust the inclination of the head to the progressive speed of the cutting process. Selection of the angle αV1 which affects the quality and degree of smoothness of the workpiece edge. This issue is the subject of research and results from the experience of working on a particular type of cutting material design. The basis of the machine model is determined by the computer software that ensures the quality and efficiency of the process. Cutting through water with a simple and intelligent cutting preparation software absorbs very little time needed for the machine operator. The important feature that has already been mentioned is that water cutting does not cause the material to heat up and thus to change its local qualities. Especially on the machined edges, which is its primary advantage in terms of post-machined properties. In the case of laser treatment, the edges of the workpiece are cured, preventing subsequent tapping of the hole. The biggest advantage of waterjet is the price, which is several times lower than laser machine's. The advantage of waterjet technology is the high precision of cutting itself, while processing any resulting stress is minimal so that even when cutting small parts, this problem does not occur. This technology advantage is that one should not prepare a processing hole earlier. Technology-adapted software allows one to design the entire process in a few minutes, with the arrangement of cut out shapes, whereas the losses in the workpiece are minimal. Cutting surface quality after machining and machine software allows one to cut any shape in any material without the need for additional machining. Cutting costs are steadily decreasing and the cost of purchasing the machine starts to pay back after a short use. Compatible software allows for quick project evaluation and cutting design to create as little waste as possible in the processing of materials.

4. Waterjet in service for architecture

The efficiency and precision of cutting with water has made it possible to create the most elaborate designs on materials in manner inaccessible so far. This trend has been reflected in the design of fine finishes. Creating ornamental floors has been known for millennia, most often by designing mosaics of gold, precious stones and marble. These materials require precise treatment combined with proper care. The waterjet technology allows them to be re-used for beautiful designs, such as mosaics or floor trusses. The tradition of rosettes dates back to the Middle Ages - they were created as components of gothic architecture and ornamental windows placed over the main entrance to the church. By combining contemporary art with elements of ancient art one can achieve an unusual effect, eg in the form of a floor rosetta as shown in Fig. 1.
These projects can be made of various materials in harmony with the projection of the room under the watchful eye of designer - architect. Precise cutting in waterjet technology allows one to cut any shape in stone or ceramic tiles. This makes the water cut used in the stone industry especially when making a mosaic using a waterjet machine, so that the technology has reacted to the art and evolved it. So far, the composition of such systems was limited due to the use of manual machining methods, which were manifested in geometrically simple shapes - the water cut eliminates these obstacles. The only restriction becomes the human imagination. The world of design and advertising also sees the possibilities of waterjet and is increasingly willing to follow it.

Fig. 1. Decorative effects obtained using water jet cutting [4]

5. Parts of water cutting devices

Water jet cutting technology was already used since the 70s of the twentieth century. It depended on raising its pressure to very high values so that it could be pushed through a very small opening in so called, water nozzle (orfice or jewel). The high-pressure water jet from such a nozzle is able to cut soft materials, and with the addition of garnet, also steel and building stone. The general view of the waterjet design of the 2000mm x 3000mm table made by Kimla in Częstochowa is presented in Fig. 2 [3].

Fig. 2. Waterjet structure [3]

The principle of the water stream generated in the nozzle, as illustrated in Fig. 3;

In order to obtain an efficient water jet, its pressure should be raised to 1'300 - 1'400 bar. The water pressure will force the flow through the water nozzle with its diamond-tipped hole of 0,18-0,4mm (0.007 "-0,015") to create a very high flow of water and energy. The next stage of development was the abrasive waterjet. The principle of the formation of such a stream is based on use of water to accelerate the abrasive grains of most common garmont. Abrasive blend, after mixing with water in the mixing chamber, so-called, mixing tube, creates an abrasive water stream capable of cutting the most durable materials in AWJ technology.

Typical parameters for AWJ technology:

Pressure 350 - 600 MPa

Water consumption 3 - 6 l / min

The consumption of abrasive material 300 - 500 g / min

Water nozzle diameter 0.3 - 0.4 mm

Shaping nozzle diameter 0.8 - 1 mm

a) b)

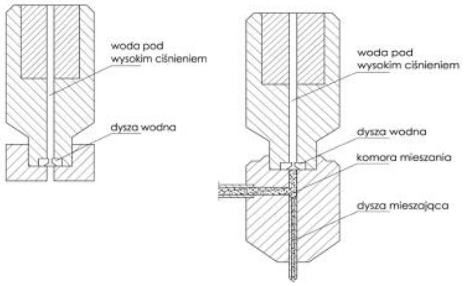

Fig. 3. Waterjet nozzle structure; a) water nozzle, b) water-abrasive nozzle

A variable head type is used for the particular type of treatment as illustrated in the following table:

Tab. 1 Recommended type of head for the type of workpiece

Water head	Water-abrasive head
Rubber	Steel
Foil	Aluminum
Soft wood	MDF
Food	Glass, Ceramics
Other soft materials	Granite, marble, Plastics

Many machines allow for rapid adaptation to different types of heads or can be installed simultaneously. The cost of the water-cutting head is between $ 500 and $ 1,000, while the water-abrasive one is $ 800- $ 2,000 to $ 800. Water-abrasive heads require additional equipment in the form of abrasive material feeding systems. These systems cost 1'000 - 5'000 $ depending on the design.

6. Conclusions

- The most common problem in cutting technology is the water nozzle - the orifice. Nozzles - orifices are usually made of sapphire, rarely diamond. These are very fragile materials and therefore very sensitive to any contaminants present in high-pressure water flowing. The flow velocities are very high, so the impurities in the water can cause the nozzle to crack. It is important to take care of the water quality, control the condition of the filters and the installation of the water treatment. If a nozzle breaks, the replacement operation takes 2 to 20 minutes;
- In case of the water-abrasive head tools, the short life of the mixing nozzle may be a problem. Shortening the operating time of the trigger is often caused by the lack of concentricity between the water nozzle opening and the mixing nozzle opening. During operation, the nozzle must work up to 140 hours;
- Extremely fast implementation - from the drawing board directly to the cutting table;
- Short setting time - low tangential forces in most cases cause complex clamping of the device unnecessary;
- High degree of precision - expensive cutting becomes unnecessary;
- High cutting speed;
- No need to sharpen the tool;
- Greater safety for the user and the environment - no fumes or dust from the material are produced; No coolant is needed;
- Cold cutting - does not change the structure of the material;
- Clean final product - eliminate the need for post-processing;
- Edges without burrs - for most applications, no further edge machining is necessary;
- Small cutting hole width;
- Ideal for rapid prototyping, flexible production
- Proven in large series;
- Optimal use of material with CAD / CAM software;
- System solutions tailored to the needs of the customer.

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INITIATIVES TOWARDS IMPROVEMENT IN ENERGY PERFORMANCE IN THE POLISH CONSTRUCTION SECTOR

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Abstract
The paper presents the idea of sustainable building and priorities for the development and areas of actions for implementation of sustainable building. Furthermore, the energy condition of the construction sector in Poland was discussed with the emphasis on various types of buildings, their character, purpose and time of use. The increasing requirements of the European Union and the obligations adopted by Poland set new objectives for activities aimed at the improvement of energy efficiency of buildings. The most important legal acts which promote climate policy in Europe and world were discussed.

Introduction
One of the basic challenges of the contemporary world is to limit energy consumption. This results from the economic and ecological reasons. The higher energy consumption, the higher emission of pollutants to the atmosphere. Reduced energy consumption also improves energy safety of the world. This thesis can be supported by the repeated conflicts in different world regions which may lead to problems with gas and petroleum supplies to the EU countries, with particular focus on Poland, and the subsequent rises of prices of energy carriers.

Therefore, the energy demand should be reduced through rationalization of the consumption. An important element of this programme should be limitation of energy consumption in buildings. According to the forecast presented in the Green Paper „Towards a European Strategy for the security of energy supply" [7], while maintaining the energy efficiency at the current level, i.e. without implementation of the savings program, ca. 70% of energy and energy resources would have to be imported from outside the EU.

Minimization of energy consumption is also one of the elements of implementation of the concept of sustainable development in the construction sector, which consists in the economical use of energy resources and environmental protection in order to maintain these resources for the generations to come.

Sustainable building
The basis for the concept of sustainable development is several documents, of which the most important are the Brundtland Report, Agenda 21 materials, documents of the earth summits and the Kyoto protocol. The first definitions concerning the sustainable development were used during the works of the UN Commission on Environment and Development established in 1987. Based on the conclusions from the commission, the president of the commission Gro Harlem Brundtland, presented the report Our Common Future, also known as the Brundtland Report. The report emphasized leaving a part of natural resources to the generations to come. The report led to the Rio de Janeiro Earth Summit held in 1992. The Agenda 21 developed during the summit contained a series of significant elements, among which the provisions concerned the role of science, industry, technology and business in initiatives towards sustainable development. A very negative effect on the environment is from energy waste excessive energy consumption in the economy, which was emphasized by the Kyoto Protocol (1997). Other world summits in Johannesburg (2002) and Copenhagen (2009) became the foundations for the actions in the European Union in various areas, including the construction sector. Sustainable building and common use of renewable energy in the European Union and, consequently, in Poland, were considered the two of six lead markets, susceptible to innovations and with huge developmental potential. Implementation of new technologies and solutions in the construction sector was found as very important since they can substantially affect the three domains defined as priorities for sustainable development [1,6]:

- environment: construction and use of buildings cause 35% of final energy consumption in the EU, emissions of 42% of greenhouse gases to the atmosphere and substantial consumption of such raw materials as water or aggregate (sand, gravel etc.). Environmental threats are also caused by the waste after demolishing of building structure,
- society: average citizen of a developed country spends around 80% of their life indoors. This leads to a natural need for creation of adequate environment and the comfort of using the houses, flats and offices, which have a substantial effect on the quality of human life,
- economy: the construction sector in the European Union generates around 10% of GDP and employs 7% of the labour force [2,3].

Mutual relations and synergy of the triads of domains being at the core of the contemporary understanding of sustainable development i.e. the environment, society and economy are illustrated in Figure 1.

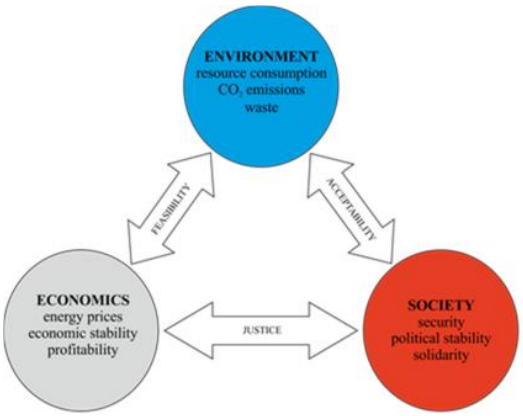

Fig. 1. Illustration of mutual relations of sustainable development

These interpretations indicate that the development of construction, which consists in implementation of innovative technologies and modern solutions that combine beneficial economic effects with care for health and comfort of the users while reducing the negative effect of buildings on natural environment and climate, is a necessary component to drive the economy, both European and global, into sustainable development.

For this reason it is extremely important from the standpoint of environmental protection to develop and implement new and effective technologies and products with good utility parameters that limit the demand for renewable energy and primary resources. They should increase the contribution of energy from renewable sources to total consumption and allow for utilization of recycled materials, which, apart from environmental benefits, leads to measurable financial savings. Sustainable development is also important for quality of users' life. Innovative solutions in building determine the comfort of use of flats and offices. They can, through formation of internal environment, positively affect health status of people staying in these spaces. Another problem is opportunities for adjustment of buildings to the needs of older adults or disabled people using modern, environmentally-friendly technologies and products.

Renewable energy in civil engineering

The costs of construction are significantly determined by energy consumption and costs of its generation. Therefore, one of the major tasks at individual stages of construction of buildings is reduction in consumption and searching and utilization of the cheapest and most environmentally-friendly sources, that is, mainly renewable energy sources [2,3].

The energy consumed in the building is divided into:

• embodied energy i.e. energy accumulated in the building during its construction in the form of energy consumed for production of materials, transport, embodying processes and energy needed for repairs and maintenance,

• operational energy, i.e. energy consumed during the use for heating, ventilation, air conditioning, lighting and energy consumed to prepare meals,

• processed energy, that is, energy needed for the demolition of buildings and waste management.

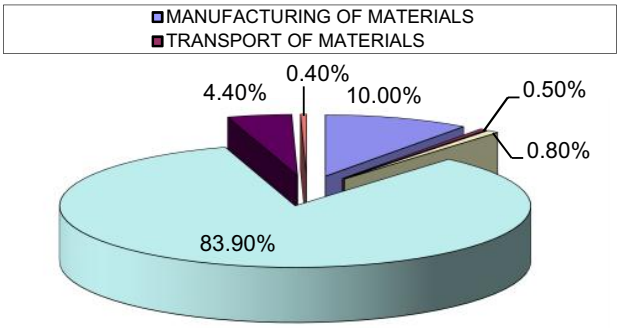

Fig. 2. Structure of energy consumption in the technological life cycle of the building [1]

Figure 2, which illustrates the structure of the energy consumption balance in the building life cycle, reveals that the most important type of energy is the energy connected with using the building, representing 83.9% of total energy. The embodied energy, which concerns manufacturing of materials, transport, construction and repairs accounts for 15.7%. With the level of 0.4%, the energy balance is closed by the energy connected with using the building at the end of building life [.1].

As mentioned before, the construction sector in the European Union is responsible for the substantial contribution to the total energy consumption. Therefore, the problems of energy efficiency of buildings should be approached as priorities. The effects obtained in this sector in terms of energy efficiency can be the most pronounced and have an effect on the whole energy balance.

The energy efficiency of buildings can be evaluated through the analysis of the indices:

EP- index which determines annual demand for renewable primary energy per unit of surface area of rooms with controlled air temperature expressed in kWh/(m^2year);

EK- index which determines annual demand for final primary energy per unit of surface area of rooms with controlled air temperature expressed in kWh/(m^2year);

The essential effect on the level of these indices is from the year of construction and the presence of relevant law or the lack of the law. Depending on the year of building, current construction resources in Poland are very varied in terms of the demand for renewable primary energy **EP** and final energy **EK**, as presented in Table 1. Due to the high number of buildings with high energy demand (over 80% of buildings in Poland are 20 years old or older), their modernization and renovation will have a significant effect on limitation of energy consumption from the renewable sources in the whole building sector. Development of a low level of primary energy demand (EP) in current buildings is usually more expensive but necessary. With the increasing requirements that have limited energy consumption in the last years, the declining tendency is being observed for the energy demand for residential resources in Poland, which is reflected by the content of the Table 1 [4].

Table 1. Age structure in residential resources in Poland and energy consumption index

No.	Period of resuming the building years	Buildings thousand	%	Flats million	%	EP kWh/(m^{2}year)	EK kWh/(m^{2}year)
1	before 1918	404.7	7.3%	1.18	9.1%	>350	>300
2	1918-1944	803.9	14.5%	1.45	11.2%	300-350	260-300
3	1945-0970	1363.9	24.6%	3.11	24.0%	250-300	220-260
4	1971-1978	659.8	11.9%	2.07	16.0%	210-250	190-220
5	1979-1988	754	13.6%	2.15	16.6%	160-210	140-190
6	1989-2002	670.9	12.1%	1.52	11.7%	140-180	125-160
7	2003-2007	321.6	5.8%	0.6	4.6%	100-150	90-120
8	2008-2011	205.1	3.7%	0.41	3.2%	---	---
9	under construction	27.7	0.5%	0.04	0.3%	---	---
10	not defined	332.7	6.0%	0.43	3.3%	---	---
	Total	5544.3	100.0%	12.96	100.0%	---	---

Energy performance of the building depends on many factors. It is also closely related to the structure of the building and the technical solutions used, location and environment around the building and its designation and method of using. Figure 3 presents mean values of EP and EK indices for the buildings with various designation, which were evaluated based on the energy performance certificates [5]. The figure shows that the level of energy consumption is significantly affected by the designation and character of the building. Therefore, public utility buildings consume several times energy more compared to residential buildings.

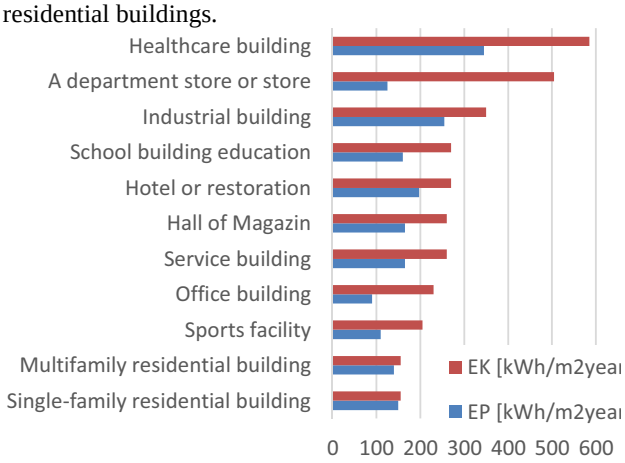

Figure 3. Mean values of EP and EK indices for buildings with various designation [5].

The activities aimed at limitation of energy consumption should be oriented at the following aspects:

- method to supply energy to the building,
- systems of heating, cooling, hot water preparation, in-built lighting installation, thermal insulation of the walls,
- method of using,
- building geometry, its situation and functional solutions.

Figure 4 presents the structure of energy consumption in residential buildings in Poland [5]. This figure shows that the dominant importance in the energy balance of residential buildings is played by ensuring adequate microclimate through heating and ventilation. Therefore, the activities that improve energy efficiency of buildings should be oriented especially towards energy saving. This direction is observed by all the institutions that act for the limitation of the energy needs in the construction sector. The International Energy Agency developed the recommendations concerning the structure of energy consumption in the residential construction, which is presented in Fig. 5. Comparison of Figs. 4 and 5 confirms the previous notion of the need for a substantial reduction in the contribution of energy used for heating and ventilation, from 72 to 38. Therefore, further initiatives should be oriented at improvement of the energy efficiency of buildings through reduction of the demand for energy for heating purposes.

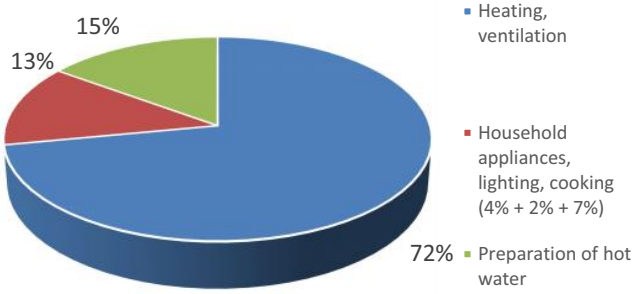

Figure 4. Structure of energy consumption in residential buildings in Poland [5]

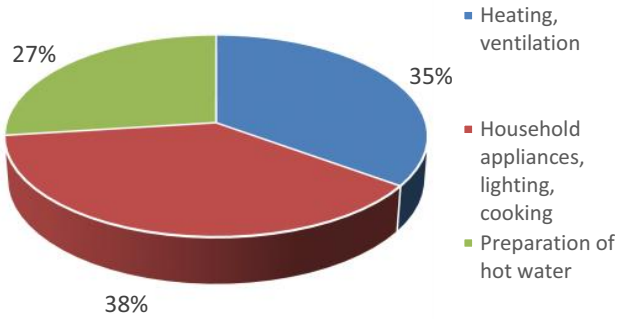

Figure 5. Structure of energy consumption in residential buildings recommended by the International Energy Agency [5]

Adequate development of climate and energy policy that ensures e.g. reduction of greenhouse gas emissions and supports the increase in the use of renewable energy and improved energy efficiency through more energy-efficient buildings is one of the major challenges that result from the membership in the European Union and the recipe for energy independence of the country and reduction in carbon dioxide emissions.

The initiatives towards this policy should also improve innovativeness, help implement new technologies in the construction and installation sectors, reduce energy intensity and generate new workplaces, consequently leading to the increase in competitiveness of the economy and citizen's well-being.

Legal initiatives of the EU

The major instrument of the EU's law used currently in the area of the effective use of energy in buildings is 2010/31/UE directive replacing the previous 2002/91/EC on energy performance of buildings.

These regulations are of framework character, which means that no uniform required levels were specified for all the EU countries but they were only obliged to determine the concrete requirements and implement adequate mechanisms with respect to energy performance of buildings which should be used in their territory.

The provisions of the above directives concern the problems of limitation of the demand for energy by newly build and used buildings through implementation of various regulatory instruments. These include in particular:

- the requirement of determination of optimal (in terms of costs) requirements concerning the energy performance for newly constructed and used buildings,
- the requirement of the analysis of the available technical, environmental and economic opportunities for implementation of high-efficiency alternative systems,
- the requirement of determination by the member states of the requirements that result in nearly zero energy consumption in newly constructed buildings.

Furthermore, the two directives related to the problems of energy consumption of buildings should be indicated: the 2012/27/EU directive and 2009/28/WE directive on promotion of the use of energy from renewable sources [10].

In the opinion of the European Union, key components of new frameworks concerning the climate and energy by 2030 should concern the goals in terms of reduction of greenhouse gas emissions at the level of the EU, distributed equally among the member states in the form of binding national targets. The key elements of the new frameworks is the reform of the system of trading of emissions; the EU's target level of the contribution of renewable energy and new European process of management of the policy in terms of energy and climate.

Legal initiatives of Poland

Before 9 March 2015, the problems connected with the requirements concerning energy efficiency of buildings and the improvement in the energy standards of buildings according to the resolutions of the directive 2002/91/WE were regulated by the regulations of the Building Law Act [8] and secondary legislation to this act. In 9 March 2015, the system of evaluation and improvement in energy efficiency of buildings was modified and since then it has functioned based on the regulations on energy performance of buildings [4].

The aim of the act on energy performance of buildings is to promote improvement in energy performance of buildings and to improve current systems of evaluation of energy performance of buildings while taking into consideration the previous experience.

The act contains the following regulations:

- system for energy evaluation of buildings,
- requirements for people authorized for preparation of energy performance certificates,
- requirements for people authorized to perform inspections of the heating systems or air conditioning systems,
- obligation of the inspections of the heating systems and air conditioning systems,
- verification of the energy performance certificates and protocols from inspections of the heating systems and air conditioning systems by the minister responsible for construction, land planning and management and residential building,
- authorization for the minister responsible for building, planning and land management and residential building for development of the national plan aimed at increasing the number of buildings with low energy consumption,
- obligation of preparation of the energy performance certifications for the buildings where the institutions of justice, prosecutor's office and public administration institutions take surface area of more than 250 m^2 and where the services are provided for clients and placing them in a conspicuous position,
- obligation of providing information in terms of energy efficiency of buildings and their parts in the advertisements concerning the rent or sales if the certificate has been already prepared for the building.

Furthermore, with the act coming into effect, the central register of energy performance of buildings was created with the following lists:

1) People authorized for preparation of energy performance certificates;
2) People authorized for performing inspections of the heating systems or air conditioning systems;
3) Energy performance certificates;
4) Protocols from inspections of the heating systems or air conditioning systems;
5) Buildings where the institutions of justice, prosecutor's office and public administration institutions take surface area of more than 250 m^2 and where the services are provided for clients.

The lists from points 1, 2 and 5 can be accessed at the website (https://rejestrchebmib.gov.pl/), showing the current lists of experts who prepare energy performance certificates and perform the inspections of heating systems and air conditioning systems.

Consequently, the modernized system for evaluation and improvement in energy efficiency of buildings involved

modification of current and implemented new regulations in order to ensure the economically feasible improvement of energy performance of buildings. This improvement results from the reduced demand for energy used for the purposes of heating, cooling, preparation of hot water and lighting and ensuring adequate energy standard of constructed buildings and the buildings under redevelopment.

According to the amended regulations, the buildings and its heating, ventilation, air conditioning and hot water supply installations and also for embedded lighting in the case of public utility buildings, collective residential buildings, production buildings and warehouse buildings, should be designed and constructed in a manner ensuring meeting the minimal requirements.

The minimal requirements for the building are understood to mean:

• ensuring the value of the EP index [kWh/(m^2·year)], which determines annual demand for non-renewable primary energy for heating, ventilation, cooling and preparation of hot water, and also for embedded lighting in the case of public utility buildings, collective residential buildings, production buildings and warehouse buildings, calculated using the regulations concerning the methodology of calculation of energy performance of building, lower than the limit defined in the ordinance on technical conditions that should be met by buildings and their situation;

• meeting the requirements concerning thermal insulation of walls (e.g. external walls, internal walls, windows, doors, roofs) and technical equipment in the building.

Level of requirements in terms of energy savings and thermal insulation will be gradually increased. These requirements for various types of buildings are presented in Fig. 6. The stage-based changes will allow for e.g. smooth meeting the current legal requirements by all participants in the construction market (investors, designers, engineers, construction product manufacturers).

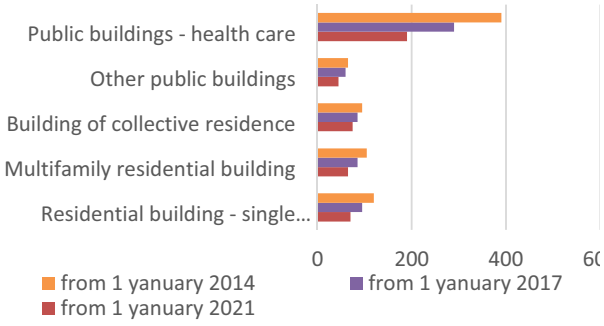

Figure 6. Changes concerning the value of EP index in the nearest years

One of the elements of the state's strategy is promotion and creation of respective regulations and policies oriented towards the improvement in the energy efficiency of buildings and environmental protection while being consistent with further integration of the European market, increased competition and achievement of the goals concerning the climate and energy at the EU's level.

With respect to the above objectives, the national plan was developed to increase the number of buildings with low energy consumption [13] which, on 22 June 2015, was adopted by the resolution of the Council of Ministers of the Republic of Poland.

The national plan contains in particular: the definition of buildings with low energy consumption and their details, activities of governmental administration taken in order to promote the buildings with low energy consumption, including design, construction and redevelopment of buildings in a manner that ensures energy efficiency and the increase in the use of energy from renewable sources in new and current buildings. The schedule for reaching these goals was also taken into consideration.

Conclusion

The substantial improvement in energy efficiency of buildings in Poland is a very ambitious initiative, which cannot be implemented without the support of the central administration and local governments. Furthermore, it represents one of the few examples of activities, which nearly immediately yield positive and measurable economic, environmental and social benefits. The scale of these benefits depends not only on the magnitude and rate of public spending, but also, or first and foremost, on well-considered strategy and consequence in implementation of this strategy. The discussed objectives for modernization of the building contain several solutions that meet the expectations of the groups acting for promotion of sustainable development. However, they should be monitored while legal solutions should be improved to help overcome political and economic problems with thermomodernization by removing the barriers that limit activities in this area.

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11. The Building Law Act of 07 July 1994 (Journal of Laws 2013, Pos. 1409 with further amendments)
12. Act of 29 August 2014 on energy performance (Journal of Laws 1200)
13. Resolution of the Council of Ministers as of 22 June 2015: National plan to increase the number of buildings with low energy consumption

INNOVATIVE METHODS OF STRENGTHENING THE TIMBER CONSTRUCTIONS

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Keywords:timber constructions, reinforcement, FRP (Fibre Reinforced Polymer), fiber composites.

Abstract

The paper presents an overview of the innovative methods of strengthening the timber construction with the use of various materials. The article is focused on the use of the fiber composites because of their increasing popularity and favorable properties. Moreover, other less popular methods in a phase of research, are also presented.

1. Introduction

The best known method of reinforcing the constructions is the use of steel bars while reinforcing concrete, by means of which it is possible to receive the material combining the properties of concrete and steel. In case of timber constructions, there are also used various methods of reinforcement in order to obtain the element maintaining the advantages of wood, but with better strength parameters.

Strengthening of the hardware elements is often necessary in order to fix, or improve the static work of the already existing constructions. It is common that such buildings are monuments, in which apart from the requirement to improve the bearing capacity and stiffness, there is also need to meet the requirement concerning aesthetical aspect. The problem was widely discussed by Jasieńko [1] and Nowak [2].

The research shows, that there are various methods of reinforcing wooden elements, analyzing them in a way that combining two, or even greater amount of materials could allow to receive an element with the best possible properties. Development of the methods of reinforcing wooden constructions is dated for the 1970s, thanks to the progress in the area of epoxy adhesives, used for fixing the reinforcing elements. Steel is a very popular material used for strengthening the wooden constructions (in a form of bars, or plates), used both in the processes of passive and active reinforcing.

Nowadays it may be noticed that the use of fiber composites reaches greater popularity in case of strengthening wooden constructions, as well as concrete, brick walls and steel. [3]. There are also less popular methods of reinforcing discussed, such as compressed wood [4], or metal screws [5].

The article presents an overview of various methods of strengthening of timber constructions together with the analysis of the research results, also taking into account the latest trends in the building and construction industry. The full overview of methods of strengthening the timber constructions (innovative and traditional) is presented in [6].

2. Reinforcement with the use of the composite materials

Because of the favorable properties of the composites (low weight, high tensile strength, resistance to chemical and biological corrosion) a great interest in this material may be noticed in the building and construction industry. The most popular and the most widely used are the fiber composites FRP (Fiber Reinforced Polymer), in which the epoxy resin, also called continuous phase, or a matrix, is reinforced by the fibers, i.e. fiberglass, carbon fiber, or aramid cords. Strength and durability of the composites reinforced by the fibers to great extent are dependent on the way of manufacturing and compaction of the fibers. There is analyzed the use of the natural fibers (such as cotton [7]) in a reinforcing process, because of the more natural character of the fibers of this type.

One of the first large-scale appliances of wooden beams reinforced with the fiber composites is the method invented and patented in the USA by Dan Tingley called FIRP. This method is based on pasting the composite tape between the last and the last but one wood slats. Such reinforcement allows to save from 25-40% of the wood usage. [9].

In Poland analyses of such type of the beams were conducted by the researchers from Silesian University of Technology in cooperation with the Austrian company Buchacher [10]. The subject for analyses included 15 glulam beams, 6200 mm long, with spacing of the pillars 5760 mm reinforced with the glass-aramid reinforced polymers (GARP) placed between the last and the last but one slats and at the bottom of the beam (fig. 1).

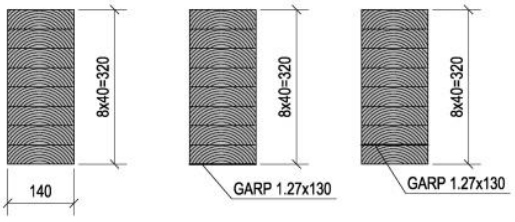

Fig. 1. Cross-sections of the beams used for the research by Brola [10]

On the basis of the research it was stated that the bearing capacity increased respectively, for 54% in case of reinforcement attached at the bottom of the beam, and 68% for reinforcement attached between the slots. An increase in stiffness was equal 15%.

The method described above became the basis for further analyses concerning the subject of adhesive bonds between the FRP and wood [11] and analyses of the

costs of the constructions of this type, which are so far quite high [12].

The new solution with the use of fiber composites proposed by the author [13, 14] suggests the use of the aramid cords between the wood slots in order to strengthen the glulam beams (fig. 2a). The above mentioned method of reinforcing wooden constructions is still in a phase of experimental and numerical studies. After the initial numerical studies it was concluded that the influence of this type of reinforcement on the character of static work is positive (fig. 2b).

Solid wooden beams are also reinforced with the use of the fiber composites. Nowak [2] is an author of the extensive studies concerning this type of beams combined with the composite material -Carbon Fiber Reinforced Polymer CFRP. There were analyzed the beams with reinforcement put in the cross-section, by means of which the conditions of reinforcing historical elements were simulated. The increase in bearing capacity of the analyzed beams was in a scale from 21 % (series F) to 69 % (series D) [15].

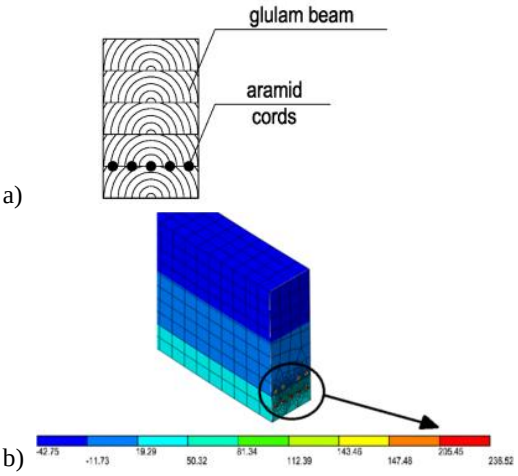

Fig. 2. A New construction of the glued laminated beam (glulam) reinforced by the aramid cords: a) cross-section, b) visualization of the numerical calculations [14]

The way of reinforcing beams with significant damages was suggested by the researchers from the Szczecin University of Technology [16], in which reinforcement with the carbon tape goes through the whole length of the beam, including the most strenuous places (in the middle of the span and over the pillars) as presented in fig. 3.

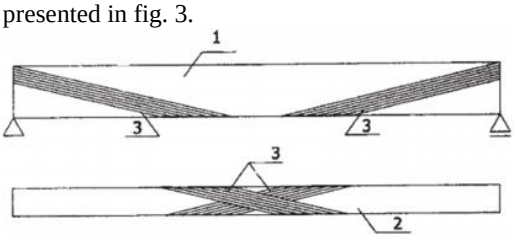

Fig. 3. Beam reinforced by the two carbon tapes [16]

There are also functioning the reinforcing methods with the use of composite bars used in a similar way like steel bars [17], and mixed methods, using both bars as well as composite tapes (fig. 4) for beams with significant burden. [18].

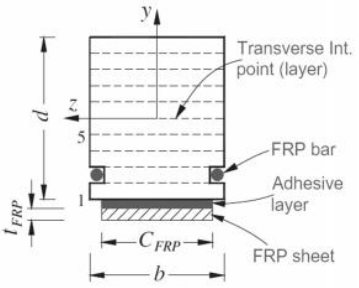

Fig. 4. Glued laminated timber reinforced by the bars and composite tape [18]

An element worth considering in the following paper is [19] an epoxy-basalt bar [20] which may be an alternative for steel bars.

A favorable phenomenon in case of the elements reinforced with composites is an increase in their fire resistance [21, 22], but the subject is not fully understood and there is still some research conducted in this field.

3. Other methods of reinforcing timber constructions

Among many methods of reinforcing the construction, here may be distinguished some of them which are not very popular, nevertheless, there are still many analyses conducted on this subject.

One of the oldest methods of reinforcing timber construction is combining wooden beams with concrete. This subject is still widely analyzed, which may be concluded on the basis of the numerous publications concerning various aspects: work of the attachment zone between wood and ultra durable concrete [23], connectors for combining concrete with a wooden beam (that could replace nails) [24] and attempts of creating hybrid beam composed of glued-laminated beam, concrete and reinforcement in a form of fiber composite [25]. The scheme of the previously described beam is presented in fig. 5.

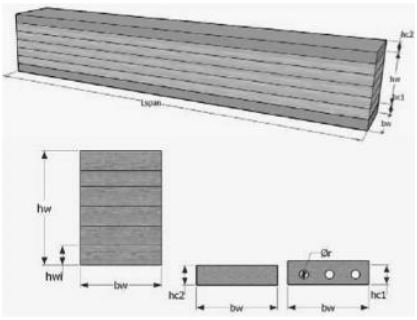

Fig. 5. The scheme of the hybrid wooden- concrete-composite beam [25]

The other method, which is also the subject for many analyses [1, 26] is reinforcing of wooden beams by filling in the damaged places with the epoxy composition. Reinforcement in this case may be based on the pressure injection of the epoxy composite, or filling a removed part of the element with it. Nevertheless, in case of wooden beams reinforced with the epoxy composite is about 15% worse than in case of standard, full beams. Combination of the composite with nails lowers the bearing capacity of the element even more. This method is worth considering while renovating less loaded elements.

Separate group of reinforcements is the use of compression and expansion. It is possible to compress, or extend the elements with the use of: bars, steel ties, composite materials and others, such as compressed wood.

Preliminary studies concerning compression of Wood were started in the 1960s. The subject for analysis was a glulam beam compressed with the use of steel cords placed in the extended zone. Quite good results were achieved in case of reinforcing the planks with low parameters, while in terms of standard beams the increase in bearing capacity was not significant. Research with the use of steel ties was conducted by Cyruliński [27], in which he analyzed compressed, extended and compressed-extended beams.. The best results were obtained in case of the compressed-expanded beams. Because the subject is wide, on account of the wide rheological properties of wood and steel which are significant elements in a process of compression, research in this area is steal conducted. [28].

While reinforcing the wooden beams there is also used the process of compressing the reinforcement in the form of fiber composites. The first attempts of compression were made with the use of Glass composite tapes glued between the last and one before last wooden slats [29], while in order to increase the effect of compression, a beam compressed with several aramid cords was invented, with the aramid tapes glued crosswise in the last slat. (fig. 7) [30].

An innovative way of compressing of wooden beams is the use of blocks of compressed wood (expose to the high pressure) situated in the previously prepared openings. Wood swells and causes the compression of the beam. [4].

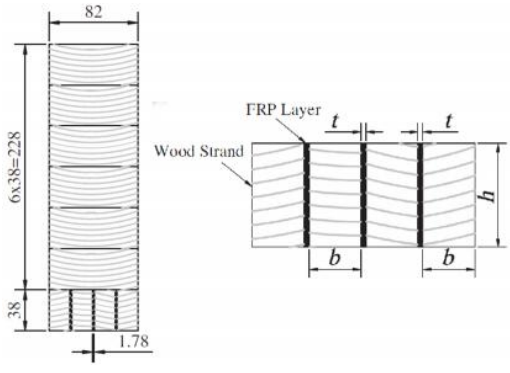

Fig. 7. Glued laminated timber reinforced by the compressed aramid belts [29]

4. Conclusions

There are many various methods of strengthening the timber structures. The most commonly used materials are: wood, steel and fiber composites. Traditional methods are being gradually replaced by the new ones, which are more effective and often less laborious. Nowadays, the most attractive method seems to be the use of fiber composites, which may be concluded on the basis of the significant number of publications concerning this subject. Fiber composites are used as a passive and active reinforcement. By using compression, the significant increase in a bearing capacity may be obtained, nevertheless some further research are required in this area, in order to analyze the influence of the rheological properties of wood and the reinforcing materials.

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NEW CONSTRUCTION AND ARCHITECTURAL SOLUTIONS FOR CIRCULAR HOUSES FOR THE NEEDS OF HUMANITARIAN AND EMERGENCY CAPACITY BUILDING PROJECTS

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Keywords: New construction and functional solution of a social building set on a circle for humanitarian buildings

Abstract: A synthesis of the appropriateness of residential housing solutions has been presented. A new material and construction solution for a cylindrical building has been proposed assuming maximization of functional benefits while minimizing material and labor inputs for the construction of a social building for humanitarian purposes. Construction nodes details drawings have been added to the solution description.

1. Introduction

According to the literature [ref. 1-3], constructions of round or dome houses can be found in many examples of the oldest dwellings that resembled bird nests or mammal beds with their oval shapes. Our ancestors observed and imitated nature as a source of inspiration, which was then then adapted for their own purposes. A sphere is a perfect shape, which is recognized as a friendly geometrical shape by many different civilizations, as this is considered to be a divine shape. As we know, the entire universe in macro and micro scales consists of spherical forms. Seemingly, a sphere is capable of absorbing energy coming from outside, just as the centre of our planet has been hot for millions of years.

Round homes with a dome cover are ideal fullerene protections against external factors such as: hurricanes, earthquakes, snow fall, winds, variable thermal loads, including fire. However, protection is ensured with proper building materials. For social buildings in humanitarian capacity construction, the main direction of our search was based on the examples of world's oldest structures.

2. General description of the construction

A circle is a shape that is used to represent intimacy and equality where life is born, for example the round shape in seeds, eggs and nests. Therefore, taking inspiration from nature, we want the new quality of life to be created with the use of this shape. We would like to refer to and harmonize with an aspect of reality, so we use many round, curved shapes, which not only bring, but also protect the energy and warmth that is a symbol of a warm home. With the description of new possibilities for the development of the living space, we want to change the fixed, squared thinking, to which the logic of squared arguments has already been attached.

3. Highly imperfect examples

It is worth presenting an alternative approach for the construction of residential space in the example in fig. 1, which goes back to the roots. This shows the foundations of a house with a dome dating back to 8 thousand years ago, which originates from nature and adapts to it. Designing a round house is as simple as making scrambled eggs; still not everybody can do it correctly. Preferably, designing should start with the central room, i.e. the living room. This room should face the south or south-west and have large windows to provide warm sunlight, which will have a positive influence on the quality of living.

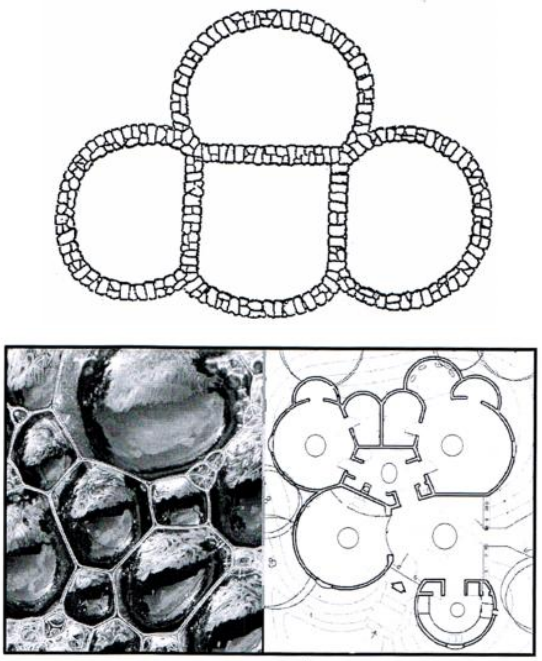


Fig. 1 - a) foundations of a domed house dating back eight thousand years; b) concept close to the nature of the roundhouse

Bedrooms are preferably located on the eastern side so that sunrise can wake us up naturally. The kitchen should be situated on the east side, if possible. Breakfast accompanied by sunrise is a good start to every day. The

pantry and bathroom should be rather positioned on the north side.

4.Arguments for the construction of round houses

According to the references [2], the operating cost of a residential building with a usable area of 100 m^2 after construction and throughout the period of use was calculated to be 90% of the building value. The rise of energy prices is currently the only forecast that is always right. Therefore, we think it is very important to build warm houses. The costs of a round house usually return after 15 years of occupation. In comparison with a house designed on a square plan, relative profits are obtained in the area of the costs of heating. Today, we can insulate houses with various foam materials to save on the energy required to provide adequate thermal comfort and a microclimate inside. A round house costs as much as the thermal insulation of a house built according to advanced residential housing technologies which are currently used in the world. In the new approach, we incur costs equal to the cost of thermal insulation works. According to the information in the reference [3], thanks to the applied insulating material, such houses can reduce the energy needed to heat the rooms by 90 %.

Materials and structures used for the construction of passive communal houses for humanitarian capacity building projects

Building a round house requires the preparation of the ground for casting a foundation of a circular shape, selecting the optimum diameter of the building based on the required comfort of living, physical properties of the building, construction and material conditions, optimum solutions for the connection of utilities and installations with respect to the financial criteria. Make a stay-in-place formwork that acts as thermal insulation and the form of a retaining side wall for a cavity in which reinforcement is placed for casting a concrete base reinforced with basalt fiber rods. The material used to build the external and internal walls includes polystyrene prefabricates with reinforcement on both sides and the reinforcement is secured with steel pins. The main construction elements are manufactured in Poland by AKSBUD. In 2015, the company started a production line of light core prefabricates based on foamed insulation materials which was supported by technology imported from the USA. The products of this prefabricate system are elements of walls, floors, foundation formwork profiles, stair runs used to build frameworks for ribbed plates with a texture layer and a constructional layer made of mineral composites based on cement binders applied with in-situ shotcreting. The benefit of prefabricates is the light weight.

The new technology eliminates the need to use heavy equipment in the erection stage. During construction, a wall prefabricate with 1.2x3 m dimensions can be carried by one man. Construction of the building is quick compared to traditional construction methods. When using prefabricates, not only walls are built but the roofing is also constructed. Construction details related to the idea of the system are presented on fig. 2-11 [4].

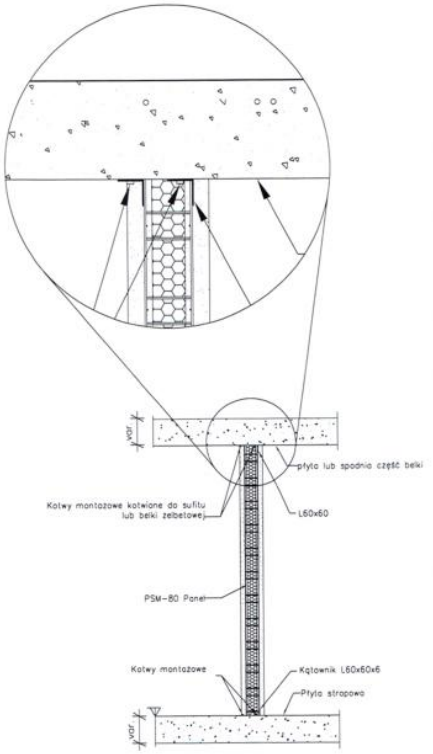


Fig. 2. Detail of wall panels connection

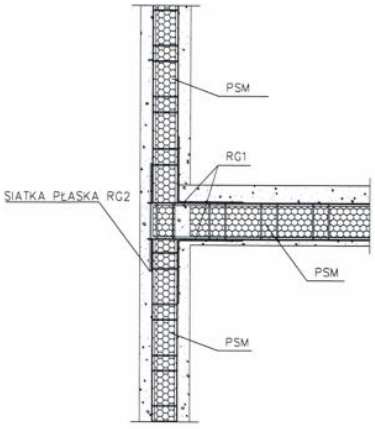


Fig. 3. Detail of wall panel connected with slab

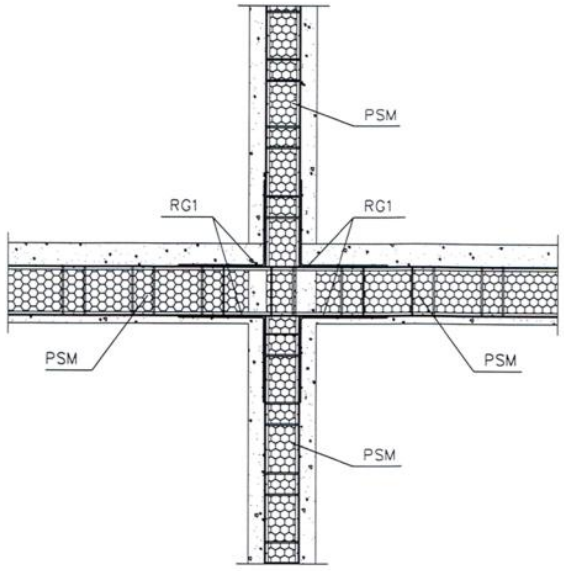


Fig. 4. Detail of wall panel connected with slab

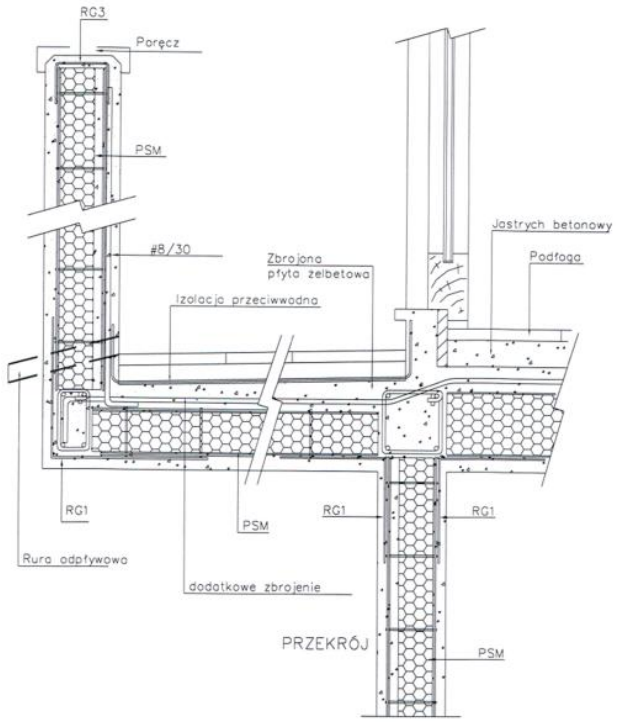


Fig. 5. Detail of balcony construction

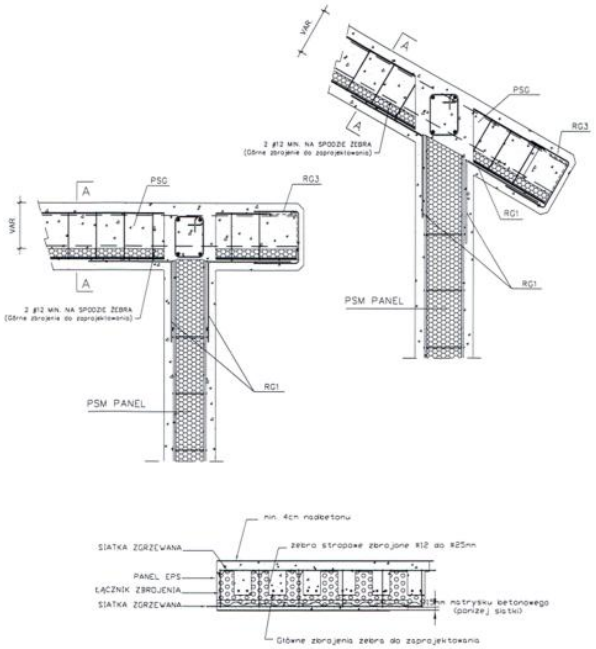


Fig. 6. Detail of wall and roof connection

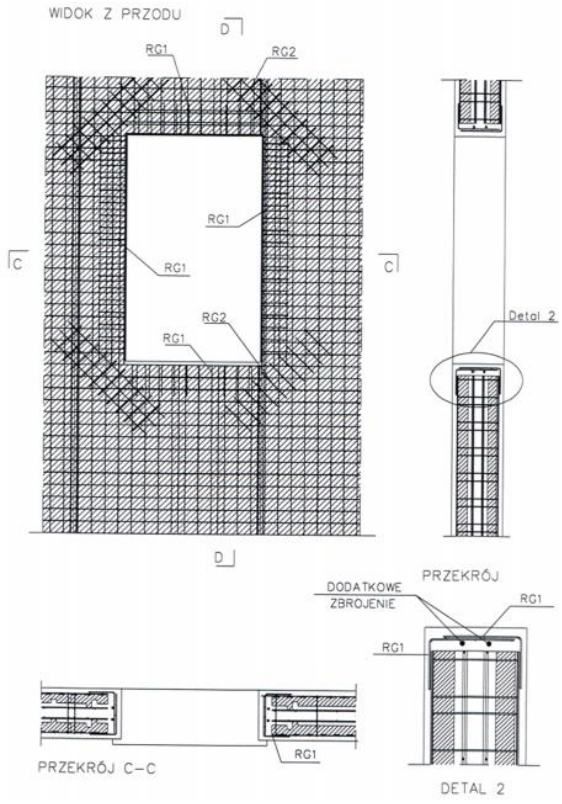


Fig. 7. Detail of reinforcement of panel with hole.

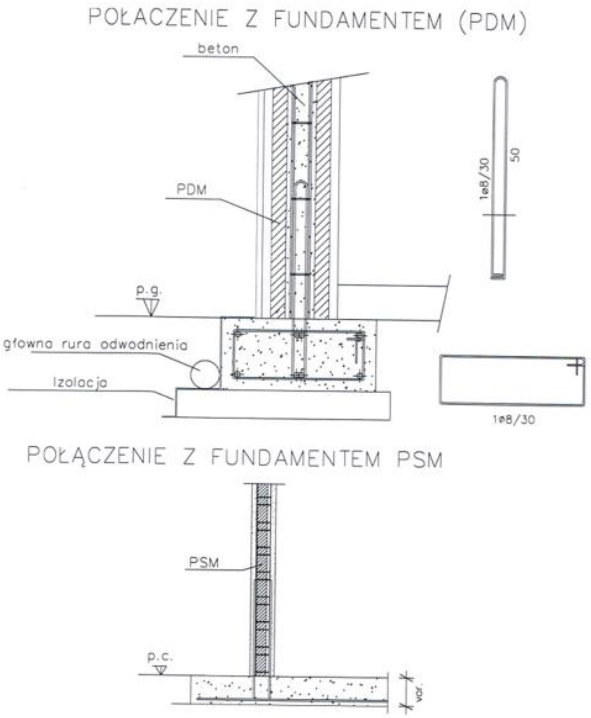


Fig. 8. Detail of wall and foundation connection

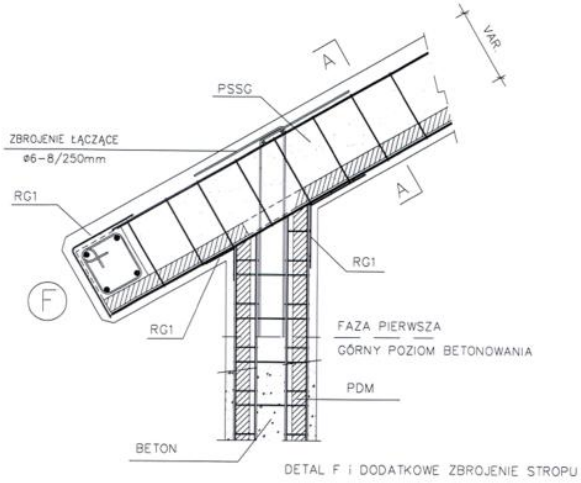


Fig. 9. Detail of wall and roof connection

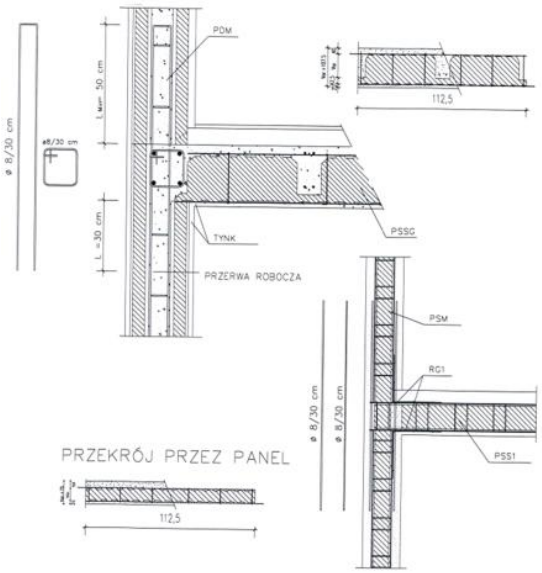


Fig. 10. Detail of reinforcement connection

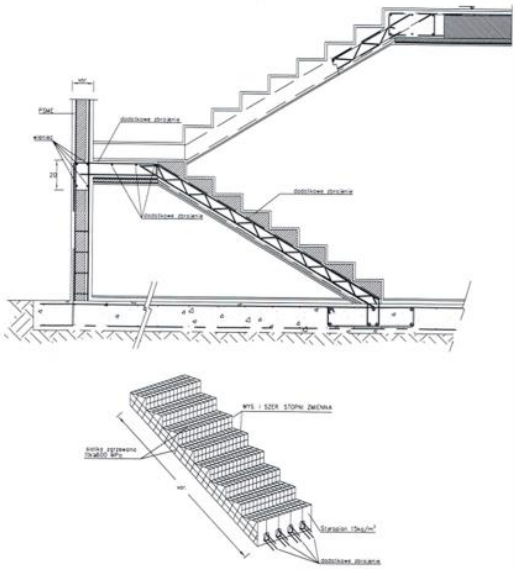
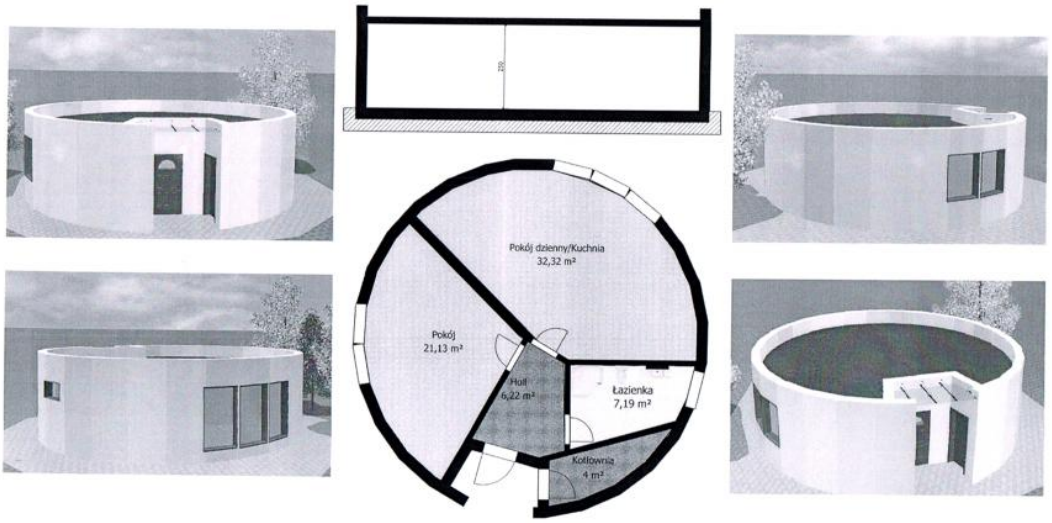


Fig. 11. Detail of stair connection

New solutions of the system and materials inspired us to develop a design for the construction of a communal house in humanitarian capacity building projects with its projection on a circle.

An example of the project is presented in fig. 12.



5. Conclusions

The new material and construction solution with details of the structure nodes in the erection technology and the plan of the function arrangement ensures the minimum energy outlay for the construction and many years of operation with the benefit of obtaining a relatively large usable area of a residential house. The measurable benefits of the new construction and assumptions for houses are represented by the following:

- the best ratio of the usable area to the area of retaining walls for a room height of 2.5 m, i.e. 1:1,
- large usable area as related to the incurred financial outlay for the house construction,
- fast erection works without the need to use expensive machines,
- excellent thermal insulation,
- excellent resistance to moisture,
- good acoustic insulation properties,
- possibility to obtain the parameters of a passive house,
- the most favourable way to provide people with homes in emergency situations, including disasters, while ensuring physical safety and comfort that is so important in such conditions for affected families,
- the proposed material and technology solution for humanitarian capacity residential building ensures the lowest financial cost of a mass project, which is repeatable with regard to material transport and processing technologies,
- usable area of 78 m^2 at 230 EUR / 1 m^2 with a construction time of no longer than 30 days in the climates of many different European countries,
- meeting the requirements for environment-friendly construction with 100% recycling when the operational life of the building ends,
- the above-mentioned features are arguments for the offer which has been developed based on the system by AKSBUD-Poland.

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Fig. 12. The house with 67 m^2 area in circle plan – conception

EXPERIMENTAL VERIFICATION OF HYPOTHETICAL STRENGTH THEORY FOR GRAVEL SOILS

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Keywords: gravel soil, shearing resistance, strength

Abstract

Gravel soils are often used as a material for hydraulic and transport constructions, and also as foundations. Reliable description of their mechanical behaviour is necessary for construction safety. In this issue the results of gravel soils triaxial tests are presented. Strength parameters are obtained using hypothetical strength theory with such aspects as particle roughness and grinding taken into account. The results are compared with other strength theories, used in soil mechanics practice. As a result, the strength theory is verified and can be suggested for practical use.

1 Introduction

Under the soil strength its shear resistance is usually recognized. It is a fundamental soil parameter, and it's appropriate esteem is vital for rational design of numerous constructions, especially soil dams. Shear resistance of soil and its strength is determined by: a) inner friction forces, depending on normal stress; b) reversible cohesion of water-colloid nature, c) rigid structural cohesion [1].

One of the most well-known yield criterion is a Mohr-Coulomb criterion [2]. The Mohr-Coulomb theory considers the sliding plane that intersects the plane of the maximum and minimum principal stresses and is parallel to the axis of the intermediate principal stress. Many authors have noted that Mohr-Coulomb theory is most suitable for solving problems of a flat stress-strain state, slopes stability problems mostly [3, 4].

Experimental studies of different soil in a wide range of stresses, performed by various researchers [5, 6] have revealed, that the intermediate principal stress has an influence on the shear resistance, hence the yield condition needs to account for this effect.

There was an interesting yield conditionsuggested by A.I. Botkin [7, 8], who suggested that the Mohr-Coulomb condition is satisfied on the octahedral plane, equally tilted to the axes of principal stresses. This theory takes into account the intermediate principal stress, however, also has some drawbacks. Malyshev M. V. [7] noted the "phenomenological" character of this theory, as plane selection is dictated by mathematical simplicity.

The results of the studies summarized by Rasskazov L. N. [9, 10] confirm a significant influence of intermediate principal stress on the shearing resistance; the nonlinearity of the envelope of the Mohr circles in the majority of cases; the effect on the strength of the loading trajectory and Nadai-Lode. In addition, to date there is no possibility of accounting for engagement of particles in coarse-grained soils and the resulting increase in strength.Especially important these phenomena are in the calculation of foundations and earth structures composed of coarse-grained soils with sand or clay filler.

At present, a large number of yield and strength conditions where developed for non-cohesive soils [11, 12]. In this study the authors made an attempt to develop the yield criterion, taking into account all the described above regularities and phenomena.

2. Methodology

The account of all three principal stresses is possible when considering the plane, which is determined by the magnitude of the principal stresses at the ultimate state. It is obvious that such a plane will have a different angle relative to each of axes of principal stresses. The position of the plane may be set by the guiding cosines of the normal vector. Using the known formulas of solid mechanics to determine the normal and shearing stresses on any plane we can get the expressions for these stresses.

The most difficult task is to identify the role of engagement in the shear strength of fine-grained soils. From physical sense of engagement, it follows that its magnitude does not depend on the compression, as even in the absence of compression with only shear forces applied there is a resistance due to engagement. The magnitude of this resistance will be constant for an infinitely strong material, as the destruction of the engagement irregularities will not occur. With decreasing strength of the material the irregularities will begin to split. In the course of the shearing theret will be gradual bridging of the bumps. Finally in the case of extremely fragile material, engagement will not be observed, since the shear plane will pass through the material of the particles.

At the same time, the magnitude of compression will change the amount of friction, the magnitude of the normal force to the surface of the prong is greater than the magnitude of the force normal to the cut surface due to the tilt of prongs. Chipping of prongs will occur when the shear stress exceeds the shear strength of the particles. Thus, the contribution of the engagement in shear resistance is easier to evaluate in the form of increased friction. Engagement is implemented before the start of the shearing prongs, and then begins to decline to zero with smoothing irregularities.

In its final form, the proposed yield criterion can be formulated in the form of three equations, allowing to determine the stress components for the sliding plane and the interaction between them (1).

$$\begin{cases} \sigma_n = \dfrac{(\sigma_1\sigma_2\sigma_3)(\sigma_1\sigma_2+\sigma_2\sigma_3+\sigma_1\sigma_3)}{\sigma_1^2\sigma_2^2+\sigma_2^2\sigma_3^2+\sigma_1^2\sigma_3^2} \\ \tau_n = \dfrac{(\sigma_1\sigma_2\sigma_3)}{\sigma_1^2\sigma_2^2+\sigma_2^2\sigma_3^2+\sigma_1^2\sigma_3^2}\times \\ \times\sqrt{3\left(\sigma_1^2\sigma_2^2+\sigma_2^2\sigma_3^2+\sigma_1^2\sigma_3^2\right)-\left(\sigma_1\sigma_2+\sigma_2\sigma_3+\sigma_1\sigma_3\right)^2} \\ \tau_n^* = \sigma_n\times tg\varphi\times\left(1-(1+n)^{\sigma-R}\right)+c \end{cases} \quad (1)$$

where*R* – axial compression resistance of particles material,*n* – parameter, depending on shear resistance of particles material.

The parameter *n* can be expressed, provided that we know the shear resistance of the particles, however, it is more convenient to define it on the basis of experimental data.

On the figure 1 there is a graph of a limit line function with different n values.

It is obvious that such a view explains the definition of engagement by many authors as an additional constant component close to cohesion. This is due to the application of the linear form of the limit line. Drawing such line through any three points shows initial shear strength larger than zero. Usually, it is interpreted as "engagement". In fact, the cause of it is limit line nonlinearity.

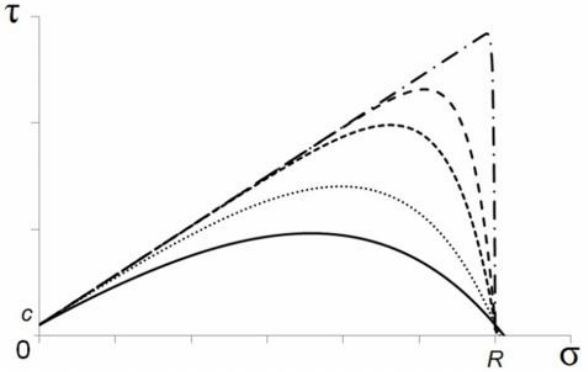

Fig. 1. Limit lines with different values of n

It should be noted that the friction angle in this criterion is not numerically equal to the friction angle for other yield conditions, in particular the Mohr-Coulomb and von Mises-Schleicher-Botkin.The angle of internal friction and cohesion should be determined according to the initial part of a limit line, where even in the area of stress concentration the strength of the particles is not exceeded.

Forexperimentalcheckofthistheoryaseriesoflaboratorytest sofgravelsoilwithhomogeneousgranulometriccomposition (quartzgravelwithparticlesfrom 2 to 5mm). This material was tested in a triaxial machine for large scaled samples of 300mm in diameter, with 500kN of axial force, and maximum cell pressure of 2MPa. Another test series was performed for natural gravel of the Kumtordamb in Kyrgyzstan.

All the tests used consolidated-drained scheme with consolidation pressures of 200, 400 and 800 kPa. Kinematicloadingratiowas 0,3mm/min (0,05%/min). The tests were performedwith different loading trajectories - CTC and TC to check the ability of accounting for intermediate main stress. The test was stopped after stable maximum vertical stress achievement.

3 Results

Allthetestresultswereprocessedaccordingtostandardproce dures, includingcorrectionforsample area change and membrane elasticity. From the results the maximum values of vertical stresses where obtained and the strength parameters were calculated according to Mohr-Coulomb criterion, Mises-Schleicher-Botkin criterion and suggested criterion. Theresultsareshowninthetables 1 and 2.

Table 1 Friction angle values

Soil type	Quartz gravel CTC	Quartz gravel TC	Kumtordamb gravel
Parameter	φ	φ	φ
Mohr-Coulomb	34,6	38,5	28,1
Mises-Schleicher-Botkin	39,0	42,2	33,3
Suggested criterion	29,1	29,8	27,9

Table 2Cohesion values

Soil type	Quartz gravel CTC	Quartz gravel TC	Kumtordamb gravel
Parameter	c	c	c
Mohr-Coulomb	67,9	2,0	156,2
Mises-Schleicher-Botkin	78,0	2,3	171,9
Suggested criterion	6,9	0,2	16,1

Itcanbeseen, thattheresults for different loading trajectories are nearly the same for the suggested criterion, but for Mohr-Coulomb and Mises-Schleicher-Botkin there is a significant difference. For CTC trajectory these criteria show high cohesion, that is unusual for non-cohesive uniform gravel.

AsfarasrealgravelsoilfromtheKumtordambisconcerned the friction angle values seem to be close, but cohesion is rather large for classical criteria. The reason is that it uses all the strength points to build a straight line, when suggested criterion is non-linear.

4 Conclusion

On the results of the research the following conclusions can be made:

1. The most common yield criteria have limited application for coarse-grained soils, as engagement and the decrease in shear resistance with increasing normal stress is not fully taken into account.
2. A yield condition suitable for coarse-grained soils must consider the engagement of the particles and enable the strength of the particles as a parameter.
3. The proposed yield condition can be used to describe the limit line, taking into account its nonlinearity and also the intermediate mean stress influence.
4. The parameters of this yield criterion can be determined on the basis of experimental data.
5. The parameters of this yield criterion can be considered as invariant - independent of loading trajectory and stress condition.

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Tendencies of the Development of Historical Cities Taking Into Account Identification Characteristics of the Established Urban Environment

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Keywords: identification, historical cities, urban environment.

Abstract

Established practice of designing the modern urban environment cannot meet the needs of the population with regard to many factors. Housing development with high-rise many-storied buildings without public spaces meets interests of developers only. The city loses its individual urban matrix, the living environment loses its comfort, and perception of the city identity does not form in inhabitants.

The goal of the article is to reveal tendencies of the development of historical cities taking into account identification characteristics. Study tasks are to define the concept of the urban environment identification and reveal identification characteristics.

In the study, identification is understood as establishment of the identity of one object to another on the basis of coincidence of characteristics. Authors conclude that identification can exist at two levels. The first level is identification of a smaller structural unit (block) with a larger one (with the city). The second level is identification of the townsman with the region of residence. Identification of the second level can be achieved taking into account the principle of a recurrent sign. Any visual images, details in the man's living environment, which practically do not occur outside of it, can be a sign. Following such principles as conservation of traditions of the urban environment development, conservation of the established density and the street development outline, as an urban environment identification feature; modularity of the urban development, use of traditional parameters of residential buildings, retention of architectural techniques associated with typological features of residential buildings, a certain color scheme of the street development, use of characteristic architectural details and signs will make it possible to preserve the familiar urban environment.

A comprehensive individual approach to the creation of the architectural environment, which is comfortable for living, is required. 1 **Introduction**

The growth in residential construction volumes in Saint Petersburg is unfortunately accompanied with the loss of quality of the environment of new residential districts, which do not fit in with architecture, culture and style of the historical part of the city. The priority of pragmatic functional and hygienic requirements turns into oblivion of aesthetic and other traditional urban values, monotony and dead-level of a new residential development. A micro-district model with free arrangement of apartment buildings in the extensive enough territory surrounded by trunk roads deprived its residents of the feeling of belonging to their own house, the "homing". They can hardly explain where they live, since there are neither sightseeing attractions, nor buildings with memorable architectural features, nor special signs in their living place.

Dense development with multi-storied high-rise buildings without creation of public spaces, neglecting of the characteristic scale of house construction, which is typical for cities, do not meet interests of the population. Only developers are interested to "squeeze" the maximum of meters out of the given site. 16–25-storied buildings are constructed with large areas, intended for public activities, and given, in practice, to kindergartens and schools. Perception of the city identity cannot form in citizens who live in such multi-apartment high-rise houses. Courtyard territories increasingly become the place of psychological discomfort and vandalism.

Looking at the buildings with turrets and "rotundas", "cottages" and penthouses on last floors designed by different authors of projects within one micro-district, it is possible to note tawdriness — lack of harmony in the combination of shapes, color and details of space.

Inefficient arrangement is present in everything from spatial planning solutions to the transport logistics. In a word, such micro-districts of Saint Petersburg do not have their own "face".

As Richard Florida, an American economist and the author of the "creative class" theory truly noted, "Human beings crave physical beauty. We look for it in the things that surround us, and especially in the places we live. Economists call this the "beauty premium". People are drawn to and pay more for something that has aesthetic appeal." [1]

A big mistake was made when they allowed the construction in Saint Petersburg beyond a certain city boundary without taking into account architectural and planning ideas.

The problem of identity loss, poor quality of the living environment in peripheral (dormitory) districts of Saint Petersburg is recognized by the city administration too.

Recently, the government of Saint Petersburg and the professional community of architects, developers and

teachers in higher education have paid significant attention to issues of improvement of the city residential environment quality, search for an answer to the question what environment can be considered comfortable, what can be called the "Saint Petersburg style" in the building development of new residential areas.

2 Methods

The goal of this article is to reveal tendencies of the development of historical cities taking into account identification characteristics of the established urban environment, in relation to conditions of designing and construction in Saint Petersburg.

Upon comprehension of general tendencies of construction, it becomes obvious that the systematic analysis is needed to understand the issues related to identification of the urban environment for the people who are living in it.

Following tasks were stated in the carried out study: to define the concept of urban environment identification and reveal identification characteristics.

In the study, identification is understood as establishment of the identity of one object to another on the basis of coincidence of characteristics.

To ensure identification of a smaller structural unit (block) with a larger one (with the city), it is necessary to ascertain which architectural and city-planning features are the most characteristic for a historical city. These distinctive features can be marked out in the zone of the historical development, where they were formed most clearly and fully.

The study allowed analyzing examples out of the international practice and draw some conclusions about approaches to designing in cities which have a long tradition of evolutionary development.

We will rely on the experience of designing historical cities in Italy, a country with the rich cultural heritage, where great attention is paid to delicate conservation of population lifestyle traditions.

In Italy, the major problems are practically solved: the legislation is well proved and there exists interaction of legislative bodies and the civil society, which does not accept construction actions that violate the urban environment integrity and introduce disharmony in the city development [2]. The legislation specifies strict requirements to the new construction in historical cities. The system of restrictions concerns both the planning structure of residential territories and construction parameters of separate buildings. These restrictions restrain legislatively the desire of developers to squeeze the maximum profit from the developed plots.

Solving of these problems becomes fundamental for development of historical cities with characteristic identification features.

3 Results

As a result of the on-site inspection, carried out in Florence, and the analysis of city architecture formation in order to determine main tendencies of the creation of a harmonious architectural environment, the following construction development principles were revealed, which allowed keeping or changing identification.

Maintenance of the urban environment development traditions and the established density of a development

The experience in Florence has shown careful attitude to the development of master plans. The new district "Giovanni Spadolini" thoroughly fits in the historical part of Florence and practically does not outstand the general footprint of the city on the master plan (Figure 1).

Figure 1. Axonometric perspective geometry of a new part of the "Giovanni Spadolini" district, Florence

During the development of master plans for new districts of historical Italian cities, the following principles are retained: development density; block development with maintenance of streetscape, arrangement of blocks with courtyards intended for use by only those citizens who are living there [3].

An important factor is maintenance of development modularity, which is reflected in maintenance of dimensions of blocks, spacing between residential streets, their historical profile. Residents of the historical city are used to its scale and, when observing necessary sanitary and hygienic requirements, no changes are welcome.

Maintenance of the street development outline as a characteristic of urban environment identification

The street development outline is an identification characteristic. Each city has its own "sky line". In Florence, projections of cornices with the use of timber rafters were applied since the Middle Ages, having survived the Renaissance period. Such a constructive and architectural technique persists to this day, it is relevant as both a means of protection against the sun in the summer, and from rains in the autumn and in the spring; it is an important element for the city, as it continues to effectively solve functional and aesthetic tasks. At the same time, the technology of creation of cornice projections has been sufficiently modernized: modern materials and methods of construction are used (Figure 2).

Figure 2. Conventional outline of modern city blocks in Florence

Modularity of the urban development, use of traditional parameters of residential buildings

The parameter "overall dimensions of the street facade of a residential building" is an important factor of urban environment identification. This parameter has been formed historically, in accordance with the rules of sale of land plots and laws on their construction development and use. Each historical city has its own characteristic parameter. In Italy, which was a fragmented country, there was no single law on the plot development parameters for all cities, therefore, each city has its own matrix (face). Principles of the plot development and division of the street development into facades of separate buildings constitute a conservative process and are used at the present time (Figure 3).

Figure 3. Modularity of the modern development. Via Vincenzo Gioberti, Florence

It can be concluded that conservation of traditions of the urban environment development, the established density and modularity of the urban development, outlines of streets, use of traditional parameters of residential buildings as signs of urban environment identification are necessary to ensure identification of a smaller structural unit (block) with a larger one (with the city). Let us call it identification of the first level.

Let us consider identification of the townsman with the region of his residence to be identification of the second level. Its essence lies in the fact that a resident of a particular region shall and can identify himself/herself with the very environment wherein he/she lives. Identification of the second level can be achieved by other means.

Retention of architectural techniques associated with typological features of residential buildings

The typology of residential buildings is primarily connected with the lifestyle of citizens, peculiarities of their everyday life and traditions. A demonstrative example is the development of streets with conventional buildings of trade institutions, roofs of which are used as open terraces with landscaped spaces and gardens. Such type of the house was established due to high density of the development and the need to create large areas for landscaping and recreation. The need in these functions remained, the architectural technique proved to be successful and stable (Figures 4, 5). This architectural technique is characteristic not only for the urban development. It is also used in the Pitti Palace ensemble, emphasizing complete conformity of the technique to the lifestyle of citizens with social and climatic specific features.

Figure 4. Residential building with a terrace above the trading space, Florence

Figure 5. Modern construction development in Via Quintino Sella in Florence

Certain color scheme of the street development

For many centuries, Florence was a republic where citizens had rights which could not be disputed by authorities. Formation of a society consisting of individuals required manifestation of the character and preferences of a person. Upon selection of the color scheme of buildings and their details, the individuality was shown, therefore, even two houses, standing next to each other, often have different color schemes. Development of a colored streetscape became an identification characteristic, which is used in the modern architecture too (Figures 6, 7).

Figure 6. Color scheme of the historical urban development, Florence

Figure 7. Color scheme of the modern urban development, Florence*Use of characteristic architectural details*

Identification of the second level can be achieved by such means, as signs, as well. To this effect, it is sufficient to introduce the principle of a recurrent sign. Any visual image that accompanies a person in his/her environment and practically does not occur outside of it can be a sign, e.g. courtyard furnishings, street design elements peculiar only to a given district, plants growing only in this block and nowhere else. Due to recurrent signs of this kind, the neighborhood residents acquire an awareness of their own distinguishing characteristic in comparison with inhabitants of other parts of the city, on the one hand, and a sense of solidarity and unity with neighbors, on the other hand.

Architectural details are signs which manifest the purpose of the building, its national and temporal identity, reflect the culture of a particular people. Details always emphasize the years of building construction and cannot be mechanically transferred from one period to another. There is a certain regularity related to identification through the use of architectural details. In this respect, an architectural detail is a rather conservative element and a means of urban environment identification (Figures 8, 9).

Figure 8. Details of the windows of a residential house of 19th century construction, Florence

Figure 9. Details of a modern residential building, Florence

4 Conclusions

Revealing of modern tendencies of the development of historical cities taking into account the established identification characteristics, based on examples of the experience in Italy, showed the need in a comprehensive individual approach to the creation of the architectural environment, which is comfortable for living.

Following the above principles such as conservation of traditions of the urban environment development, conservation of the established development density and the street development outline, as an urban environment identification feature; modularity of the urban development, use of traditional parameters of residential buildings, retention of architectural techniques associated with typological features of residential buildings, a certain color scheme of the street development, use of characteristic architectural details will make it possible to preserve the familiar urban environment.

Examples of different types of reconstruction, modernization, new construction in historical cities show the versatility of this problem and the need to search for individual ways for different actual cases of construction. There are no and can be no common, once and forever established rules. They are worked out only as a result of constant public discussions, determination of the city development strategies. This process must be evolutionary. All changes that cannot be avoided shall occur step by step, without violating rules and principles of the city development, and give people the possibility not to lose identification with their "small homeland". Only this way, new modern multi-functional high-quality districts, integrated into a system of public spaces, can be created.

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STUDY OF OSCILLATIONS DAMPING PROCESSES IN VIBRATING CONSTRUCTION MACHINES

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Keywords: vibration, resonance, shock absorber, construction machines .

Abstract. Many vibrating construction machines operate in the super-resonance mode. Resonance occurs during periods of acceleration and deceleration of the vibrational motion and is accompanied by a sharp increase in the vibration speed and amplitude. Resonance oscillations have a negative effect on the operation of machines due to the increase in dynamic loads, which reduce the resource and cause increased noise.

The goal of the article is theoretical substantiation of the possibility to design mechanical devices for controlling the oscillatory process and develop an example of the practical implementation of such device.

The method of controlling oscillatory processes in vibrating construction machines lies in the use of two-mode shock absorbers. In the operating mode, shock absorbers create a minimum resistance to motion. In the resonance mode, resistance increases, therefore, the amplitude decreases to a predetermined value.

The simulation of the oscillatory process of a vibrating construction machine is performed in the article in order to determine characteristics of a damping device ensuring the suppression of resonance oscillations and, at the same time, not affecting the working vibratory process. A new technical device is proposed which implements required oscillation characteristics is proposed. The device is made in the form of a hydro-pneumatic shock absorber, similar in design to automotive shock absorbers. Design features of the new shock absorber provide low resistance to vibration at the working amplitude and large resistance at the resonance amplitude.

The originality of the research is presented in the form of a new shock absorber design protected by the utility model patent, and in the form of the mathematical model of its operation.

Scientific novelty lies in the mathematical description of oscillations damping by the new shock absorber, in development of a computer program for calculation of the oscillatory process according to the developed mathematical model, in results of the computer simulation of the shock absorber's operation.

1. Introduction

Majority of vibrating construction machines (VCM), i.e. conveyors, screens, crushers, vibrating tables, etc., operate in the super-resonance mode [1, 2]. Resonance occurs during periods of acceleration and deceleration of the vibrational motion and is accompanied by a sharp increase in the vibration speed and amplitude. Resonance oscillations have a negative effect on the operation of machines due to the increase in dynamic loads which reduce the resource and cause increased noise [3].

Protection from the destructive effect of resonant vibratory loads is a serious technical problem in designing VCM. There are many technical solutions for damping resonance vibrations in machines. However, in our opinion, one of the most successful and proven in practice methods (by the example of the operation of automotive transport) is the use of telescopic hydro-pneumatic shock absorbers [4, 5, 6], the damping coefficient of which varies in proportion to the speed of oscillations.

2. Methods

The research subject is oscillatory processes in vibrating construction machines.

The main goal of the article is theoretical substantiation of the possibility to design mechanical devices, such as automobile shock absorbers, for controlling the oscillatory process and develop an example of the practical implementation of such device.

The proposed method of controlling oscillatory processes in vibrating construction machines lies in the use of two-mode shock absorbers. In the operating mode, shock absorbers shall create a minimum resistance to oscillatory motion. In the resonance mode, the shock absorber resistance shall increase to decrease the amplitude to a specified value.

Shock absorbers, applied in the automotive transport, operate actually in two modes, i.e. throttle and valve modes [5, 9, 10 and 11]. At small amplitudes and, consequently, slow speeds, valves are closed, and the liquid is forced through throttle channels in the piston. This is the throttle mode. With increasing the piston motion speed, the hydraulic resistance to its movement increases, and valves open, reducing the resistance. This is the valve mode.

The damping force, created by such shock absorber [5, 6] is as follows:

$$Pd = k \times v^i \qquad (1)$$

where k is the damping coefficient of a shock absorber; v is the speed of vertical VCM movement; i is the exponent of a power.

Depending on the i exponent, the shock absorber performance curve can be linear (i = 1.0), progressive (i > 1) and regressive (i < 1). The performance type depends on the design features of the shock absorber. Factors k_c (compression stage) and k_e (extension stage) have different values. Figure 1 presents a progressive performance curve, which most corresponds to the specified conditions. This is a schematic representation of the performance curve, since the real performance curve will be nonlinear (it is shown by the dotted line).

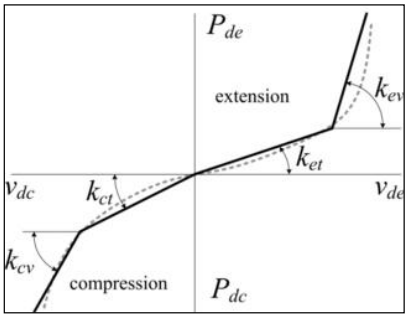


Figure 1. Shock absorber performance curve: P_d is the damping force; v_d is the piston speed; k is the damping coefficient; indices: c — compression; e — extension; t — throttle mode; v — valve mode

However, as per parameters, industrial shock absorbers (ISA) are not suitable for the use in VCM. Namely, the working stroke (amplitude of vibrations) in VCM, corresponding to the throttle mode, is 1–10 mm, while in ISA it is by an order of magnitude greater. Also, the damping value in the throttle mode is too high, at least tens of newtons, which leads to increased energy consumptions per a vibration process.

Therefore, the development of a new shock absorber design is required, which would satisfy VCM operating conditions to the fullest extent possible.

3. Results

Let us consider the procedure of substantiation of shock absorber parameters on the basis of simulation of the VCM operating process (Figure 2).

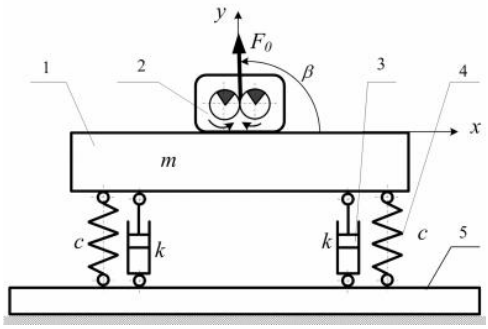


Figure 2. Design VCM model:

1 — VCM housing; 2 — vibrator of directed oscillations; 3 — shock absorbers; 4 — springs; 5 — basement; m — VCM mass with the material; β — tilt angle of the disturbing force

The VCM comprises housing 1 with the material and is driven by vibrator of directed oscillations 2. The VCM is mounted on basement 5 by means of four springs 3 and four shock absorbers 4.

Forces acting on the VCM are described by the following equation [8]:

$$G + P_i + P_s + P_d = F(t) \qquad (2)$$

where $G_к$ is the VCM weight; P_i is the inertia forces; P_s is the spring force; P_d is the damping forces of shock absorbers; $F(t)$ is the external force.

The external force produced by the vibrator varies in time according to the law [1]:

$$F(t) = F_0 \times \cos(\Omega t), \qquad (3)$$

where F_0 is the amplitude value of the vibrator disturbing force, N; Ω is the frequency of oscillations in the vibrator, sec^{-1}; t is the current time, sec.

The restoring force of the spring [5, 8] is:

$$P_s = -c \times l, \qquad (4)$$

where c is the spring force, N/m; l is the spring travel (deformation), m.

The damping force from the direction of the shock absorber is:

$$P_d = -k \times v, \qquad (5)$$

where k is the damping coefficient of the shock absorber, Ns/m (newton per second/meter); v is the travel speed of the shock absorber piston, m/s.

Then, the second law of Newton can be written in the following form [8]:

$$m \times a = -c \times l - k \times v + F(t). \qquad (6)$$

This expression for the plane design case and the so-called small forced vibrations [3] in the form of a differential equation of the vibrating conveyor motion will take the following form [2, 8]:

$$m \times \ddot{y}(t) + 2 \times c \times y(t) + 2 \times k \times \dot{y}(y) = F(t), \qquad (7)$$

where y is the value of movement in the direction of the ordinate axis.

In order to simplify the model, tilt angles of the disturbing force, axes of the shock absorbers are selected the same, therefore, their trigonometric functions are reduced in the equation.

Let us represent equation (7) in the following form:

$$\ddot{y}(t) + \omega_0^2 \times y(t) + h \times \dot{y}(t) = \frac{F(t)}{m}, \qquad (8)$$

where $h = k/(2 \times m)$ is the damping coefficient of the shock absorber (coefficient of oscillations damping), N·s/(kg·m); $\omega_0 = (2 \times c/m)^{1/2}$ is the natural oscillation frequency of the conveyor, sec^{-1}.

The following equation determines the established amplitude of oscillations [9]:

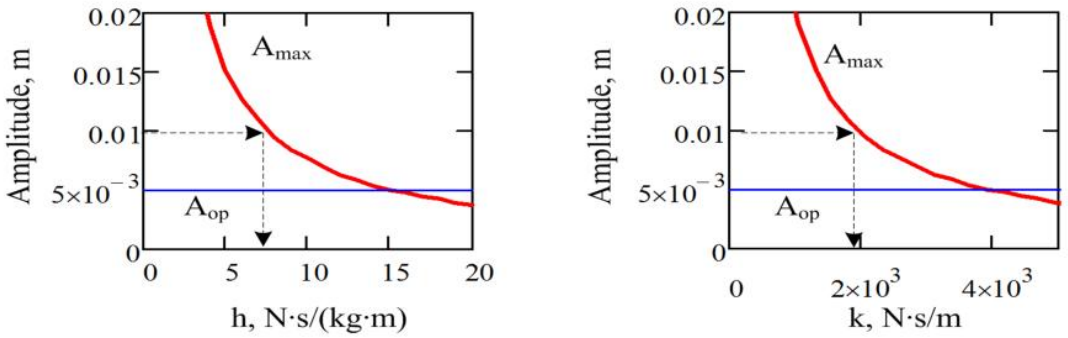
$$A_{op} = F_0 \Big/ \left(m \times \sqrt{(\Omega^2 - \omega_0^2)^2 + 4 \times h^2 \times \Omega^2} \right). \qquad (9)$$

The maximum value of the amplitude A_{max} corresponds to the cyclic frequency $\Omega_0 = \sqrt{\omega_0^2 - 2 \times h^2}$ which is somewhat less than the natural cyclic oscillation frequency of the system $\Omega = \sqrt{\omega_0^2 - h^2}$.

$$A_{max} = F_0 / (2 \times h \times m \times \Omega_0). \qquad (10)$$

An increase in the damping coefficient h leads to smoothing of the resonance curves A (Ω) and decrease in A_{max}, i.e. to significant weakening of the resonance phenomenon. Resonance completely disappears, when $h \geq \omega_0 / \sqrt{2}$

Figure 3 shows the result of the Mathcad calculation of the resonance amplitude in dependence on the shock absorber damping coefficient. Values of damping coefficients h of the shock absorber resistance k are obtained based on the calculation of the resonance amplitude. An example of the Mathcad calculation for the test bench showed that the damping coefficient should be not less than 2,000 N·c/m to limit the maximum amplitude in the value of 0.01 m. This is the maximum damping value which the shock absorber shall develop.



a b

Figure 3. Dependence of the resonance oscillation amplitude of VCM A_{max} on the damping coefficient (a) and the shock absorber damping coefficient (b)

rod; *l* — distance from the sleeve to the piston; *α* — angle of shock absorber installation

The next task is to select a shock absorber with the found oscillation damping characteristics. The authors have developed several shock absorber designs which meet this condition. The most interesting is a design (Figure 4) obtained through the modernization of the industrial prototype. The shock absorber damping characteristics are determined by the ratio of the amplitude to the stroke of pistons *l*. When the oscillation amplitude is below *l*, the liquid overflows through the gap $\delta_{sr} = (d_s - d_r)/2$ between the sleeve and the rod, and when it is above *l*, the liquid overflows through the gap $\delta_{sn} = (d_s - d_p)/2$ between the sleeve and the piston, which is significantly smaller than the gap in the first case. Therefore, the resistance to the liquid overflow will be greater in the second case, and the shock absorber damping coefficient will also be greater.

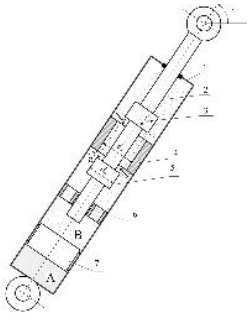


Figure 4. Shock absorber of resonance oscillations:

1 — cylinder; 2 — rod; 3, 5 — piston; 4 — sleeve; 6 — alignment sleeve; 7 — gas piston; A — gas zone; B — liquid zone; G_r— gravity force of the rod with the piston; d_p — piston diameter; d_r — rod diameter; d_s — sleeve diameter; δ — gap between the sleeve and the

The shock absorber operation is as follows. The shock absorber is installed between the vibrating and the base parts of the machine. In the operating mode of the vibrating machine, oscillatory motion of pistons 3 and 5 takes place beyond hydraulic sleeve 4, i.e. the established value of the amplitude A_{op} is less than the gap value *l*. In this mode, the resistance to rod 2 motion is conditioned by the following forces: rod friction at the contact spot with cylinder 1; hydraulic friction of moving parts inside the cylinder; the inertia of moving parts. The damping coefficient of the shock absorber is minimum in this mode.

But at the time when resonance occurs, when the amplitude exceeds the value of *l*, pistons 3 and 5 will enter inwards hydraulic sleeve 4, which will cause the liquid pushing through small gaps between sleeve 4 and pistons 3 and 5, accompanied by a sharp increase in the hydraulic resistance. The shock absorber damping coefficient will increase proportionally too. The amplitude of resonance oscillations will be decreased.

The required shock absorber damping coefficient is set by the difference between the internal diameter of hydraulic sleeve 4 and the diameter of piston parts 3 and 5. The value of the amplitude limit is set by the difference between the width of sleeve 4 and the distance between piston parts 3 and 5.

Alignment sleeve 6 serves for guiding rod 2 movement strictly along the cylinder 1 axis. Large gap between the central hole of sleeve 6 and rod 2 substantially eliminates the friction between them. Perforations (holes) in sleeve 6 make the liquid flow unobstructed in the adjacent inner space of cylinder 1.

Pneumatic piston 7 compensates the change in the internal volume of hydraulic space B during the rod 2 movement.

Let us compose a mathematical model of shock absorber operation. During shock absorber operation, processes of compression and expansion alternate. During shock absorber compression, the energy of external forces is expended for gas compression P_g, pushing liquid P_l through the annular space between the sleeve and the rod with the piston, overcoming friction forces P_f in seals, inertia forces P_i and gravity forces G_g of the moving rod with the piston:

$$P_C = P_g + P_l + P_f + P_i - G_g \cos\alpha . \quad (11)$$

During rebound travel, the force in the shock absorber is described with the following equation:

$$P_E = -P_g + P_l + P_f + P_i + G_g \cos\alpha \quad (12)$$

The main difference from automotive shock absorbers lies in the process of forming the resistance P_l to pushing the liquid through sleeve-rod gaps with a low resistance and sleeve-piston gaps with a high resistance. This resistance is the greatest of all and has a major effect on the formation of shock absorber performance. From hydraulic calculations [7] it is known that the value of resistance to the piston movement in the sleeve is determined by the hydraulic resistance of the liquid overflow through the annular gap:

a) at the oscillation amplitude, which is *less* than the distance *l*, the liquid flows through the sleeve-rod gap

$$P_{l1} = \frac{\rho_l \times v_p^2 \times F_p^2}{2 \times g \times \mu^2 \times S_{rs}^2}, \quad (13)$$

b) at the oscillation amplitude, which is *more* than the distance *l*, the liquid flows through the sleeve-piston gap

$$P_{l2} = \frac{\rho_l \times v_p^2 \times F_p^2}{2 \times g \times \mu^2 \times S_{ps}^2}, \quad (14)$$

where ρ_l is the liquid density; v_p is the piston movement speed; F_p is the effective area of the piston; S_{rs} and S_{ps} are areas of the annular gap between the rod and the sleeve and between the sleeve and the piston, respectively; μ is the flow coefficient, depending on the shape of holes and the liquid viscosity (0.65÷0.75).

The effective area of the piston and annular gap during movement:

a) movement downwards

$$F_{p\downarrow} = \pi \times d_p^2 / 4; \quad (15)$$

$$S_{ps} = \pi \frac{(d_s^2 - d_r^2)}{4}; \quad (16)$$

b) movement upwards

$$F_{p\uparrow} = \pi (d_p^2 - d_r^2) / 4. \quad (17)$$

$$S_{ps} = \pi \frac{(d_s^2 - d_p^2)}{4} \quad (18)$$

Figure 5 shows shock absorber performance simulated in the Mathcad.

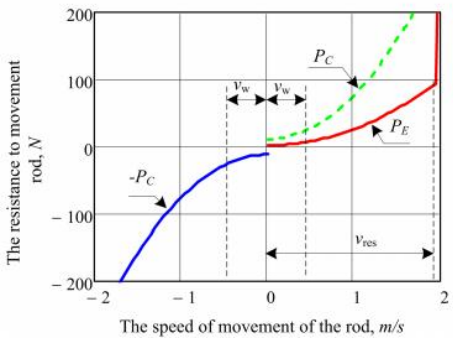


Figure 5. Shock absorber performance:

v_w is the rod working in the operation mode of the vibrating machine (in the moment of oscillation axis crossing); v_{res} is the rod working in the mode of entering resonance It can be seen from the diagram that it is the resonance velocity which is limited by the damping force of the shock absorber, and P_E performance gives the most explicit visual representation. The moment of the force jump corresponds to the piston entry into the sleeve space. The shock absorber begins to damp sharply the oscillatory process, i.e. limit the oscillation amplitude.

Superposition of the vibrating machine performances (see Figure 3) and the shock absorber allows selecting the required values of the shock absorber damping coefficient in the

operating and resonance modes (Figure 6), and thus selecting the required values of the shock absorber design parameters, namely: sizes of the sleeve, pistons and the rod, and pressure in the gas chamber.

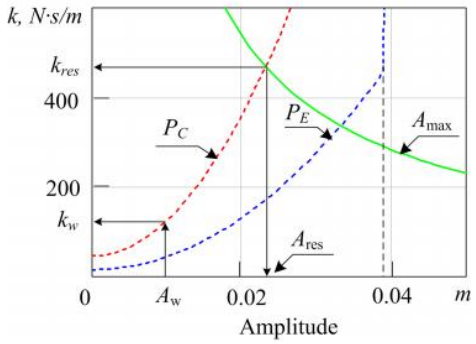


Figure 6. Dependence of the shock absorber damping coefficient on the vibrating machine oscillation amplitude: A_w is the working oscillation amplitude; A_{res} is the resonance amplitude

4. Conclusions

The mathematical simulation of the oscillatory process of the vibrating construction machine with the installed damping device was carried out in the article. The mathematical expression of the damping coefficient value was obtained; this coefficient ensures damping resonance oscillations.

A new design of the hydraulic shock absorber for the vibrating construction machine is described, this design allows obtaining the minimum resistance in the machine operating mode and required resistance upon occurrence of resonance oscillations.

A mathematical model of the joint operation of the vibrating machine and the shock absorber was compiled and implemented in the Mathcad environment. Superposition of the vibrating machine performances and the shock absorber allows selecting the required values of the shock absorber damping coefficient in the operating and resonance modes, and thus selecting the required values of the shock absorber design parameters.

5. Acknowledgments

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SOLVING PROBLEMS OF MAJOR REPAIR OF LARGE-PANEL BUILDING EXTERNAL WALLS

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Keywords: technical condition, major repair of residential buildings, panel joints of large-panel building external walls, thermal protection of building.

Abstract

Analysis of the technical condition of facades' external wall structures in frameless large-panel residential buildings of 137 series, located in residential areas of Saint Petersburg, analysis of design concepts of wall panels by material, types of vertical and horizontal inter-panel joints were carried out.

Visual inspections of facade walls of 137 series large-panel buildings were performed. Confirmatory thermal-technical calculations for external walls were carried out. Results of the technical condition inspections of facade wall panels and their joints made it possible to draw a conclusion about the need for carrying out major repair of facades.

According to the results of the carried out analysis of the present state of facade walls of large-panel residential buildings, the study goal is the following: development of scientifically substantiated options of design considerations for major repair of facades' external walls of 137 series frameless large-panel residential buildings in Saint Petersburg for increase of thermal-physical properties of external facade walls, decrease in heat losses of the building as a whole and improvement of the architectural expression of facades, as a result of their major repairs.

To implement the set goal, the following two problems were solved:

- a technology of thermal and hydraulic insulation of inter-panel seams of external wall panels during their repair with the use of modern materials of seam filling was developed;
- options of facade insulation were developed on the basis of the analysis of modern facade systems, taking into account import substitution of materials and structural details;
- a technological sequence of facade thermal insulation arrangement in a large-panel building was developed for various facade systems.

On the basis of the technical and economic comparison of options of facade systems, an economically feasible facade system for large-panel residential buildings of 137 series was chosen. The proposed option of the facade system was submitted for consideration to the Housing Committee of the Saint Petersburg Government.

1. Introduction

Ensuring adequate living conditions and the housing stock integrity is the priority tendency of the social and economic development of the modern Russia.

Year by year the housing stock of Russia becomes notably dilapidated, the service life and wear degree of residential houses increases. According to the Russian Federal State Statistics Service, more than 1.5 million residential buildings in Russia today, wherein 45 million people live, need major repair. The area of dilapidated residential space and hazardous dwellings is 93.3 million m2 [1].

The problem of the housing stock reproduction is relevant for Saint Petersburg too. The housing stock of the city is characterized by a large number of buildings built in different periods, which need major repair and reconstruction. At the present time, 90.2 million m2 of housing in Saint Petersburg have the dilapidation of up to 30%, 29.2 million m2 of housing have the dilapidation of 31–65%, 0.15 million m2 have the dilapidation of 66%–70% and 0.03 million m2 of housing have the dilapidation of more than 70% [1].

According to the Saint Petersburg Statistics Service, 992 houses have emergency structural elements, including 353 houses in the Tsentralny District, 158 houses in Admiralteysky District, 147 houses in Nevsky District, 138 houses in Vasileostrovsky District, 116 houses in Petrogradsky District, 80 houses in Kalininsky District of Saint Petersburg as of 01.01.2016 [2].

One of the most relevant tasks in the reproduction process of the housing stock in Saint Petersburg is the reproduction of the residential development of the industrial house building period, as a result of major repair.

According to [3], major repair of capital facilities represent the replacement and/or the renovation of building structures of capital facilities or elements of such structures, except for load-bearing construction structures, as well as the replacement of separate elements of load-bearing construction structures with similar or other elements, improving performances of such structures, and/or the renovation of the specified elements.

Gradual transition from subjective selection of residential houses for major repair to responsible assignment in dependence on a building service life represents a serious qualitative change in the approach to major repair. One of the most important issues in the organizational system for major repair of residential buildings is determination of the time of the repair beginning and its intervals. The analysis of the building state in time shows a complex interconnection of a "building – elements – time" system.

Non-fulfillment of the timely repair of structures leads to the increased wear and a sharp increase of repair cost. For example, rescheduling of major repair of a standard panel five-storied building for 3–4 years after expiry of the set time limits increases its cost by 18–20%.

The most important part of the major repair organization is the development of its strategy. Theoretically, there are two repair options: according to the technical condition, when the repair begins after appearance of a fault; preventive repair, when the repair begins before a fault occurs (in order to prevent it). Studies showed economic and social advantages of the second option.

The combination of two strategies is applied in practical use of buildings in Saint Petersburg: major repair is scheduled according to the useful life of a building, and the volume of repair works is determined according to the technical condition. According to [4], the duration of the effective operation of large-panel buildings is 15–20 years before their scheduling for major repair. Such a combined strategy should be considered as the transition stage to the system of preventive repairs.

The goal of the state policy in the field of major repair of common property in multi-apartment buildings is to create and provide sustainable functioning of regional systems for major repair of common property in multi-apartment buildings which ensure its timely implementation, the required quality and reasonable costs of works (services) with a gradual transition from prevalence of the system of the state organization of major repair through the system of regional operators to prevalence and distribution of the model of major repair organization through a system of special accounts.

In 2014, regional major repair systems were launched in the Russian Federation. These systems are based on the major repair funding at the expense of property owners in multi-apartment buildings [5, 6].

2. Methods

Considering issues related to solving tasks of major repair of residential buildings, the authors of this article, who are members of an expert group of the Government of Saint Petersburg on issues related to the implementation of the regional program for major repair of common property in multi-apartment buildings of our city, consider the development of design decisions for major repair of facades of panel multi-apartment buildings, constructed in the period of the 1970–1980, to be one of the topic issues. It is known that numerous defects and damages, which take place during the long-term operation of frameless large-panel buildings, include, first of all, defects and damages in external wall panels and inter-panel joints. Special attention should be paid to the condition of sealing and strengthening of horizontal and vertical expansion joints, availability and nature of cracks in the main layer of wall panels and their texture layer with regard to walls of large-panel buildings. Approximately 30–35% of leakages, frost penetrations, breaking-down of the internal finishing of rooms, peeling and breaking-down of the external finishing of large-panel building walls is due to unreliable sealing of joints of wall structure elements. This is due to the incompleteness of design solutions, poor-quality joint sealing, etc. In order to reveal non-compliance with regulatory requirements, it is necessary to know these requirements very well. The most important of all existing operating requirements are considered below.

Joints of panels shall meet the following requirements: water intrusion protection by means of sealing mastics, subject to the technology of their application, and qualitative preparation of the surface; air protection by means of sealing gaskets made of porous elastic insulation, porous rubber gasket, sealing cord, oakum and other materials with compulsory squeezing of not less than 30–50%; thermal protection by means of the installation of thermal insulation blocks. Standardized opening of joints as a result of temperature deformations is 2–3 mm for vertical joints and 0.6–0.7 mm for horizontal joints.

Water intrusion protection in sealed joints is achieved with the help of a sealant, i.e. an air sealing material with compulsory squeezing of 30–50%; thermal insulation — with thermal insulation blocks or arrangement of haunches with a width of at leats 300 mm. Leaking end joints shall be sealed from the outside with effective sealing materials (elastic gaskets and mastics).

Maintenance of walls should be carried out during the entire period of operational service. Minimum effective service time for walls is 50 years for large-panel buildings with mineral wool insulation, and for large-panel single-layer lightweight concrete structures — also 50 years. Minimum service time for sealed joints is 80 years for panels of external walls with non-hardening mastics; for panels of external walls with hardening mastics — also 80 years, and 60 years for joints of windows and door blocks connected to faces of openings.

3. Results

To ensure the leak proof of joints, it is necessary to carry out preventive activities for sealing of joints and repair of wall panels within the time frame preventing the loss of their operating ability. Planning the development of design solutions of the facade major repair, the expert group selected standard 137 series frameless large-panel residential buildings in Saint

Petersburg, built in the 1970s, as the object of the study.

A brief specification of this series is as follows: the building structural scheme is frameless with longitudinal and transverse bearing walls; foundations are piled.

External walls are suspended and self-supported (projection walls), 1,000 = ɣ kg/m³. The bond of external walls is single-row. Structural concepts of external longitudinal walls are single-layer claydite concrete panels, 300 mm thick. External face of wall panels is finished with a layer of ceramic tiles on 30 mm thick cement mortar; internal face — on 15 mm thick cement mortar.

Basement panels are claydite concrete, 300 mm thick under the suspended walls, and 450 mm thick under the bearing end walls. Floor structures are reinforced concrete flat panels, 160 mm thick. Projecting and recessed balconies and bay windows are made of heavy reinforced concrete. Panel joints of external walls: for series 137 — vertical with labyrinth joint and with self-fixing elements in the form of inserted locks. The pit is infilled with claydite concrete. Horizontal joints are with a protective "tooth", the panel is installed on the cement mortar in the area of the floor slab support, with caulker's oakum in the end of the floor slab. Sealing mastic is applied along sealing gaskets, followed by a protective layer.

In 137.11, 137.12 and 137.13 series with a labyrinth joint and welded joints, self-fixing is absent. In 137.13.2 and 137.12.2 series with a symmetrical joint on welded joints, self-fixing is absent. Stairs: reinforced flights and landings. The roof is made of prefabricated reinforced-concrete thermal insulation slabs. The roof has a warm through attic and roofing from the rolled material. Downpipe: internal. In Saint Petersburg, panels are manufactured at DSK-2 factories (Obukhovsky, Polyustrovsky, and Parnassky).

According to the developed technique of major repair of frameless large-panel building facades, visual or visual-tool inspection is needed for detection of defects, causes of their occurrence and their further elimination. During he study, the authors proposed the technological sequence of works aimed at visual inspection of panels and joints of external wall panels of frameless large-panel buildings:

1. Exterior check of wall panels of the building is performed. Maps of panel defects and damages, including basement and parapet panels, are drawn up. Panels, which have signs of joint damages, are marked out. Photo fixation of the most dangerous defects and panel damages is carried out.

2. Thorough selective inspection of joints of external wall panels is carried out. All joints of external walls in the places of location of kitchens, balconies (recessed balconies), including staircase galleries, are inspected. If any types of joint damages are detected, they are opened and cleared from the inside of the building premises. Special attention is paid to corner assemblies of panel joints (when 4 panels are connected). This assembly is opened for inspection of inserts and their mounting studs.

3. Inserts and studs are cleaned of residual concrete and dust. Then they are inspected to assess actual percentage of corrosion damages of inserts and studs made of reinforcing bars.

4. If corrosion of metal elements is detected, technical solutions for restoring the needed serviceability of joints are developed.

5. Duplicate studs made of reinforcing bars are welded to the inserts of panels. After welding of rods, the opened assembly is foamed with frothing mastics.

An important stage in the process of major repair of inter-panel joints of frameless large-panel building external wall panels is their thermal and hydraulic insulation.

The technology of thermal and hydraulic insulation of inter-panel joints of external wall panels upon major repair of residential buildings, developed by the authors, provides for repair operations in the following sequence:

1. Removal of old external insulation and thermal insulation inside the joint.

2. Cleaning of the panel edge from dirt and stratifications.

3. Leveling of the panel edge with cement repair composition.

4. Stripping the masking tape on both edges of the joint.

5. Putting the foamed polyethylene cord "Vilaterm" into the joint.

6. Laying mineral wool into the joint (according to GOST 31913-2011).

7. Putting the second layer of "Vilaterm" into the joint.

8. Application of the "Sika Primer 3N" primer in the area of glue pasting.

9. Application of the EUROLASTIK TK 51 glue paste with the help of a spray gun onto the surface prepared with the primer.

10. Application of the "EUROTEK TK Fugenband" polysulfide tape.

11. Smoothing of the edge with a spatula.

12. Removal of the masking tape.

13. Smoothing of the bevel with a spatula.

14. Butt splice of the tape with strengthening from the back side.

In the process of visual inspections, on the basis of confirmatory thermal-technical calculations for external wall panels of the standard 137 series, it was established that the external panels of this series met thermal-technical requirements only in 40%, according to the requirements of energy saving. Additional thermal insulation of external walls is required.

The authors developed an option of facade insulation of series 137 large-panel buildings with thermal insulation plaster, i.e. a non-ventilated facade system. For the proposed plastering system for repair and thermal insulation of facades of large-panel

buildings, the following materials, including materials of import substitution, are recommended:

1. Materials for repair and leveling of the concrete surface (thixotropic fast-hardening dry repair materials, ultimate compressive strength — 30–40 MPa, adhesion to concrete — not less than 1.7 MPa, maximum aggregate fineness — 0.63–1.25 mm):

- RUNIT, finishing repair fast-hardening composition (Russia, Saint Petersburg);
- Hydrocem R3 T300 FINISH (Russia, Saint Petersburg);
- BIRSS RSM-350-2M (Russia, Moscow);
- MasterEmaco N 5100 (Germany, there is also a plant in the Moscow Region);
- Planitop 400 (Italy);
- Sika® MonoTop®-620 (Switzerland, there are also plants in the Tver Region and Moscow Region).

2. Profiled drainage membrane (profile height — 8 mm).

3. Thermal insulation plaster (thermal conductivity factor — not more than 0.1 W/m·K, ultimate compressive strength — not less than 2.0 MPa).

4. Welded galvanized mesh, 20×20×1 mm.

5. Anchor system for wet facades.

6. Decorative plaster (rough coating).

7. Structures for expansion joints.

8. Facade paint.

The authors developed the following technological sequence for thermal insulation of large-panel building facades:

1. Mechanical cleaning of the surface from peeling elements of surfacing and destructed concrete is performed. The surface of areas to be repaired is cleaned from dust with compressed air and primed with a polymer primer. A repair composition is applied on the areas where concrete or decorative elements are lost to level flush with the surface of panels.

2. A profiled drainage membrane is installed to serve as an element forming a ventilated air gap between the external surface of a panel and a layer of thermal insulation plaster.

3. Spacers for thermal insulation plaster are installed and fixation of a galvanized welded mesh is carried out (these spacers can serve as additional fixing for the drainage membrane). The number of spacers is determined at the rate of 6–7 units per one square meter of the wall surface.

4. Thermal insulation plaster with the total thickness of up to 40 mm is applied.

5. Expansion joints are arranged. It is recommended to arrange them with the method of cutting along the plaster layer with the subsequent treatment of cut edges with strengthening polymer primer and further installation of an overlaid structure.

6. Mineral decorative plaster is applied. The plaster is applied with the thickness equal to the size of the aggregate grain (1.5–3 mm).

7. Facades are painted with acrylic paints.

The estimated cost of works performed during the repair of facades is around 6,800 RUB/m2 for sealing of joints and thermal insulation of external walls with plaster.

The authors also developed an option of 137 series large-panel building facade thermal insulation in the form of a suspended facade system, which has the following advantages:

- Ventilated facades have excellent thermal and sound insulation properties, and perform decorative and protective functions. Various surfacing materials allow creating a wide range of design solutions for building appearance. Ventilated facades protect walls from weather, temperature and other impacts.
- Facades can be installed both on old buildings and on buildings under construction. Preparation of walls for installation of facades is not required. In this case, the prime cost of installation works is significantly reduced.
- Due to generality and multi-functionality, ventilated facades have no restrictions with regard to installation; they can be installed on buildings with wooden, brick, stone or concrete walls. Structures are mounted both on multi-storied modern houses and on outdoor concession stands.

A suspended ventilated facade is a technology of facade implementation, a system, which consists of surfacing materials that are attached on a galvanized steel frame, a stainless steel frame or an aluminum frame to the load-bearing wall layer or floor structures. Air circulates freely through the gap between the surfacing and the wall, removing condensate and moisture from structures.

An effective heat insulator is attached to the wall by means of disk-shape dowels or flexible connections for additional insulation of building walls. Extruded polystyrene foam PENOPLEX is often recommended to be applied on the basement part of the building. It is not pervious and does not absorb the moisture. The space between the insulation and the facade of the building should not be less than 40 mm (from 20 to 50 mm according to different sources; the greater values of the gap are accepted in Russia than in the USA and Europe).

We suggest a wide range of finishing materials for surfacing of suspended ventilated facade systems of frameless large-panel buildings: fiber-cement panels, metal siding, aluminum composite panels, ceramic granite, vinyl siding, glass panels, and fiber cement siding.

Various design solutions of the ventilated system of thermal insulation and finishing of external walls of frameless large-panel residential buildings are developed and recommended.

4. Conclusions

As a result of the carried out studies, upon execution of the set tasks, the authors developed:

- the technology of thermal and hydraulic insulation of inter-panel joints of external walls of standard series 137 large-panel buildings during their repair with the use of modern materials of joint filling;

- options of facade insulation on the basis of the analysis of modern facade systems, taking into account import substitution of materials and structural details;
- a technological sequence of facade thermal insulation arrangement in a large-panel building for various facade systems.

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DRIVERS OF THE GROWTH OF THE RUSSIAN CONSTRUCTION SECTOR

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1 Introduction

Implementation of the Russia's development strategy requires searching new approaches to the regulation of economy leading sectors' activities. Construction is the priority area of the economy, which directly affects the quality of life of the population. The dynamics of construction is one of the key indicators, by which it is possible to judge the economic and social development of the country. Along with this, reforms carried out in recent decades in Russia have revealed problems in all areas of the economy, including construction. First of all, they include the low specific weight of qualified blue-collar personnel; non-development of the competitive environment, due to the relatively low proportion of small and medium-sized enterprises in the structure of the construction sector; unformed legislative base governing the construction sector; low effectiveness of investments; non-compliance of construction norms and rules with modern requirements for construction facilities; lack of a well-developed mechanism for coordinating inter-firm cooperation. The latter circumstance has a dominant character, since it directly affects the occurrence of other problems facing the construction sector.

Problems of the organization of various systems of inter-firm cooperation in Russia are not new and underwent a number of fundamental changes, the essence of which reduces to the evolution of industry management forms. However, realities of the market economy require a fundamentally different involvement of the state in the regulation of industry sectors and nature of interaction between commercial entities. The role of other institutions that affect functioning of the investment-construction sector significantly increases: firstly, of research and educational institutions as sources of the knowledge production; secondly, of professional organizations as institutions for cooperation and coordination of activities as well as promotion of cluster initiatives. All these circumstances bring to the fore the cluster policy that can significantly influence both the competitiveness of the sectors themselves and the competitiveness of the national economy.

2 Methods

The goal of this work lies in scientific justification of the directions to improve the effectiveness of the construction sector on the basis of integrating of its participants.

The goal is achieved by solving the following tasks:

- to reveal peculiarities of the formation and functioning of industrial sectors in the Russian economy;
- to identify the role of the cluster policy in the development of the construction sector under conditions of the market economy;
- to determine institutional characteristics of the regional construction clusters' functioning;
- to propose tools for cluster policy implementation in the development of the construction sector.

Scientific novelty lies in the development of a cluster model of "state–construction sector–higher educational institution" interaction, wherein the institutional (state (regional, municipal) bodies, public and professional communities, research and educational centers, small, medium and large organizations of the industry, infrastructure of the cluster) and organizational (regulatory support, methodological support, organizational support, technical support, informational support) subsystems of the investment-construction cluster' functioning are revealed. Introduction of this model contributes to the growth of employment and entrepreneurial initiative, raising of living standards of the population, integrated use of domestic resources, development of domestic markets, infrastructure, competitiveness of construction companies, development of associated industries and territories.

The work is based on the study of features of integration processes in the planned, market and transformation economies described in the works of Russian and foreign scientists. The nature, reasons and consequences of the integration of market participants are investigated in the works of Andersson T. [1], Laffer A. [2], Porter M. [3, 4, 5], Oliver E. Williamson [6], Enright A. [7] and Arrow K. J. [8] from the standpoint of various schools and trends. In the Russian science, the works of Asaul A. N. [9, 10], Dubrovskiy V. Zh. [11], Markov L. S. [12], Tokunova G. F. [13, 14] and others are dedicated to this issue.

Besides, certain aspects of the organization and functioning of clusters in the construction sector are considered in the works of Volkova N. N. [15], Lisichkin V. A. [16], Loydesdorff L. and Etzkowitz H. [17, 18], Matkin G. [19], Savelieva I. P. [20], Suvorova A. P. [21] and others.

3 Results

The construction sector as a concept in the Russian science was formed only in the 1980s and comprised specific conditions of its functioning not within the framework of a narrow industry-specific approach, but as a sphere with a high multiplicative effect due to the

inter-industry nature of the interaction of construction sector members in the territory of a certain region.

Transition to market relations required direct interaction of construction and investment activities in the form of capital investments. Stepped-up construction of real estate for production and non-production purposes was a result of this interaction. Besides, the concept of "investment-construction sector" appeared in this regard.

Organizational changes in the economy (increasing competition, increasing role of small and medium-sized enterprises determining flexibility of production communications, increasing role of innovations, convergence of interests of large corporations, greater value of information as a factor in production, development of new management methods and many others) required a review of the organization of conventional structures of interaction of business entities, which acquired more flexible and dynamic nature in modern conditions. As a result, sectors are replaced by inter-industry clusters. The production structure of a cluster is characterized by tighter engineering communications than the traditional sectoral structure, which determines its competitiveness.

The main subjects of the construction cluster are the following: small, medium and large construction companies, contracting agencies and supplying organizations, market infrastructure companies (credit, insurance, leasing, logistics companies, commercial agents, real estate organizations), research and educational organizations, professional and public organizations (unions, associations, self-regulating organizations), business associations, chambers of commerce and industry, cluster infrastructure (cluster council, small and medium enterprise support funds, technology parks, business incubators, etc.).

The complex inter-industry nature of interaction within the construction cluster requires a review of approaches to the development regulation of both the cluster and the construction sector as a whole. This is also driven by a changing role of the state, which becomes an equal participant in economic transformations within the cluster, when advantages of private enterprises are combined with the capabilities of the public sector.

Considering the construction cluster as a system, we can point out the following main characteristics: first of all, it has a complex structure and includes a number of subsystems interacting in a certain way; secondly, being open from one side, it is detached from the external environment on the other side, constantly interacting with it; thirdly, it is partially controlled, i.e. it combines elements of centralization and elements of self-organization and self-regulation under independence of business structures; fourthly, it is spatially localized, therefore, its development is inseparably connected with the development of the surrounding space.

The following properties are characteristic for the regional investment-construction cluster as a system:

1) properties related to the system structure:
- emergence: it occurs when elements and links are combined, when system quality and properties are not reduced to its separate elements (non-additivity);
- hierarchy: it is a way of arrangement and organization of complex systems, that ensures its stability in relation to external influences;
- divisibility: it is a possibility to divide into subsystems that are in a certain interaction with each other;
- integrity: it is a cohesion of its main elements;
- probability: the number of elements and links between them cannot be fixed accurately in advance and for a long time;
- multicriteriality: it implies multiple goals of functioning;

2) properties related to the system functioning:
- integratedness: it is incorporation of elements, deepening of their interaction and development of connections between them;
- adaptability: it is an ability of the system and its separate elements to adapt to the changing conditions of the external environment;
- equilibrium: it is a state of the system, characterized by presence of balance and stability;
- dynamism: it is a capacity of the system for continuous changing and development;
- feedback: it is information about the state of a control object and its response to the control action;
- management: it implies an impact on the system in order to obtain the desired result.

While functioning, clusters are guided by a number of principles that can be divided into general management principles, special principles for managing sectoral clusters, and principles governing members that make up the cluster (Table 1).

Table 1. Principles of functioning of the regional investment-construction cluster

No.	Principle	Description
1. General principles		
1.1	Scientific approach	Control of the cluster activities should be based on the system analysis and other scientifically sound methods
1.2	Combination of sectoral and territorial interests	Serving the interests of investment-construction entities with simultaneous consideration of interests of the region inhabitants
1.3	Consistency and integrity	Application of principles of the theory of systems in combination with comprehensive coverage of both regulated and regulating subsystems
1.4	Accounting for historical, geographical, economic, sociocultural and other parameters of the region development	Use of all control methods is inseparable from characteristics of the region
1.5	Effective combination of positive synergy in activities of organizations located in the region, and competition between them	Increase in the efficiency of activities as a result of joining, integrating and merging of separate parts into a single system due to emergence and simultaneous contest between its elements
2. Special principles		
2.1	Large-scale implementation of innovations into production	Use of innovations in engineering, technology, work organization and management, based on application of scientific achievements and best practices
2.2	Free distribution of information within the cluster	Conditioning for free distribution of information, as well as provision of management systems with means for search, accumulation, storage, processing and transmission of information using computer and other equipment
2.3	Access of all participants to tangible and intangible resources of the cluster, as well as to the results of the cluster activity (equal rights of all cluster members)	Ensuring equal rights and obligations of all cluster members, as well as state guarantees and guarantees of local self-government bodies
2.4	Maximum use of local resources for the needs of the cluster	The principle is based on the idea of local development of the region
2.5	Development of various integration types of interaction within the cluster	Coexistence of soft and strong forms of integration of cluster members
2.6	Development of inter-industry, inter-regional and international connections	A cluster as an open system can exist only in conjunction with the external environment
2.7	Development of small and medium enterprises	Implementation of the cluster policy implies multiplicity of investment-construction entities
3. Principles of activity control for cluster members		
3.1	Economic, social, ecological efficiency of activities within the cluster	When calculating efficiency, it is necessary to follow the overall efficiency of activities of cluster members, including economic, social and environmental efficiency
3.2	High performance as a result of interaction with other cluster members	Interaction of cluster members in the value chain leads to the concentration of efforts that reduce the threats of the external environment impact and increase the efficiency of their activities (synergy)
3.3	Maintaining a high quality of products (works, services) while reducing their cost price	The interaction effect leads to emergence of embeddedness, displacing motivation from the immediate economic gain towards enriching ties with trust relationships; besides, sustainable links lead to savings in the number of costs, primarily transaction costs
3.4	Maximum satisfaction of effective demand	The cluster structure allows flexible responding to changes in effective demand
3.5	Flexible strategy of behavior within the cluster	The flexibility of the organizational structure generates a flexible strategy for behavior of cluster members in the economic area

General principles come from the theory of management and define general requirements for economic systems; special principles and principles for regulating activities of the constituent members of the cluster determine the nature of links within the cluster, requirements for the cluster management structure, the nature of management decision-making regarding the functioning and development of the cluster.

Effective functioning of the construction cluster is possible under a balanced development of its main subsystems, including the following:

1. Institutional subsystem: state (regional, municipal) bodies, public and professional communities, research and educational centers, small, medium and large organizations of the industry, infrastructure of the cluster.

2. Organizational subsystem: regulatory support, methodological support, organizational support, technical support, informational support.

State, regional and municipal authorities. A regulating subsystem defining the framework conditions for the functioning of the cluster includes various departments, committees and bodies that represent interests of both the state and regional municipal authorities. They are represented by the following bodies:

1. Departments of the Ministry of Regional Development of the Russian Federation.
2. Branch executive bodies of the state.
3. Branch structures of federal authorities.

The principal mission implemented by the system of the state regulation of the construction cluster is solving of issues of the reproduction of economic, social and natural resources in the region.

Besides, the functioning of the construction cluster depends on activity of various engineering departments, institutions and organizations that determine conditions for the operation of construction objects.

University academic organizations act as the main member which is a carrier of knowledge and innovations in the cluster.

This is due to the fact that universities, besides their basic function of the training of specialists for the needs of the construction sector, become business entities within the framework of entrepreneurial structures, such as resource centers, small innovative enterprises, and technological platforms.

The cluster council is the main coordinating member in the cluster. The head of the cluster council is a manager, whose duties include coordinating activities in the following areas:

- determination of the feasibility of the cluster in the region;
- determination of industry affiliation and cluster type, initial analysis of the cluster environment;
- cluster team building;
- determination of desired results and drawing up of a schedule plan;
- drawing up of a plan of priority actions and its implementation;
- defining of the cluster development, updating of the cluster development plan;
- monitoring of the cluster development;
- alignment of technological chains between all clusters of the region.

Information support. Such a number of regulatory bodies and entities carrying out scientific and technical activities requires the coordination of their actions, which is impossible without an effective system of the technical and information support of the communication process in decision-making. The information base should include the following sources:

- databases of studies conducted (analytical, expert) and forecasts for the development of the investment-construction sector, national and world economies;
- regulatory sources, concepts of the national economy development and long-term development plans of the country;
- analytical studies, forecasts and plans of industry and professional associations and self-regulatory organizations.

Information sources are represented by statistical data, analysis of patent information, results of expert panels, data obtained in the result of application of scientometric and bibliometric methods in solving information retrieval problems, i.e. quantitative analysis of documentary flows (statistical method, counting of publications, quotation indices).

Cluster infrastructure. Effective functioning of the cluster implies an integrated infrastructure consisting of the following:

- market infrastructure companies (credit, insurance, leasing, logistics companies, commercial agents, real estate organizations);
- cluster infrastructure (small and medium enterprise support funds, technology parks, business incubators, credit promotion funds, investment funds, etc.).

A number of approaches to cluster model engineering have been developed; they are as follows:

The first one is "top-down", which implies initial formation of bodies of advisory coordination and monitoring, defining of strategies for the cluster as a whole and its resource support.

The second one is "bottom-up", providing for the development of programs integrating the cluster members.

The third one is mixed and characterized by simultaneous development of the first two indicated approaches.

In our opinion, only the latter approach meets the requirements of creating a viable construction cluster to the full.

The efficiency of the functioning of the construction sector can be assessed on the basis of integration of its participants at the following levels: cluster control system, development of the construction sector,

development of inter-industry (inter-regional, international) connections, social and economic development of territories.

Assessment should be carried out in several successive stages:

Stage 1 — preparation: defining targets and tasks of the assessment of the cluster state; statement of the problem that would be revealed during the assessment; formulation of a hypothesis that would find confirmation during the assessment, selection of criteria, quantitative and qualitative assessment parameters, development of the assessment methodology.

Stage 2 — assessment: gathering information on the socio-economic status of cluster members and the level of the regional development in dynamics; analysis of internal and external factors of the social and economic development of cluster members and the level of the region development; interpretation of the data obtained during the assessment.

Stage 3 — drawing conclusions and presentation of the assessment results: summarizing of the results obtained during the assessment; drawing conclusions and recommendations for the improvement of the efficiency of the cluster policies; preparation of a final report.

The result of the assessment is drawing conclusions and presentation of recommendations for the further socio-economic development of clusters.

4 Conclusions

Implementation of the cluster policy would benefit all the following cluster members:

regional authorities, acting as participants in the cluster policy would receive access to reliable information on activity of economic entities and the state of the regional market, which is a prerequisite for the development of sound strategic plans and development programs for territories, increase in domestic regional product, raising of the reputation of the territory at the national and international levels;

construction and related organizations: comfortable conditions for business; additional opportunities for the development of small and medium enterprises; sources for minimizing costs, including transaction costs; opportunities for improving the quality of construction products; access to necessary resources, including qualified personnel;

educational institutions would be more actively involved in the training and development of personnel, which would increase their competitiveness in the market of educational services;

research and development institutions would fulfill their capabilities in the field of development and implementation of results of their activity, which, in its turn, would promote the development of innovations in construction;

population: raising of standards of living through additional employment opportunities, increase in housing affordability, obtaining of quality goods and services, etc.

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ON ONE MIXED BOUNDARY PROBLEM FOR AN ELASTIC EXPONENTIALLY NONHOMOGENEOUS BY DEPHTH LAYER AT ANTIPLANE DEFORMATION

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Abstract

The mixed boundary problem for elastic layer, shear modulus of which varies exponentially in depth, is considered in this paper in the formulation of the mathematical theory of elasticity. Lower face of layer is rigidly clamped. Component of the elastic displacements is given on its upper face, on systems of intervals. On the rest of upper face of layer tangential stresses are zero. It is assumed that the layer is at anti-plane deformation of lateral displacement in the direction of one of the coordinate axes. We discuss two formulations of the problem, when on the system of co-linear intervals of the upper bound component of displacement is given directly and there is need to find a law of distribution of tangential stresses providing preassigned displacement mode. In the second formulation considered problem of transmission of tangential strains from the co-linear system of stringers to the elastic layer. The known model of Melan for the stringers is applied at the same time. Solutions of the assigned tasks are reduced to the solution of singular integral equations in both of formulations, which are solved by known numerical analytical method. The explicit formulas for contact stresses and coefficients of their intensities are obtained and the numerical analysis in particular cases is presented.

1. Introduction

We consider the problem of contact transmission of tangential forces from the system of any finite number of collinear stringers to the elastic inhomogeneous layer at anti-plane deformation in two formulations. The problems of described type are concerning to the class of mixed boundary problems of the mathematical theory of elasticity and classical contact problems of the theory of elasticity. They are connected with important problems of engineering practice about transmission of external loads from thin –walled elements in the form of stringers to massive deformable bodies. The need for research of this class of problems is dictated by growing demands of structural mechanics, mechanical engineering, aircraft building, measuring mechanics, mechanics of composite materials and other fields of applied mechanics. In view of its relevance, they became the subject of research by many authors and many works have been devoted to them. In this article, summarizing the results of the paper [4,5], within the framework of the physical model of Melan for the stringer at the anti-plane deformation [6] solution of the considered problem in two statements is reduced to the two singular integral equations with very close analytical structures. These SIE are solved by known numerical-analytical method of solution of SIE [7-9], based on quadrature formulas of Gauss for Chebishev nudes. Main characteristics of the problem, i.e. tangential contact stresses under the stringers, coefficients of their concentrations at the endpoints of the stringers are presented by explicit formulas, particular cases are considered.

2. The formulation of the problem and derivation of the basic equations

Let an elastic layer $\Omega = \{-\infty < x, z < \infty;\ -H \le y \le 0\}$ referred to the right-hand rectangular coordinate system $Oxyz$ has a height H and a shear modulus G, which is varying in depth according to exponential low

$$G = G_0 e^{\alpha y} \ (\alpha = const, -H \le y \le 0).$$

Further, let the lower face $y = -H$ of the layer be rigidly pinched, but on the upper face $y = 0$ along the system of segments $L = \bigcup_{k=1}^{n} L_k \ (L_k = [a_k \le x \le b_k])$ be strengthened by the collinear system of stringers in the form of elastic bands

$$\omega_k = \{a_k \le x \le b_k;\ 0 \le y \le h_k; -\infty < z < \infty\} \ (\mathrm{k} = \overline{1,\mathrm{n}}),$$

which have G_k shear modules and h_k heights (fig. 1).

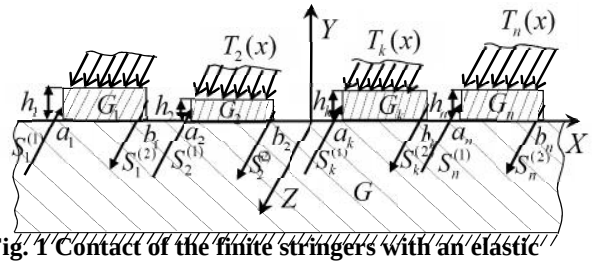

Fig. 1 Contact of the finite stringers with an elastic exponentially inhomogeneous layer

Assume that on the upper face $y = h_k$ of the strip ω_k, in the direction of the axis Oz, are acting

uniformly distributed tangential forces of intensity $T_k(x)$, i.e.

$$\tau_{yz}\Big|_{y=h_k} = T_k(x) \quad (a_k < x < b_k;\ \mathrm{k} = \overline{1,\mathrm{n}}),$$

where τ_{yz} is component of tangential stresses. Furthermore, assume that on the boundary sections $x = a_k$ and $x = b_k$ of the strip ω_k $(k = \overline{1,n})$ in the direction of the axis Oz uniformly distributed along the axis Oz tangential concentrated forces $S_k^{(1)}$ and $S_k^{(2)}$ $(k = \overline{1,n})$ are acting, respectively.

Under the action of those power factors the system of stringers-layer is at the anti-plane deformation in the direction of axis Oz by the baseband $\Omega_0 = \{-\infty < x < \infty;\ -H \le y \le 0\}$

In the first formulation of the problem it is required to determine tangential contact stresses $\tau(x)$ under the stringers, when the stringers are at the action of the given strain, i.e. the functions

$$\tau_{yz}\Big|_{y=0} = \tau_k(x) \quad (a_k < x < b_k;\ \mathrm{k} = \overline{1,\mathrm{n}}),$$

and the coefficients of concentration of those stresses at the end-points of the stringers.

We will introduce equations of considered problem. For this purpose, we use expressions for the movement of the boundary points of the strip Ω_0, at $y = 0$. According to the results obtained in paper [4], in this case we have

$$w(x,0) = \frac{2e^{-\alpha H}}{\pi G_0}\sum_{k=1}^{n}\int_{a_k}^{b_k} L(x-s)\tau_k(s)ds \quad (-\infty < x < \infty) \quad (1)$$

$$L(x) = \int_0^{\infty} \frac{th(\mu H/2)\cos(\lambda x)}{\mu - \alpha th(\mu H/2)} d\lambda; \quad \mu = \sqrt{\alpha^2 + 4\lambda^2},$$

where $w(x,0)$ are displacements of the boundary points of the strip $\Omega_0\,(\mathrm{y} = 0)$ in the direction of the axis Oz.

From the formula (1), after the differentiation with respect to x, we obtain

$$\frac{dw(x,0)}{dx} = \frac{2e^{-\alpha H}}{\pi G_0}\sum_{k=1}^{n}\int_{a_k}^{b_k} M(s-x)\tau_k(s)ds \quad (-\infty < x < \infty) \quad (2)$$

$$M(x) = \int_0^{\infty} \frac{th(\mu H/2)\lambda \sin(\lambda x)}{\mu - \alpha th(\mu H/2)} d\lambda.$$

Next, we will write differential equation of deformation of the k-th stringer by the model of Melan at the anti-plane deformation [6]

$$h_k G_k \frac{d^2 w_k}{dx^2} = \tau_k(x) - T_k(x) \quad \left(a_k < x < b_k; k = \overline{1,n}\right), \quad (3)$$

where $w_k = w_k(x)$ is the component of displacements of the points of the k-th stringer in the direction of the axis Oz. The condition of equilibrium of the stringer has the form

$$\int_{a_k}^{b_k} \tau_k(x)dx = S_k^{(2)} - S_k^{(1)} + \int_{a_k}^{b_k} T_k(x)dx \quad (k = \overline{1,n}). \quad (4)$$

Integration of (3), gives us

$$h_k G_k \frac{dw_k}{dx} = \frac{1}{2}\left(S_k^{(1)} + S_k^{(2)}\right) + \frac{1}{2}\int_{a_k}^{b_k} \operatorname{sign}(x-s) \times \times \left[\tau_k(s) - T_k(s)\right]ds \quad \left(a_k \le x \le b_k;\ k = \overline{1,n}\right) \quad 5)$$

Let us substitute expressions for $\frac{dw}{dx}$ and $\frac{dw_k}{dx}$, from (2) and (5) respectively, into the following condition of contact of the elastic strip Ω_0 and the k-th stringer

$$w(x,0) = w_k(x) \quad \text{or}$$

$$\frac{dw(x,0)}{dx} = \frac{dw_k(x)}{dx} \quad \left(a_k \le x \le b_k;\ k = \overline{1,n}\right)$$

After the simple transformations relative to the unknown tangential contact stresses $\tau_k(x)$, we will come to the following SIE:

$$\frac{1}{\pi G_0}\sum_{k=1}^{n}\int_{a_k}^{b_k}\left[\frac{1}{s-x} + R(s-x)\right]\tau_k(s)ds + \frac{e^{\alpha H}}{2h_k G_k}\int_{a_k}^{b_k}\operatorname{sign}(s-x)\times \times \tau_k(s)ds = \frac{e^{\alpha H}}{2h_k G_k}\left[S_k^{(1)} + S_k^{(2)} + \int_{a_k}^{b_k}\operatorname{sign}(s-x)T_k(s)ds\right] \quad (a_k < x < b_k;\ \mathrm{k} = \overline{1,\mathrm{n}}) \quad (6)$$

$$R(x) = \int_0^{\infty} \frac{(\alpha + 2\lambda)th(\mu H/2) - \mu}{\mu - \alpha th(\mu H/2)} \sin(\lambda x)d\lambda, \text{ where the}$$

first integral at $s = x$ is understood in the sense of the Cauchy principal value. Solution of the SIE (6) should satisfy the condition (4).

In the second formulation of the problem we will assume, that in advance the mode of transmissions of stringers is given, that means in differential equations (3) are given the functions

$$w_k = w_k(x) = f_k(x) \quad (a_k \le x \le b_k;\ k = \overline{1,n}).$$

That is required to find lows of distributions of tangential contact stresses $\tau_k(x)$ $(k = \overline{1,n})$. After that from equation (3) to determine functions $T_k(x)$ $(k = \overline{1,n})$, which are the lows of loading of stringers by the tangential distributed forces.

The second staging of the broblem leads to the mixed boundary problem for the elastic strip Ω_0, when its lower face $y = -H$ is rigidly pinched, and on the upper face $y = 0$ on complex of segments L is given component of transmissions

$$w(x,0)=f(x)=f_k(x)\ \ (a_k\le x\le b_k;\ k=\overline{1,n}),\qquad (7)$$

and on the rest of the upper face of the strip Ω_0 tangential forces are equal to 0. For completeness of the problem, on each of the segments L_k of the system L must be given resultant T_k of the of the unknown tangential stresses $\tau_k(x)$:

$$\int_{a_k}^{b_k}\tau_k(s)ds=T_k\quad (k=\overline{1,n}).\ (8)$$

In the second staging of the problem we will derive basic equations of the considered problem. For this purpose we will again use condition of contact

$$w(x,0)=w_k(x)=f(x)=f_k(x)\ \ (a_k\le x\le b_k;\ k=\overline{1,n}).$$

After that, by the help of (1) relative to the unknown contact stresses $\tau_k(x)$, we will come to the following integral equation of Fredholm of the first kind

$$\frac{1}{\pi}\sum_{k=1}^{n}\int_{a_k}^{b_k}L(x-s)\tau_k(s)ds=\frac{G_0e^{\alpha H}}{2}f(x)\ (a_k\le x\le b_k;\ k=\overline{1,n}).$$

After differentiation of both of sides of that equation by x and highlighting singular part of the core, it will come to the following determining SIE of the problem from the second formulation

$$\frac{1}{\pi}\sum_{k=1}^{n}\int_{a_k}^{b_k}\left[\frac{1}{s-x}+R(s-x)\right]\tau_k(s)ds=G_0e^{\alpha H}f'(x)\qquad (9)$$

$$(a_k\le x\le b_k;\ k=\overline{1,n}).$$

Solution of the SIE (9) should satisfy the conditions (8). We note, that from the core $R(s-x)$ that is possible to isolate discontinuous core $\operatorname{sign}(s-x)$, after that SIE (6) and (8) actually will have the same analytical structures. Therefore, further we can confine oneself to one of them, for example SIE (9).

3. The solution of the SIE (9)-(8) by the numerical analytical method

At first in SIE (9) we will go to dimensionless quantities

$$\xi=x/a,\ \eta=s/a;\ \tau_k^{(0)}(\xi)=\tau_k(a\xi)/G_0;\ \ f_0(\xi)=e^{\alpha_0H_0}f'(a\xi);$$

$$\alpha_k=a_k/a,\ \beta_k=b_k/a\ \ (k=\overline{1,n});\ H_0=H/a;\ \alpha_0=\alpha a;$$

$$L_0=\bigcup_{k=1}^{n}[\alpha_k,\beta_k];\ \omega=a\lambda;\ \mu_0=\sqrt{\alpha_0^2+4\omega^2};$$

where a-some characteristic linear parameter, for example, $a=a_1$, if $a_1\ne0$. As a result SIE is coming to the folloing form

$$\frac{1}{\pi}\sum_{k=1}^{n}\int_{\alpha_k}^{\beta_k}\left[\frac{1}{\eta-\xi}+N_0(\eta-\xi)\right]\tau_k^{(0)}(\eta)d\eta=f_0(\xi)$$

$$N_0(\xi)=\int_0^{\infty}\frac{(\alpha_0+2\omega)\operatorname{th}(\mu_0H_0/2)-\mu_0}{\mu_0-\alpha_0\operatorname{th}(\mu_0H_0/2)}\sin(\omega\xi)\,\mathrm{d}\omega.\ (10)$$

$$(\alpha_k<\xi<\beta_k,k=\overline{1,n})$$

In this dimensionless quantities conditions (8) will be transformed to the following conditions

$$\int_{\alpha_k}^{\beta_k}\tau_k^{(0)}(\eta)d\eta=T_k^{(0)}\ \ (T_k^{(0)}=T_k/aG_0;\ k=\overline{1,n}).\ (11)$$

After this integrand of the nuclear function $N_0(\xi)$ we will represent in the form

$$h_0(\omega)=\frac{(\alpha_0+2\omega)\operatorname{th}(\mu_0H_0/2)-\mu_0}{\mu_0-\alpha_0\operatorname{th}(\mu_0H_0/2)}=$$

$$=\frac{(\alpha_0+2\omega)\left[\operatorname{th}(\mu_0H_0/2)-1\right]+\alpha_0+2\omega-\mu_0}{\mu_0-\alpha_0\operatorname{th}(\mu_0H_0/2)}$$

That is easy to show that $h_0(\omega)\to\frac{\alpha_0}{2\omega}$, when $\omega\to\infty$.

So, we can write

$$N_0(\xi)=\int_0^{\infty}\left[h_0(\omega)-\frac{\alpha_0}{2\omega}\right]\sin(\omega\xi)\,\mathrm{d}\omega+\frac{\alpha_0}{2}\int_0^{\infty}\frac{\sin(\omega\xi)}{\omega}d\omega,$$

where the second integral is discontinuous integral of Dirichley. Attending to its value, we get

$$N_0(\xi)=\frac{\pi\alpha_0}{4}\operatorname{sign}\xi+\int_0^{\infty}G(\omega)\sin(\omega\xi)\,\mathrm{d}\omega\quad(-\infty<\xi<\infty)\qquad(12)$$

$$G(\omega)=\frac{(\alpha_0+2\omega)\left[2\omega\operatorname{th}(\mu_0H_0/2)-\mu_0\right]+\alpha_0^2\operatorname{th}(\mu_0H_0/2)}{2\omega\left[\mu_0-\alpha_0\operatorname{th}(\mu_0H_0/2)\right]}.$$

The following asymptotic formulas hold

$$G(\omega)\to-\frac{\alpha_0}{2\omega}\ ,\ \text{at}\ \omega\to0;\ G(\omega)\to\frac{\alpha_0^2}{8\omega^2},\ \text{at}\ \omega\to\infty,$$

which provides rather fast convergenceof the integral (12).

Taking into account representation (12), SIE (10) we can represent in the form

$$\frac{1}{\pi}\sum_{k=1}^{n}\int_{\alpha_k}^{\beta_k}\left[\frac{1}{\eta-\xi}+\frac{\pi\alpha_0}{4}\operatorname{sign}(\eta-\xi)+M_0(\eta-\xi)\right]\tau_k^{(0)}(\eta)d\eta=$$

$$=f_0(\xi);\ M_0(\xi)=\int_0^{\infty}G(\omega)\sin(\omega\xi)\,\mathrm{d}\omega.\ \ \alpha_k<\xi<\beta_k,k=\overline{1,n})\qquad(13)$$

Now, each of the intervals (α_k,β_k) we will transform into $(-1,1)$, believing

$$\xi=\frac{\beta_k-\alpha_k}{2}t+\frac{\beta_k+\alpha_k}{2},\ \eta=\frac{\beta_k-\alpha_k}{2}u+\frac{\beta_k+\alpha_k}{2}$$

$$(-1<t,u<1;\ k=\overline{1,n})$$

As a result, SIE (13) will transform into following SIE

$$\frac{1}{\pi}\int_{-1}^{1}\left[\frac{1}{u-t}+\frac{\pi\alpha_0}{4}\operatorname{sign}(u-t)+\mathrm{Q}_{kk}(t,u)\right]\overline{\tau}_m(u)du+ \\ +\frac{1}{\pi}\sum_{\substack{m=1\\(m\neq k)}}^{n}\int_{-1}^{1}L_{km}(t,u)\overline{\tau}_m(u)du=g_k(t)\quad(k=\overline{1,n}) \tag{14}$$

$$L_{km}(t,u)=\frac{1}{u-\gamma_{km}t+\delta_{km}}+\frac{\pi\alpha_0}{4}\operatorname{sign}(u-\gamma_{km}t+\delta_{km})+\mathrm{Q}_{km}(t,u)$$

$$(k\neq m;-1<t,u<1);\ g_k(t)=f_0\left(\frac{\beta_k-\alpha_k}{2}t+\frac{\beta_k+\alpha_k}{2}\right);$$

$$\gamma_{km}=\frac{\beta_k-\alpha_k}{\beta_m-\alpha_m};\ \delta_{km}=\frac{\beta_m+\alpha_m-\beta_k-\alpha_k}{\beta_m-\alpha_m};$$

$$Q_{km}(t,u)=M_0\left(\frac{\beta_m-\alpha_m}{2}(u-\gamma_{km}t+\delta_{km})\right);$$

$$\overline{\tau}_m(u)=\tau_m^{(0)}\left(\frac{\beta_m-\alpha_m}{2}u+\frac{\beta_m+\alpha_m}{2}\right).$$

Conditions (8) will transform into conditions

$$\int_{-1}^{1}\overline{\tau}_k(u)du=S_k^{(0)}\quad\left(k=\overline{1,n};\ S_k^{(0)}=2T_k^{(0)}/(\beta_k-\alpha_k)\right). \tag{15}$$

Further following to the procedure of the numerical-analytical method of solution of SIE mentioned above, we put

$$\overline{\tau}_k(t)=\Omega_k(t)\Big/\sqrt{1-t^2}\quad(-1,1),$$

where $\Omega_k(t)$ are functions of the Holder class on the segment $[-1,1]$. As a result, solution of the SIE (14)-(15) is reducing to the solution of the following system of system of linear algebraic equations (SLAE).

$$\sum_{p=1}^{N}R_{rp}^{(km)}\mathrm{X}_p^{(m)}=c_r^{(k)}\quad(r=\overline{1,N};k=\overline{1,n};m=\overline{1,n}); \tag{16}$$

$$R_{rp}^{(km)}=\begin{cases}M_{rp}^{(km)}\ \left(p=\overline{1,N};\ r=\overline{1,N-1};\ k,m=\overline{1,\mathrm{n}}\right);\\ \dfrac{\pi}{N}\qquad\left(r=N;\ p=\overline{1,N};\ k,m=\overline{1,\mathrm{n}}\right),\end{cases}$$

$$c_r^{(k)}=\begin{cases}a_r^{(k)}\quad\left(r=\overline{1,N-1};\ \ k=\overline{1,\mathrm{n}}\right);\\ S_k^{(0)}\qquad\left(r=N;\ \ k=\overline{1,\mathrm{n}}\right);\end{cases}$$

$$M_{rp}^{(km)}=\begin{cases}\dfrac{1}{N}\left[\dfrac{1}{u_p-u_r}+\dfrac{\pi\alpha_0}{4}\operatorname{sign}(u_p-u_r)+\mathrm{Q}_{kk}(\mathrm{t}_r,u_p)\right]\left(m=k\right);\\ \dfrac{1}{N}\sum_{m=1}^{N}L_{km}(u_r,u_p)\qquad\left(m\neq k\right);\end{cases}$$

$$a_r^{(k)}=g_k(t_r)\ \ (\mathrm{r}=\overline{1,\mathrm{N}-1});\ \ \mathrm{X}_p^{(m)}=\Omega_m(u_p);$$

$$u_p=\cos\frac{(2p-1)\pi}{2N}\ \left(p=\overline{1,N}\right);\quad t_r=\cos\frac{\pi r}{N}\ \ \left(r=\overline{1,N-1}\right).$$

Here N is any fixed natural number, u_p and t_r are Chebyshev nodes, i.e. roots of Chebyshev polynomials of the first kind $T_M(u)$ and of the second kind $U_{M-1}(t)$, respectively.

Particular cases are considered, when 1) $n=1, a_1=-a; b_1=b$ и 2) $n=2, a_1=-a, b_1=-b; a_2=b, b_2=a$, $(b<a)$. In this cases the system of SIE (14)-(15) and the system of SLAE (16) are simplified

Coefficients of concentrations of tangential contact stresses (KS) are calculated by the formulas

$$K_{a_k}=\lim_{x\to a_k+0}\sqrt{x-a_k}\,\tau_k(x),\ K_{b_k}=\lim_{x\to b_k-0}\sqrt{b_k-x}\,\tau_k(x)(k=\overline{1,n}).$$

In this formulas at first, as above, we should go to variable ξ and to the parameters α_k,β_k, than to the variable t $(t\in(-1,1))$. As a final result, (KS) can be expressed by the solutions of the system SLAE (16).

In the particular cases there were done numerical analysis of the problem and depending on characteristic physical and geometrical parameters regularities of tangential contact stresses and (KS) were studied.

4. Results

In the article the following main results are obtained:

a) In two closely related formulations of the problem of contact-interaction of a system of collinear stringers with an exponentially inhomogeneous elastic layer the determining singular integral equation is derived.

b) An exploration of the cores of the singular integral equations is carried out, their singular parts are highlighted.

c) Within the framework of the known numerical-analytical method an effective method of solution of the determining SIE is offered.

5. Conclusion

In article

a) In two closely related formulations, which have theoretical and practical interest, there was examined problem of contact interaction of the system of any finite number of collinear stringers with nonhomogeneous elastic layer at anti-plane deformation, shear module of which is varying in depth according to exponential low;

b) The patterns of changes of main characteristics of the problem, i.e. tangential contact stresses and their KS, are identified.

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PROSPECTS OF DEVELOPMENT OF NANOTECHNOLOGIES IN PRODUCTION OF CELLULAR GLASS BASED ON VOLCANIC GLASS WOOL AND OTHER SILICATE ROCKS OF THE REPUBLIC OF ARMENIA

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Abstract

The purpose of this study is to show prospects of production of energy-saving materials using the volcanogenic silicate and aluminosilicate rocks in Armenia. Benefits of the developed cellular glass produced through nanotechnology and designed for different applications has been shown. Considering the wide scope of energy-saving materials possible application organization of their production has become an issue of vital importance.

1. Introduction

One of the main natural resources of Armenia is rich and varied base of volcanogenic perlite, pumice-stone, tufa, slag and other silicate and aluminosilicate rocks, widely used in construction and various areas of technology [1-3].

In the currently existed economic situation it is especially important for the Republic of Armenia to identify the most profitable prospective industries and by maximizing the potential of specific natural properties of this natural raw material to provide valuable and much needed products at a relatively low cost price..

In this regard, for Armenia, manufacturing of cellular glass as a highly effective thermal insulating inorganic valuable material based on recent advancements of nanotechnologies undoubtedly can be considered as one of more important and prospective branch of national economy [4].

The cellular glass is characterized by high physical and mechanical, thermophysical, and operational properties. As a thermal insulation it is widely used in all spheres of construction, in high-temperature and low-temperature technique, in the chemical industry and power engineering, as a decorative and acoustic material, filler artificial conglomerates (light concretes, cryogenic materials), shipbuilding, telecommunications, etc.

The use of relatively cheap foam plastics in construction as thermal insulating material causes serious problems with fire risk, ecological insecurity, adhesive incompatibility with concrete and stone structures. In addition to these shortcomings foam plastics with time (7 to10 years) are subjected to oxidative degradation and destruction [5.6]. Based on identified deficiencies, we can conclude that the use of foam plastics in construction should be prohibited.

In industrialized countries glass wool is considered the most best insulating material for use in construction and various branches of industry. The Corning Firm has more than 70 years produces Foarnglas in United States, and today also at four plants. Such enterprises are successfully working in Japan, China, Belarus, and Ukraine. Products of Foarnglas are supplied to different countries, including Russia, and Kazakhstan. Eurotherm product line development in Russia produces 650x450mm plates of 40 ... 180 mm thickness at the price of 848 to 1086 Euro/m^3. In Ukraine they are sold at the price of \$672/m^3, and blocks of its home-made units – at \$243/m^3. Russian "NEOPORM" blocks cost 30500 to 35000 Rub/m^3.

The high cost of foam glass compared with cheap, short-lived and environmentally hazardous foam plastics, in the main, defines large intake of these materials to construction as heat insulators. In the current situation, necessity of cost reduction of cellular glass is unquestionable.

2. Methodology

Traditionally, in the production of cellular glass specially boiled glass is used, which is associated with significant energy costs, capital investments in construction of glass-melting furnaces and labor costs. In this regard, the exclusion of glass melting process, replacement of artificial glass in the given works by a cheaper alternative material-glass wool rock, is one of the rational ways of reducing material and energy costs in production of cellular glass, therefore, material prime cost.

For the Republic of Armenia, having no own efficient energy resources, and at the same time possessing large potential reserves of high-quality natural raw materials in the form of glass-wool and glass-crystalline rocks, the development of energy-saving technology of new types of cellular materials based on volcanic silicate and aluminosilicate rocks - perlites, pumice, tuffs, slag, etc is of great interest.

Developed by the authors the energy-saving technology of cellular glass on the basis of silicate and aluminosilicate glass wools in a single technological process combines the synthesis of a given glass

composition and the formation of its cellular structure, due to the introduction of a nanodispersed modifier and the mechanical activation of the charge, which make it possible to influence on the system at the molecular level.

In the structure of acidic volcanic glasses, in particular, perlites, there are aluminosilicate and siliceous framework "patterns", of which an alkaline modification during sintering synthesizes a new composition of aluminosilicate glass - more fusible, but possessing rather high structural strength and relaxation ability - important properties necessary for the formation of thin walls of cells.

In accordance with the methods of building materials science, the charges of cellular glass are obtained by both dispersing the glass with the gas former, as well by the method of condensation in the system of a volcanic glass-alkaline solution with the formation of a contact-condensation structure, the subsequent synthesis of a new glass composition during sintering and expansion of the pyroplastic mass.

Experimental identification of qualitative and quantitative regularities of the change of properties depending on the kinetic parameters: the number of components, the contact surface of the reacting phases (the grain size and the degree of their convergence), the temperature and rate of its growth, the thermal past of the sintered samples, allowed to establish the optimum values of these parameters and develop technology of materials with given properties.

The most effective alkaline compound, introduction of which into the composition of the masses provides not only a lower density of cellular glass, but also significantly lower temperatures of foaminging, is sodium hydroxide, the amount of which varies within the range of 5 to12 percent depending on the purpose of the cellular glass.

Sodium hydroxide is introduced into the composition of the charge as an water solution. Water is a necessary component and homogenizer of the mixture. In its presence diffusion and mass transfer of reacting substances and reaction products are facilitated; it forms a part of hydrate neoplasms, the dehydration of which, upon reaching the pyroplastic state foaming the mass.

The largest gasifying effect of hydrated new formations, as the main source of the gas phase, which foaming the glass, is provided by that part of the water that forms hydrogen bridged compounds with a relatively weakly hydroxyl-bound center of the first kind.

3. Results

The introduction of an additional gasifier agent made it possible to obtain cellular glasses with a low average density of 100-500 kg / m^3 and a coefficient of thermal conductivity of 0.04 -0.14 W /(mK), a significant compressive strength of 0.6-10 MPa with operating temperature -164^0 to + 650^0C. The reduced indicators characterize the produced cellular glass as cold-and-heat insulating.

The energy-saving technology of the developed cellular material, which excludes the glass manufacturing process, in combination with the properties indicators, enables to consider the produced cellular glass as one of the best modern and promising material.

From briquetted perlite-based blend containing as an additional gasifier of 0.015 to 0.09 percent of the carbon mass, cellular glasses of 250-490 kg/m^3 density, compressive strength of 4.3-10.1 MPa, and water absorption of 0,2 to 1.9 percent. Such cellular glass of a uniform fine porous and closed microstructure is designed for manufacturing of structures that must have high strength and thermal insulating ability, low water absorption, for example, granular cellular glass with a bulk density of 150-300 kg/m^3 for lightweight concrete.

The structure formation of spongy cellular glass is studied and their acoustic properties are determined. The coefficient of sound absorption of decorative blocks of cellular glass in the frequency range 125 to 4000Hz is 0.43-0.88, and the reverberation coefficient of sound absorption of bulk sound absorbers of finely crushed glass fractions of 5-10 and 10-20 mm in the frequency range 500-8000 Hz, respectively 0.31-0.78 and 0.32-0.66.

In developing a chemically resistant cellular glass using low-temperature alkaline treatment of the volcanic glass a low-alkali region (less than 6 percent NaOH) was studied. Standardized for chemically resistant materials of water absorption of less than 4% are characterized cellular glasses with an average density of more than 400 kg/m^3, and cellular glasses with an average density of more than 500 kg/m^3 possess water absorption of less than 2%. The most optimal mode of foaming temperature is 900^0C, exposure 30 - 60 min. Material parameters: density - 400 kg/m^3, compressive strength 10 MPa, acid resistance - 99.9 percent.

By means of ultra-thin dispersion of perlite glass with a gasifier, i.e. developed using nanotechnology, cellular glass with a wider temperature range of use - up to 1000^0C has been produced. The main indicators are the average density of 200-500 kg/m; compressive strength 3 to 7 MPa; coefficient of thermal conductivity at temperatures: 25^0C - 0,081 to 0,132, 300^0C - 0,135 to 0,187, 600^0C - 0,196 to 0,247 W/(m · K); water absorption by mass 1-3%. The nature of porosity is closed, inaccessible to the penetration of liquid and gases; the structure is uniform, finely porous.

Areas of application: chemical, metallurgical, construction industry, etc.

The field of application of cellular glasses in the form of products and granules is further expanded by involving in the sphere of their production other silicate and aluminosilicate rocks. As such, we have conducted thorough tests for diatomites, flasks, trefoils,

vitrophyres, zeolite-containing tuffs, clays, quartz sand and industrial wastes of silicate composition [7,8].

On the basis of granular cellular glass, heat-insulating concretes and aerated concrete, structural-heat-insulating and structural concrete (see the below Table) have been fabricated.

Table 1 Indicators of Foamglass Granular Concrete

Type of concrete	Average density, kg/m3	Compressive strength, MPa	Coefficient of thermal conductivity, W/(m · k)
Thermal insulation	300 ... 550	1,1 ... 2,5	0,07 ... 0,12
Structural and thermal insulation	600 ... 1000	3,5...10,0	0,14...0,24
Structural	1300 ... 1400	16.4 ... 18.6	0.36-0.40

Compared with full-strength concrete on natural lightweight aggregates, foam glass granulate concrete is advantageously distinguished by the main parameters. When replacing expanded-slag concrete (ρ=1400kg/m^3, R_{comp} = 7.5MPa, λ = 0.41 W/(m · K) to foam glass granular concrete, it becomes possible to reduce the thickness of concrete fence by 4 times, while the mass of a square meter is reduced by 3 – 4 times depending on the density of foam glass granular concrete. The achieved reduction of materials' specific consumption is especially significant for cement economy, for example, with a 2.2 to 3.0 m^2·deg/W heat transfer resistance, the cement savings is 228 to 315 kg/m^2 for fences without cladding and 207 to 310 kg/m^2 for fencing with 2 cm tuff facing [9].

It is also important to note that the developed technology of foam glass from local raw materials does not lead to predatory practices in consumption of natural resources. The consumption of lithium pumice as a concrete aggregate is 4 times higher than the consumption of this raw material for the production of foam glass-granular concrete [10].

4. Conclusion

The organization of cellular glass production according to the developed energy-saving nanotechnology on the basis of silicate and aluminosilicate glass-wool rocks, in which Armenia is very rich, is undoubtedly an urgent task capable to introduce high technology into an industry, and to organize the production of high-quality and to meet the growing need in such materials both at home and abroad.

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THE ROLE AND IMPORTANCE OF DRAINAGE SYSTEMS IN THE RECONSTRUCTION OF DAMAGED HEADWALLS

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Keywords: headwall, drainage, geotextile arrangement**,** rock fragment, macadam/crushed rock.

Abstract

The aim of the article is to review the organization of expeditious reconstruction (restoration) of drainage systems damaged during combat operations due to bombing and artillery shelling. These works result in the restoration of the stability of headwalls, which provide the gradation of various slopes (areas of difficult terrain) of military camps, and prevent said slopes from shifting.

As of today, drainage systems are of great importance in the international construction industry.

In several countries (France, Germany, Denmark, and Russia), geotextile posters and plastic (PVC) tubes are widely implemented in the construction and reconstruction of drainage systems.

1. Introduction

In the Republic of Armenia (especially in areas of combat operations), urgency and scarcity of time incentivize the use of simplified drainage systems in the reconstruction (restoration) of headwalls.

Following the examination of military camps, buildings and structures damaged during the April 2016 combat operations, it has become apparent that headwalls, which provide the gradation of areas with difficult slopes (of multifunctional significance) were damaged in some military camps. Some headwalls were damaged during artillery shelling, as a result of direct exposure to the explosion of shells, as well as soil shifts on slopes as a result of the shelling.

The rapid reconstruction (restoration) and reinforcement of headwalls and drainage systems is of vital importance to the rapid restoration of buildings and structures, which were built on slopes that are gradated using headwalls, and were damaged during combat operations.

The impact of atmospheric and ground water on headwalls

Areas surrounding headwalls may retain excessive moisture or leak as a result of both **natural** and **artifical factors**.

Natural factors are: hydrogeological factors, soil and physical landscape factors.

Artificial factors are: bombardment and shelling of headwalls and surrounding areas by the adversary during combat operations. (A practical example is the bombardment by the adversary of the Madaghis military camp during the April 2016 combat operations, as a result of which seepage and drainage systems were damaged and disabled). Another artificial factor is misuse of seepage (and drainage) systems, which result in various (different) disruptions.

As a result of torrential rains and snow water (as well as artificial factors), the water accumulated in the rear of the headwalls applies hydrostatic pressure to the structure of the headwall, which reduces its stability and strength, and can result in its collapse. The most effective method to counter the effects of underground waters is the construction of drainange systems.

The roles of drainage systems and particulars of their construction.

Drainage systems are implemented in areas where it is impossible to organize the removal of water via natural means. Drainage systems are necessary in headwalls, in order to prevent their collapse as a result of contact with grounds saturated with water. Drainage systems allow the reduction of water levels in the ground.

The drainage system is composed of a tube (with separate components) and a filtering layer (sand, pebbles, gravel, boulder, hard rocks, etc). The abovementioned system removes ground, snow, and rain water from the headwalls and surrounding areas, and pours it in dams, potholes and gorges.

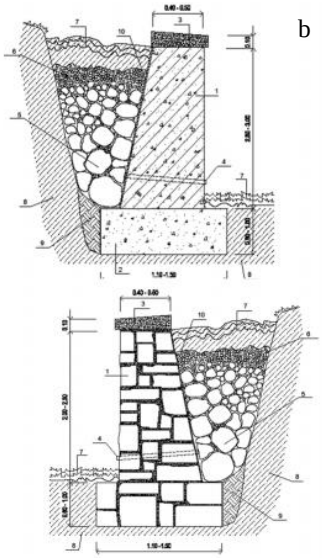
The drain can be, **transverse, longitudinal** and **balanced** (both transverse and longitudinal).

Transverse drains are capable of removing water from the rear of the headwall using the body of the headwall. This is the least costly method. Openings with 10 cm diameters are left in the body of the headwall, or tubes with diameters of 5cm to 7cm are inserted in the lower part of the body at a height of approximately 15 to 20cm, with steps of between 1.5

and 2 meters. If the layout of the headwall is composed of rocks with straight cuts, it is possible to leave a transverse, horizontal, non-cemented seam, with a 2 to 2.5m step.

In unpredictable situations characterized by lack of time, the rapid restoration of collapsed (damaged) headwalls using drainage systems becomes an issue. In such situations, it is recommended that the restoration of the headwalls take place using simplified drainage systems, which utilize different rock fragments, hard rocks, sand rubble, etc. found in the surrounding vicinity. The headwalls can be built (rebuilt) using reinforced concrete, irregularly cut rough stones, cement-sandplaster, and rock fragments with plaster (Fig. 1.2).

a b



b

Fig. 1 Headwalls: simplified drainage systems
a Reinforced concreteb. Layout of irregularly cut rough stones1. Body 2. Base (concrete, unrefined rock)3. Cornice 4. Water-removal tube 50-70mm 5. Hard irregular rock 15-25cm 6. Macadam 20-40mm 7. Vegetation layer 8. Natural soil 9. Pressed clay 10. Waterproofing layer

a **b**

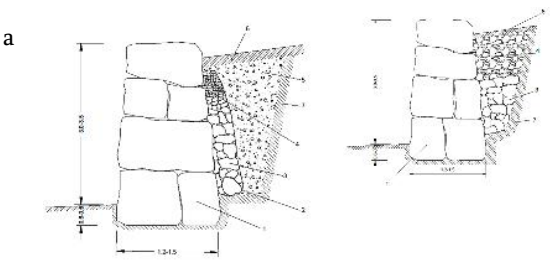


Fig.2 Headwalls with a cyclopean layout, with simplified drainage systems
a With the use of geotextile posterb. Simplified 1. Rock fragment (width 0.6-0.8m, length 0.8-1m, height 0.5-1m) 2. Geotextile arrangement 3. Irregular hard rock 15-25cm4. Macadam 40-80mm 5. Gravel 6. Vegetation layer 7. Natural soil

The construction of the abovementioned headwalls using simplified drainage systems has several advantages. They are:

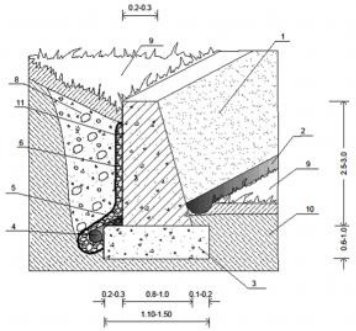
- short construction times
- use of the naturally existing foundation (rock fragments of varying sizes, hard rocks, macadam, gravel)
- the flow of ground water is directed in transverse, as well as longitudinal directions.

Several headwalls and drainage systems in Nagorno Karabakh, damaged as a result of war, have been restored using similar techniques.

Headwalls with cyclopean layouts and simplified drainage systems have been implemented in military base N of the Armed Forces of the Republic of Armenia.

Longitudinal drainage provides the opportunity to direct the flow of groundwater towards the surface inside thebodyof the headwall. The construction of such systems requires the implementation of various substances and materials, which theoretically, decide the value the structure (Fig. 3).

a



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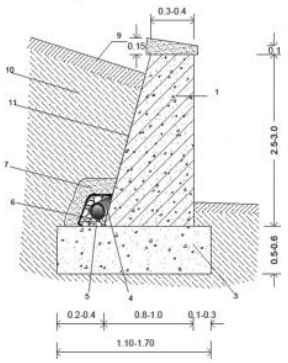

Fig. 3 Reinforced concrete headwalls with longitudinal drainage systems

1. Reinforced concrete body 2. Rill 3. Concrete foundation 4. Tube 5. Macadam 40-60mm 6. Geotextile arrangement 7. Sand 8. Gravel 9. Vegetation level 10. Natural soil 11. Waterproofing layer

The following is the procedure for building a longitudinal drainage systen

- A canal is dug along the entire length of the inner side of the headwall, at the level of the base
- The canal is filled with small-grain sand and a cushion is my made by ramming
- A geotextile poster is inserted
- A 20cm wide basalt stone is poured in
- A wavy drainage tube with a PVCexternal surface is inserted at a length of 1 linear meter with a 1-2cm bend (the inside of the tube is smooth, in order to reduce resistance to the flow of water).

- A 40cm wide macadam cover is poured over the tube and is covered by a geotextile poster, which was inserted beforehand
- Backfill is carried out.

Water is absorbed from the geotextile arrangement and is pour into the tubes through the holes, and removed from the terrace.

2. Conclusion

Rapid reconstruction and reinforcement of drainage systems through non-conventional methods based on the principle of time scarcity, makes it possible to recover the stability of headwalls that provide the gradation of difficult slopes, and neutralizes the movement of soil on hillsides.

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UNIFORM STATIONARY LAMINAR FLOW OF VISCOUS FLUID IN OPEN CHANNELS OF PARABOLIC CROSS-SECTION

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Keywords: Cannel, viscous flow, parabolic cross-section, flow regularities, hydrodynamic parameters.

Abstract

Studies of regularities of viscous fluid flow in channels of parabolic cross-section, except theoretical interest also have important application significance, for they are widely used in irrigation, water disposal and other fields. The paper presents a simplified model of flow, when in the water layers only viscous forces are developed. In such cases the slotion of the problem is reduced to integration of Laplace equation. As a result of integration formulae for calculation of hydrodynamic parameters of flow has been obtained. These formulae enable to assess running phenomena and to get quantitative indices on friction stresses developing on the channel stationary walls. On the basis of the study the grounding of rational, in terms of hydraulics, cross-section dimensions becomes possible.

1. Introduction

In open channels of parabolic cross-section laminar flow of viscous fluid is possible under low velocity or flow of fluid of high viscosity [2,3,4]. Under these conditions in open channels fluid particles execute only longitudinal displacements [1,2]. When the fluid flow direction coincides with z axis, then $U_x = U_y = 0$, $U_z = U_z(x, y)$ [1,2,4]. Therefore, under these conditions the differential equation of the fluid flow will have the folloing form

$$\nu\left(\frac{\partial^2 U_z}{\partial x^2}+\frac{\partial^2 U_z}{\partial y^2}\right)+F_z=0, \qquad (1)$$

where Fz is the component of a unit spatial force directed along z axis.

In open channels spatial forces are projection of the gravity force in floe direction, hence

$$F_z = g\cdot\sin\alpha, \qquad (2)$$

where α is the channel's slope with the horizon.

Taking into consideration Eq.(2) the differential equation of the viscous fluid flow in open channels can be written as [1]

$$\nu\left(\frac{\partial^2 U_z}{\partial x^2}+\frac{\partial^2 U_z}{\partial y^2}\right)=-g\cdot\sin\alpha. \qquad (3)$$

To integrate the above equation the problem's boundary conditions are set, which depend on the shape of cross-section of the channel.

Let us study most commonly occuring relationships of viscous fluid flow in cylindrical channels of parobolic cross-section.

The outline of cylindrical channels of parabolic cross-section shown in Fig.1, in xoy plane of the xyz coordinate system is determined by $ax^2 - h = y$ equation. Study of laminar flow of viscous fluid in such channels is reduced to integration of Eq.(3) under the following boundary conditions

$$U_z(x,y)=0, \quad when \quad ax^2-h=y, \qquad \frac{\partial U_z(x,y)}{\partial x}\Big|_{y=0}=0, \quad when \quad x=0. \qquad (4)$$

Solution of Eq.(3) satisfying the above boundary conditions, according to which on the outline of the channel it becomes zero, will be

$$U_z(x,y)=A\left(1-\frac{ax^2-y}{h}\right), \qquad (5)$$

where A is an arbitrary constant the vaue of which is determined by substitution of Eq.(5) into Eq.(3), as a result we get

$$\nu\cdot\frac{2Aa}{h}=g\sin\alpha,$$

from which we have

$$A=\frac{g\cdot h\cdot\sin\alpha}{2a\cdot\nu}. \qquad (6)$$

Having found the value of A constant, we can get velocity distribution relationships in free cross-sectional area of parabolic cross-sectional channels

$$U_z(x,y)=\frac{g\cdot h\cdot\sin\alpha}{2a\cdot\nu}\left(1-\frac{ax^2-y}{h}\right). \quad (7)$$

It follows from the derived formula of velocity distribution that velocity vectors form a paraboloid of revolution passing through the given point of the full cross-section. The maximum velocity is developed at the centre of the free surface, and when $x=y=0$, we have

$$U_{z\max}=\frac{g\cdot h\cdot\sin\alpha}{2a\cdot\nu}. \quad (8)$$

After that Eq.(7) of velocity distribution in the full cross-section will have the following form

$$U_z(x,y)=U_{z\max}\left(1-\frac{ax^2-y}{h}\right). \quad (9)$$

In the free surface, where $y=0$, the regularity of velocity distribution will be

$$U_z(x,0)=U_{z\max}\left(1-\frac{ax^2}{h}\right), \quad (10)$$

therefore, in the free surface velocities are distributed according to quadratic parabola (Fig.1). In the vertical plane, where $x=0$, velocities are distributed occurred linearly (Fig.2),

$$U_z(0,y)=U_{z\max}\left(1+\frac{y}{h}\right) \quad . \quad (11)$$

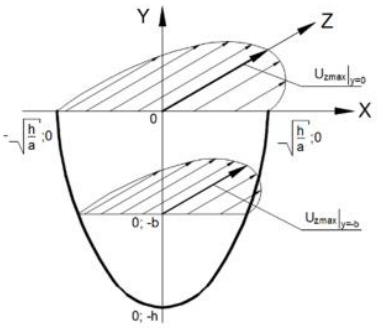

Fig.1 Velocity diagrams in $Y=0, Y=-b$ planes

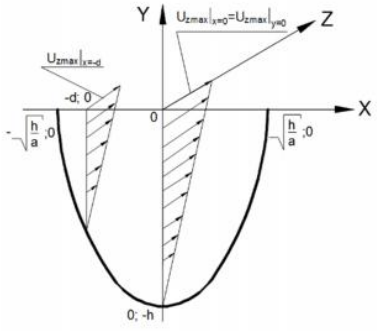

Fig.2 Velocity diagrams in $X=0, X=-d$ planes

According to Eqs.(10) and (11) in the fluid layers friction stresses are developed, which are determined by a known Newton formula [3,4]

$$\tau=\pm\mu\frac{dU}{dn}. \quad (12)$$

In $y=0$ plane velocith changes occur along x axis, therefore, in this plane τ_{xoz} friction stresses will appear, we have

$$\tau_{xoz}=\pm\mu U_{z\max}\frac{2ax}{h}. \quad (13)$$

It follows from the obtained formula that in $x=0$ plane friction stresses are distributed linearly (Fig.3), in which maximum stresses of friction are developed near fixed walls, where $x=\pm\sqrt{\frac{h}{a}}$,

we have

$$\tau_{zox}^{\max}=\pm\mu U_{z\max}\frac{2ax}{h}. \quad (14)$$

In $X=0$ plane velocity changes occur along Y axis, therefore, in this plane τ_{yoz} fricton stresses will be developed, we get

$$\tau_{yoz}=\pm\mu U_{z\max}\cdot\frac{1}{h}. \quad (15)$$

It follows from Eq.(15) that in $Y=0$ plane friction stress depth along th remain constant (Fig.4).

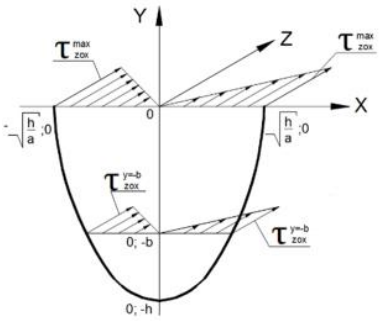

Fig.3 Diagrams of friction stresses in $Y=o, Y=-b$ plane

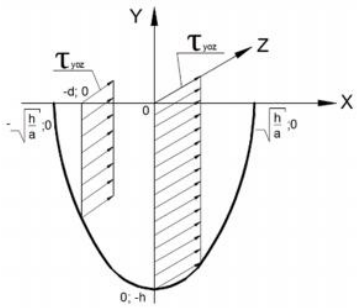

Fig.4 Diagrams of friction stresses in $X = o, X = -d$ plane

In planes parallel to the free surface velocity distribution regularities are obtained by Eq.(9) by substituting $y = -b$, where $-b$ is the distant of the given plane from the free surface, we get

$$U_z(x,y) = U_{z\max}\left(1 - \frac{ax^2 + b}{h}\right). \quad (16)$$

According to this regularity of velocity distribution friction stresses developed in $y = -b$ horizontal plane are determined by Eq.(13), at that near fixed walls their value will be (see Fig.3)

$$\tau_{xoz}\big|_{y=-b} = \pm\mu U_{z\max}\frac{2a}{h}\cdot\sqrt{\frac{h-b}{a}}\,. \quad (17)$$

In a vertical plane where $x = \pm d$ distribution of velocities occurs linearly as shown in Fig.4

$$U_z(x,y) = U_{z\max}\left(1 - \frac{ad^2 - y}{h}\right), \quad (18)$$

and to which respective friction stresses will be determined by Eq.(15).

To obtain curves of constant velocities in the full cross-section Eq.(7) is used

$$U_z(x,y) = \frac{g\cdot h\cdot\sin\alpha}{2a}\left(1 - \frac{ax^2 - y}{h}\right) = const,$$

from which we will have

$$ax^2 - y = const\,. \quad (19)$$

It follows from Eq.(19) that curves of constant velocities are parabols of which verticies are located at different heights as seen in Fig.5.

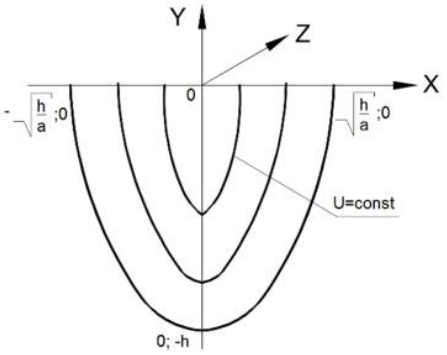

Fig.5 Curves of constant velocities

The volumetric flow passing through the full cross-section is [1,5]

$$Q = \iint_A U_z(x,y)\,dzdy = U_{z\max}\iint_A\left(1 - \frac{ax^2 + y}{h}\right)dxdy$$

:

Substituting the surface integral by a double integral,

we have

$$Q = U_{z\max}\int_{-\sqrt{h/a}}^{\sqrt{h/a}} dx \int_{ax^2-h}^{0}\left(1 - \frac{ax^2 + y}{h}\right)dy =$$

$$U_{z\max}\int_{-\sqrt{h/a}}^{\sqrt{h/a}} dx\cdot\left(y - \frac{ax^2}{h}y + \frac{y^2}{2h}\right)\Bigg|_{ax^2-h}^{0} = \quad (20)$$

$$= -U_{z\max}\int_{-\sqrt{h/a}}^{\sqrt{h/a}}\left(ax^2 - \frac{a^2x^4}{2h} + \frac{h}{2}\right)dx =$$

$$= \frac{2}{3}h\sqrt{\frac{h}{a}}\cdot U_{z\max}.$$

Substituting the value of velocity $U_{z\max}$ into Eq.(20), we get

$$Q = \frac{h^2 g\cdot\sin\alpha}{3a}\sqrt{\frac{h}{a}}\,. \quad (21)$$

The average velocity of the full cross-section will be

$$V = \frac{Q}{A}, \quad (22)$$

were A is the area of the full cross-section and is determined by $A = \iint_A dxdy$.

Replacing the last surface integral by a double integral, we have

$$A = 2\int_0^{\sqrt{h/a}} dx \int_{ax^2-h}^{0} dy = 2\int_0^{\sqrt{h/a}}\left(ax^2 - h\right)dx = \frac{4}{3}h\sqrt{\frac{h}{a}}\,. \quad (23)$$

Substituting values of Q and A we will have the average velocity of the full cross-section

$$V = \frac{\frac{2}{3}h\sqrt{\frac{h}{a}}\cdot U_{z\max}}{\frac{4}{3}h\sqrt{\frac{h}{a}}} = \frac{U_{z\max}}{2}. \qquad (24)$$

5. Conclusion

Findings of the study of regularities of the viscous fluid flow in cylindrical channels of parabolic cross-section enable to evaluate hydrodynamic parameters of the flow and friction stresses developed on the fixed wall. As a result of the careflly made analysis and on the basis of the condition of hydraulically reasonable cross-section, it is possible to chose geometrical dimensions of the cross-section.

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CONTACT PROBLEMS ON INTERACTION OF THIN-WALLED ELEMENTS WITH AN ELASTIC LAYER UNDER TORSION

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Abstract

Two problems of mathematical theory of elasticity on contact interaction of thin-walled elements in form of round washers with an infinite elastic layer under torsion are considered. These kinds of problems are closely related to important in terms of engineering practice issues of load transmission from thin-walled elements to massive deformable bodies. They are encountered in calculations of durability of elements of engineering constructions in construction mechanics, in measuring equipment, mechanical engineering, and other spheres of practice. In the first problem, the infinite elastic layer is rigidly fastened on its lower surface, whereas the upper one is enhanced with a round thin washer. On the washer, tangential twisting forces are acting which are transmitted to the elastic layer-foundation through the contact area. In the second problem, the elastic layer is twisted by two round washers of different radii fastened with the layer on its upper and lower surfaces. Meanwhile, twisting tangential forces are acting on the washers. The layer is investigated in the framework of precise equations of elasticity theory. For the round washers simplified physical model of Melan's type for stringers is applied.

1. Introduction

Results of studies of problems on torsion of elastic bodies are broadly used in calculations of durability and longevity of many engineering constructions and their components. The main results of this field of elasticity theory are, enough completely, reflected in the well-known monographs [1-3], as well as in the collective monograph [4]. A similar problem is as well considered in the paper [5].

In the present paper with the help of mathematical apparatus of Hankel integral transforms, the solutions of the formulated contact problems are reduced to the solution of Fredholm's integral equation of the first kind and to the solution of a system of two such integral equations. The solutions of the latter equations are obtained through their reduction to infinite systems of linear algebraic equations or by the numerical and analytical method of collocations with the application of Gauss's quadrature formulae.

2. Discussion

Problem formulation and derivation of the basic equations. To solve the formulated problems, let us first determine stress-strain state of the elastic layer in the first problem. Let in the cylindrical system of coordinates (r,φ,z) an elastic layer of thickness h_0 and shear modulus G_0 occupies the domain

$$\Omega_0=\{0\le r<\infty,\ 0\le\varphi<2\pi,\ \ 0<z\le h_0\}.$$

Further, let on the upper plane $z=h_0$ of the layer twisting forces of intensity $\tau_0(r)$ are acting, and the lower plane is rigidly fastened. Then determination of the stress-strain state can be reduced to the solution of the following boundary-value problem

$$\begin{cases}\dfrac{\partial^2V_0}{\partial z^2}+\dfrac{1}{r}\dfrac{\partial V_0}{\partial r}-\dfrac{V_0}{r^2}+\dfrac{\partial^2V_0}{\partial r^2}=0\ \ (0\le r<\infty,\,0<z\le h_0)\\ \tau_{z\varphi}\big|_{z=h_0}=G_0\dfrac{\partial V_0}{\partial z}\Big|_{z=h_0}=\tau_0(r)\\ V_0\big|_{z=0}=0\\ \tau_{z\varphi}^2+\tau_{r\varphi}^2\to 0\ \ when\ r^2+z^2\to\infty\end{cases}\qquad(1)$$

Here $V_0=V_0(r,z)$ – is the component of the displacement of points of the layer in circular direction under torsion whereas $\tau_{z\varphi}$ and $\tau_{r\varphi}$ are the twisting tangential contact strains.

For the second problem, when on the boundary surfaces $z=h_0$ and $z=-h_0$ of the layer tangential forces of intensities $\tau_0(r)$ and $\tau_1(r)$, respectively, are acting, the described problem is mathematically formulated similarly, as described above

$$\begin{cases}\dfrac{\partial^2V_0}{\partial z^2}+\dfrac{1}{r}\dfrac{\partial V_0}{\partial r}-\dfrac{V_0}{r^2}+\dfrac{\partial^2V_0}{\partial r^2}=0\ \ (0\le r<\infty,\,-h_0<z\le h_0)\\ G_0\dfrac{\partial V_0}{\partial z}\Big|_{z=h_0}=\tau_0(r)\ \ G_0\dfrac{\partial V_0}{\partial z}\Big|_{z=-h_0}=\tau_1(r)\end{cases}\qquad(2)$$

The solution of the boundary-value problem (1) is

constructed by the method of Hankel integral transforms assuming

$$\left\{\overline{V}_0(\lambda,z),\overline{\tau}_0(\lambda),\overline{\tau}_1(\lambda)\right\}=\int_0^\infty\left\{V_0(r,z),\tau_0(r),\tau_1(r)\right\}rJ_1(\lambda r)dr$$

Here $J_1(r)$ is Bessel function of the first kind of index 1 and λ is spectral parameter with $0<\lambda<\infty$. Using the well-known relation [5]

$$\int_0^\infty r\left(\frac{d^2f}{dr^2}-\frac{1}{r}-\frac{df}{dr}-\frac{f}{r^2}\right)J_1(\lambda r)dr=-\lambda^2\overline{f}(\lambda),$$

we eventually arrive at the boundary-value problem for ordinary differential equation:

$$\begin{cases}\dfrac{d^2\overline{V}_0}{dz^2}-\lambda^2\overline{V}_0=0 \quad (0<z<h_k)\\ \left.\dfrac{d\overline{V}_0}{dz}\right|_{z=h_k}=\dfrac{\overline{\tau}_0(\lambda)}{G_0} \quad \left.\overline{V}_0\right|_{z=0}=0 \quad (0<r<\infty).\end{cases} \tag{3}$$

The solution of the problem (3) has the form

$$\overline{V}_0(\lambda,z)=Ach(\lambda z)+Bsh(\lambda z) \quad ,$$

(4)

where A and B are determined from the boundary conditions as follows:

$$A=0,\qquad B=\frac{\overline{\tau}_0}{\lambda G_0}\frac{1}{\mathrm{ch}(\lambda h_0)}$$

Substituting these expressions for A and B into (4), we obtain

$$\overline{V}_0(\lambda,z)=\frac{\overline{\tau}_0}{\lambda G_0}\frac{\mathrm{sh}(\lambda z)}{\mathrm{ch}(2\lambda h_0)} \tag{5}$$

Applying the inverse transform to this equality, we find the displacements of points of the upper surface of the layer

$$V_0(r,h)=\int_0^\infty W_0(r,r_0)\tau_0(r_0)r_0dr_0 \quad (0<r<\infty) \tag{6}$$

where

$$W_0(r,r_0)=\frac{1}{G_0}\int_0^\infty J_1(\lambda r)J_1(\lambda r_0)th(\lambda h_0)d\lambda$$

Similarly, we find the solution of the boundary-value problem (2), i.e. the displacements of points of the upper and the lower surface of the layer respectively

$$V_1(r)=\int_0^\infty W_1(r,r_0)\tau_0(r_0)r_0dr_0-\int_0^\infty W_2(r,r_0)\tau_1(r_0)r_0dr_0$$

$$V_2(r)=\int_0^\infty W_2(r,r_0)\tau_0(r_0)r_0dr_0-\int_0^\infty W_1(r,r_0)\tau_1(r_0)r_0dr_0 \tag{7}$$

$(0<r<\infty)$

where

$$W_1(r,r_0)=\frac{1}{G_0}\int_0^\infty J_1(\lambda r)J_1(\lambda r_0)cth(2\lambda h_0)d\lambda$$

$$W_2(r,r_0)=\frac{1}{G_0}\int_0^\infty J_1(\lambda r)J_1(\lambda r_0)\frac{1}{sh(2\lambda h_0)}d\lambda \tag{8}.$$

Now let us turn to the solution of the formulated contact problems. In the first problem, a round washer of radius a and constant thickness h_1 $(h_1<<a)$ is fully fastened with its foundation on the upper surface of the layer whereas the lower surface of the layer is rigidly fastened (Fig. 1). It is required to determine the rule of distribution of contact stresses $\tau_0(r)$ on the contact area of junction of the washer with the layer as well as the angle of rigid torsion of the washer when a tangential force of intensity $q_0(r)$ is applied to the washer.

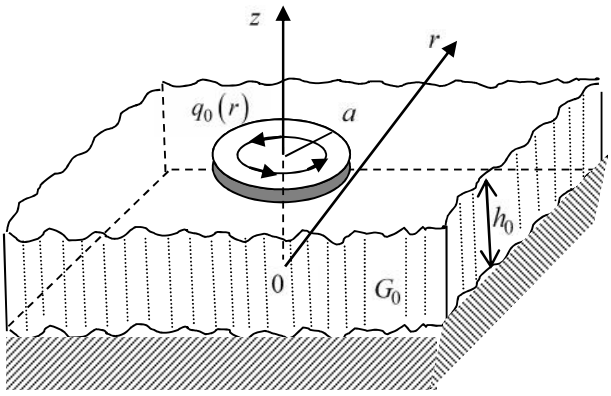

Fig.1 The Elastic layer twisted by one washer

With analogy to the well-known results from [6, 7],

the equation of equilibrium of the washer in the investigated problem is reduced to the Melan's model for torsion of a thin round washer

$$G_1h_1\left(\frac{d^2\overline{V}_1}{dr^2}+\frac{1}{r}\frac{d\overline{V}_1}{dr}-\frac{\overline{V}_1}{r^2}\right)=\tau_0-q_0 \tag{9}$$

$(0<r<a).$

Here $\overline{V}_1(r)$ is the displacement of points of the washer and G_1 is its shear modulus.

The solution of the equation (9) can be represented in the form of

$$\overline{V}_1(r)=\frac{1}{2rG_1h_1}\int_0^r\left(r^2-r_0^2\right)\left(\tau_0(r_0)-q_0(r_0)\right)dr_0+C_1r. \tag{10}$$

Here C_1 is the angle of washer rotation.

Now taking into account the contact condition on the plane $z = h_0$

$$\overline{V}_1(r) = V_0(r) \quad (0 \le r < a) \tag{11}$$

and introducing the following immeasurable variables

$$r = a\rho, \quad r_0 = a\rho_0, \quad h = h_0/a, \quad \lambda a = s, \quad c = c_1 a,$$
$$\mu_0 = G_0 a / 4G_1 h_1, \; K_0(\rho, \rho_0) = aW_0(a\rho, a\rho_0)$$
$$\tau(\rho) = \tau_0(a\rho)/G_0, \quad q(\rho) = q_0(a\rho)/G_0,$$

after performing several elementary transformations, we obtain the following determining integral equation

$$\int_0^1 K_0(\rho, \rho_0)\tau(\rho_0)\rho_0 d\rho_0 -$$
$$-\mu_0 \frac{1}{\rho}\int_0^1 \left(1 + sign(\rho - \rho_0)\right)\left(\rho^2 - \rho_0^2\right)\tau(\rho_0)d\rho_0 = \tag{12}$$
$$= -\mu_0 \frac{1}{\rho}\int_0^1 \left(1 + sign(\rho - \rho_0)\right)\left(\rho^2 - \rho_0^2\right)q(\rho_0)d\rho_0 + c\rho$$

where

$$K_0(\rho, \rho_0) = \int_0^\infty J_1(s\rho)\, J_1(s\rho_0)\, th(sh)ds$$

The equilibrium condition of the washer

$$\int_0^1 \left[\tau(\rho_0) - q(\rho_0)\right]\rho_0^2 d\rho_0 = 0 \tag{13}$$

should be added to the equation (11).

In the second problem, an elastic layer of thickness $2h_0$ is twisted by two round washers of different radii a and b fastened with the layer on its upper and lower surfaces, respectively. In that, twisting tangential forces of intensities $q_0(r)$ and $q_1(r)$ are acting on the washers (Fig. 2).

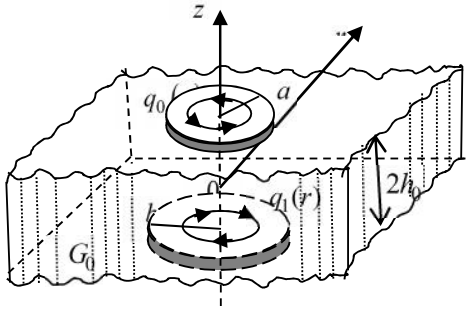

Fig. 2 The elastic layer twisted by two washers

Let us put down the contact conditions of the elastic layer Ω_0 with the round washers fastened on its upper and lower surfaces

$$V_1(r) = \overline{V}_1(r) \quad (z = h_0) \quad (0 < r \le a);$$
$$V_2(r) = \overline{V}_2(r) \quad (z = -h_0) \quad (0 < r \le b); \tag{14}$$
$$\overline{V}_2(r) = \frac{1}{2rG_1h_1}\int_0^r \left(r^2 - r_0^2\right)\left(\tau_1(r_0) - q_1(r_0)\right)dr_0 + C_2 r,$$

where $\overline{V}_2(r)$ is the displacement of points of the lower washer and C_2 is its rotation angle.

Substituting the expressions for $V_1(r)$, $\overline{V}_1(r)$ and $V_2(r)$, $\overline{V}_2(r)$ from (7) and (10) into (14), we obtain a system of two Fredholm-Volterra integral equations of mixed type with respect to the sought contact tangential stresses $\tau_0(r)$ and $\tau_1(r)$:

$$\begin{cases} \int_0^a W_1(r, r_0)\tau_0(r_0)r_0 dr_0 - \int_0^b W_2(r, r_0)\tau_1(r_0)r_0 dr_0 = \\ = \frac{1}{2rG_1h_1}\int_0^r \left(r^2 - r_0^2\right)\left(\tau_0(r_0) - q_0(r_0)\right)dr_0 + C_1 r \quad (0 < r \le a) \\ \int_0^a W_2(r, r_0)\tau_0(r_0)r_0 dr_0 - \int_0^b W_1(r, r_0)\tau_1(r_0)r_0 dr_0 = \\ \frac{1}{2rG_1h_1}\int_0^r \left(r^2 - r_0^2\right)\left(\tau_1(r_0) - q_1(r_0)\right)dr_0 + C_2 r \quad (0 < r \le b) \end{cases} \tag{15}$$

In particular, when of $a = b$, the system (15) has the following form

$$\begin{cases} \int_0^a \left[W_1(r, r_0)\tau_0(r_0) - W_2(r, r_0)\tau_1(r_0)\right]r_0 dr_0 = \\ = \frac{1}{2rG_1h_1}\int_0^r \left(r^2 - r_0^2\right)\left(\tau_0(r_0) - q_0(r_0)\right)dr_0 + C_1 r \\ \int_0^a \left[W_2(r, r_0)\tau_0(r_0) - W_1(r, r_0)\tau_1(r_0)\right]r_0 dr_0 = \\ = \frac{1}{2rG_1h_1}\int_0^r \left(r^2 - r_0^2\right)\left(\tau_1(r_0) - q_1(r_0)\right)dr_0 + C_2 r, \; (0 < r \le a) \end{cases} \tag{16}$$

This system is equivalent to the following system of Fredholm integral equations of the first kind:

$$\begin{cases} \int_0^1 \left[K_1(\rho, \rho_0)\tau(\rho_0) - K_2(\rho, \rho_0)\tilde{\tau}(\rho_0)\right]\rho_0 d\rho_0 - \\ -\frac{\mu_0}{\rho}\int_0^1 \left(1 + sign(\rho - \rho_0)\right)\left(\rho^2 - \rho_0^2\right)\tau(\rho_0)d\rho_0 = f_1(\rho) \\ \int_0^1 \left[K_2(\rho, \rho_0)\tau(\rho_0) - K_1(\rho, \rho_0)\tilde{\tau}(\rho_0)\right]\rho_0 d\rho_0 - \\ -\frac{\mu_0}{\rho}\int_0^1 \left(1 + sign(\rho - \rho_0)\right)\left(\rho^2 - \rho_0^2\right)\tilde{\tau}(\rho_0)d\rho_0 = f_2(\rho) \quad (0 < \rho \le 1) \end{cases} \tag{17}$$

where the following denotations are used:

$$K_1(\rho,\rho_0)=aW_1(a\rho,a\rho_0),\ \tilde{c}_1=c_1a$$
$$K_2(\rho,\rho_0)=aW_2(a\rho,a\rho_0),\ \tilde{c}_2=c_2a,$$
$$\tilde{\tau}(\rho)=\tau_1(a\rho)/G_0,\quad \tilde{q}(\rho)=q_1(a\rho)/G_0,$$
$$f_1(\rho)=-\mu_0\frac{1}{\rho}\int_0^1\left(1+sign(\rho-\rho_0)\right)\left(\rho^2-\rho_0^2\right)q(\rho_0)d\rho_0+\tilde{c}_1\rho$$
$$f_2(\rho)=-\mu_0\frac{1}{\rho}\int_0^1\left(1+sign(\rho-\rho_0)\right)\left(\rho^2-\rho_0^2\right)\tilde{q}(\rho_0)d\rho_0+\tilde{c}_2\rho$$

Meanwhile, the equilibrium conditions hold for the washers:

$$\int_0^1\left[\tau(\rho_0)-q(\rho_0)\right]\rho_0^2d\rho_0=0$$
$$\int_0^1\left[\tilde{\tau}(\rho_0)-\tilde{q}(\rho_0)\right]\rho_0^2d\rho_0=0. \tag{18}$$

3. Methodology. On the solution of determining integral equations

With the help of spectral relations from [9, 10] the equation (12) as well as the system of equations (17) can be reduced to an infinite regular system of linear algebraic equations.

Now let us turn to the solution of integral equation (12) and rewrite it in the form

$$\int_0^1K(\rho,\rho_0)\tau(\rho_0)\rho_0d\rho_0+\int_0^1R(\rho,\rho_0)\tau(\rho_0)\rho_0d\rho_0-$$
$$-2\mu_0\int_0^\rho\frac{1}{\rho}(\rho^2-\rho_0^{\,2})\tau(\rho_0)d\rho_0=f_0(\rho) \tag{19}$$

where

$$K(\rho,\rho_0)=\int_0^\infty J_1(s\rho)\,J_1(s\rho_0)\,ds$$
$$R(\rho,\rho_0)=\int_0^\infty J_1(s\rho)\,J_1(s\rho_0)\left(1-th(sh)\right)ds$$
$$f_0(\rho)=-2\mu_0\frac{1}{\rho}\int_0^\rho\left(\rho^2-\rho_0^2\right)q(\rho_0)d\rho_0+c\rho.$$

Let us represented the solution of (19) in form of series

$$\tau(\rho_0)=\sum_{n=0}^\infty X_nP_{2n+1}^1\left(\sqrt{1-\rho_0^2}\right)\Big/\sqrt{1-\rho_0^2} \tag{20}$$

where $P_n^1(x)$ are associated the Legendre functions orthogonal on the interval $(0,1)$ with the weight $\left(\sqrt{1-x^2}\right)^{-1}$ and $\{X_n\}_{n=0}^\infty$ are the unknown coefficients to be determined. Substituting the expression for $\tau(\rho_0)$ from (20) into (19) and using the spectral relations [9, 10]

$$\int_0^1K(\rho,\rho_0)\frac{P_{2n+1}^1\left(\sqrt{1-\rho_0^2}\right)}{\sqrt{1-\rho_0^2}}\rho_0d\rho_0=$$
$$=h_nP_{2n+1}^1\left(\sqrt{1-\rho^2}\right) \tag{21}$$

where

$$h_n=(2n+1)\Gamma^2(n+1/2)\Big/\left((4n!)^2(n+1)h_n\right),$$

we arrive at the equality

$$\sum_{n=0}^\infty h_nX_nP_{2n+1}^1\sqrt{1-\rho^2}+$$
$$+\sum_{n=0}^\infty X_n\int_0^1R(\rho,\rho_0)\frac{P_{2n+1}^1\sqrt{1-\rho_0^2}}{\sqrt{1-\rho_0^2}}\rho_0d\rho_0- \tag{22}$$
$$-2\mu_0\sum_{n=0}^\infty X_n\frac{1}{\rho}\int_0^\rho\left(\rho^2-\rho_0^2\right)\frac{P_{2n+1}^1\sqrt{1-\rho_0^2}}{\sqrt{1-\rho_0^2}}d\rho_0=f_0(\rho).$$

Now we multiply both sides of the latter equation by $\rho\dfrac{P_{2k+1}^1\sqrt{1-\rho^2}}{\sqrt{1-\rho^2}}$ and integrate it from 0 to 1. We also take into consideration the orthogonality of the associated Legendre functions [10]:

$$\int_0^1\frac{P_{2n+1}^1\sqrt{1-\rho^2}P_{2m+1}^1\sqrt{1-\rho^2}}{\sqrt{1-\rho^2}}\rho d\rho=\begin{cases}0, & m\neq n\\ \dfrac{(2m+2)(2m+1)}{4m+3}, & m=n\end{cases}$$

After elementary transformations, we get an infinite system of algebraic equations with respect to the unknown coefficients X_n

$$X_m+\sum_{n=0}^\infty R_{m,n}X_n=F_m\quad(m=0,1,2,...\infty) \tag{23}$$

where

$$R_{m,n}=R_{m,n}^{(1)}+R_{m,n}^{(2)}$$
$$R_{m,n}^{(1)}=\alpha_m\int_0^1\int_0^1R(\rho,\rho_0)\frac{P_{2m+1}^1\sqrt{1-\rho_0^{\,2}}P_{2n+1}^1\sqrt{1-\rho^2}}{\sqrt{1-\rho_0^2}\sqrt{1-\rho^2}}\rho_0\rho d\rho_0d\rho$$
$$R_{m,n}^{(2)}=-2\mu\alpha_m\int_0^1\int_0^\rho(\rho^2-\rho_0^2)\frac{P_{2m+1}^1\sqrt{1-\rho_0^{\,2}}P_{2n+1}^1\sqrt{1-\rho^2}}{\sqrt{1-\rho_0^2}\sqrt{1-\rho^2}}\rho_0d\rho_0d\rho$$
$$F_m=\int_0^1f_0(\rho)\frac{P_{2m+1}^1\sqrt{1-\rho^2}}{\sqrt{1-\rho^2}}\rho d\rho$$
$$\alpha_m=\left[2(4m+3)(m!)^2\right]\Big/\left[(2m+1)^2\Gamma^2(m+1/2)\right]$$

Thus, the solution of the first problem is reduced to the solution of the infinite system of algebraic equations (23).

It can be shown, in a similar way, as in the first problem, that the solution of the second problem is also reduced to the solution of a system of infinite systems of algebraic equations.

Numerical and analytical method of collocations, in combination with the application of Gauss quadrature formulae with Chebyshev knots or with knots coinciding with zeroes of the Legendre generalized polynomials, is also applicable to the above mentioned equation (12) and the system (17).

4. Results

The following results are obtained in the paper:

a) Contact problems of new type on torsion of an elastic layer with one or two round washers are formulated;
b) The solutions of the formulated problems are reduced to the solution of a single Fredholm integral equation of the first kind or to the system of such equations that allow an effective solution.

5. Conclusion

The paper offers an effective method of solution of the determining integral equations of the formulated problems based on the application of spectral integral relations that contain the associated Legendre polynomials, as well as on the application of Gauss quadrature formulae. The solutions of these equations, the tangential contact stresses, are expressed by explicit formulae with distinctive character peculiarities.

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PROBLEMS OF CONSERVATION AND USAGE OF WILD GREENS AND WAYS OF THEIR SOLUTION IN ARMENIA

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Keywords: useful wild plants, safety of raw materials.

Abstract
The article presents data on diversity of the flora of Armenia where useful plants are of great importance and are used by the people as food, vegetables, herbs, honey stuff, spice, dye-stuff, etc. The article argues that population of Armenia uses different quantity of greens and raw materials in different regions of Armenia. According to this data the first place is occupied by Lori region and as a result of it the research was mainly carried out in Lori and wild greens were examined for the study.
Through annual monitoring it is based that as a result of global warming and unsparing use of herbal resources by humanity and other human factors, vegetables and many other useful wild plants are threatened with extinction. Though there are known ways of their conservation and rational use of reserves, protected areas and measures to regulate their gathering from nature, they are not carried out for wild greens. So far, safety problems of wild greens raw materials are not solved as well. The article highlighted some species of wild greens putting into cultivation.

1. Introduction
As a result of favorable geographical position, from the point of view of complex geological structure, climate, soil diversity, vertical zoning of relief as well as flora-genetic viewpoint, Armenia is distinguished with rich vegetation, plant diversity and variety.
Today there are over 3650 vascular plants in Armenia. Together with the floristic composition of plants and diversity of plant forms there are a great number of plants useful for food and for human economic activity, these are especially greens. Many interesting facts on flora of Armenia are found in records of ancient historians and travelers of different nationalities (Greek, Italian, Russian, etc.). Here it is worth mentioning French botanist Tournefort, who visited Araksvalley in 1700-1712, explored the diversity of the soils, then climbed up the Mount of Ararat and substantiated the essence of vertical zoning. An interesting investigation was carried out by German botanist S. Koch in 1843. Wagner M. described Ararat and Aragats vegetation and Buhse F.A. (1889) studied flora and plants of Yerevan-Nakhichevan-Tabriz route in 1847-1849. Zeidlitz N.K. (1883) collected 2000 types of greens to explore flora and plants of Armenia. Amirdovlat and Shahrimanyan (1818) described in details and grouped many types of wild vegetables and spice plants that have been used by the Armenian people for centuries. Mkhitar Heratsi (1184) and Amirdovlat (1478,1482) mention that Transcaucasia peoples, especially Armenians, as well as many foreign nations still use many wild plants as food, as well as a way to spice the food with a new taste [1,5]. Along with them, many Armenian researchers referred to the subject of diversity of Armenian plants [5,6,7]. Role of Vavilov N.I., Zhukovski M.F., Pryanishnikov D.N., Maltsev A.Kh., Grossgeym A.S., Troitski A.G. and others is invaluable for study of flora of Armenia. Vavilov N.I. once advised Armenian scientists to found a reservation of corn plants in Armenia which was implemented in 1981 [2].
The mankind has always been interested in wild plants that are classified to national wealth as vegetable resources. Here their usage for genetic breeding purposes is interesting. There is comparatively small number of scientific researches concerning many useful wild plants, including problems of biological, economic, environmental and sustainable development, conservation and rational use, of quantity and quality of raw materials derived from them [3]. Up to now no importance is given to problems of their usage for mankind, particularly to their efficiency. In conditions of the global warming of the Earth, intensive use and desertification of lands, degradation of landscapes, their structure violations, construction of settlements and roads, new industrial technologies, new enterprises and due to other anthropogenic factors, the area and quantity of wild greens, very important for mankind, decrease [2, 3].
Natural vegetable raw materials as an area for receiving crop production, is the most appropriate and effective with countries having scarce land unfit for agriculture, such as Armenia. It should be noted that Armenia is among unique countries in the world with many kinds of wild greens and diverse flora, these plants varied flora storage, selection of raw materials and a unique source of vegetable raw materials [3].
Many researchers believe that although the abundance of agricultural crops improved heterogeneous technology development, production of high-yielding varieties are invested in relatively, which made high and sustainable agricultural raw materials, however some people always feel the need to pick up crop material from nature. In fact, in conditions regulated by nature, without human intervention and additional expenses people take raw materials from nature. Moreover, some of the greens are used for food, some for sale at the markets (Fig.).

Fig. Sale of wild greens in Vanadzor market

Moreover, our research showed that certain families consider money received from the sale of greens as their only source of income.

Though Armenia is a country where the population uses great quantity of wild greens, but scientific data are not fully presented in this area, especially in terms of the number of plants and the use of wild greens raw materials in the regions. The above circumstances justify the relevance and purpose of the article.

2. The study material and method

Studies have mainly been conducted in the region of Lori, northern part of Armenia (2010-16), where most of the population uses wild greens. The types of plants most commonly and widely used by the population were chosen for studies on wild greens. Their taxonomic data are presented in Table 1.

Ways of usage of the described greens are of significant interest (Table 2).

Table 1 Studied Herbs and their Taxonomy

№	Name of the Herb	Family	Genus:	Type species:
1	Rumexacetosa	*Polygonaceae*	*Rumex L*	*Rumexacetosa L.*
2	Malva	*Malvaceae*	*Malva L*	*MalvaneglectaWallr.*
3	Urticadioica	*Ürticaceae*	*Urtica L*	*UrticaDioica L.*
4	Heracleum	*Apiaceae*	*Heracleum L.*	*Heracleumsosnowskyi*
5	Falcaria	*Umbelliferae Ziliacea*	*Falcaria [RIV.] FABR.*	*Falcaria vulgaris BERNH.*
6	Chaerophýllum	Apiaceae	*Chaerophyllum L*	*Chaerophyllumbulbosum L.*
7	Polygonátum	*Ruscaceae*	*PolygonatumL*	*Polygonatumodoratum Adans L.*
8	Sempervivum	*Crassulaceae*	*SempervivumL.*	*S. globiferum L.*

Table 2 Ways of usage of wild greens raw materials

In the current work we have studied the market and current problems of vegetable market with regard to sales and profitability. As a research method we choose social survey, annual monitoring and field observations. In cooperation with sociologists a questionnaire consisting of 30 questions has been formed which was distributed to 1,500 families, we also used services of part-time students. To determine the quantitative changes of the wild vegetable plants in the nature and for annual monitoring we have chosen four communities: natural landscapes adjacent to the city of Vanadzor, villages of Dsegh, Pambak, Arjut separated by 5 stations with about 1000 square meters surface each. Observations and statistics were prepared for the territory closest to the community (2016) where herbs exist today. Quantitative calculation of the greens was made once in a year, in September-October. Surveys with the population were held in communities and researches for finding out quantitative problems of usage of the greens were carried out in all over republic.

3.Results of the research

Studies showed that the region of Lori occupies the first place (about 20 types of greens) with the indexes of both the number of types of wild greens and the usage of herbs' raw materials by the population. Moreover, there are greens which are popular in the whole republic, are used in fresh and raw state but in small quantity. Those are tragopogon (Tragopogondubius) and Capsélla búrsa-pastóris. In the city of Yerevan, regions of Armavir, Ararat, Aragatsotn the population mainly uses spinach (Falcaria), ramson (Chaerophýllum) and portulaca (Portulacea) which are sold at markets in spring. Fennel (Cachrys microcarpa) is mostly used in the region of Tavush. In the Republic of Armenia population of the regions of Syunik and Vayk uses comparatively small quantity of vegetable greens and raw materials. This is conditioned with the populations' traditions and customs and absence of natural greens in the area.

We have found out that greens differs with the usefulness of the raw material which conditions the duration of their usage by the population and with quantitative indexes. Here the quantity of the raw materials gathered by one person is also of importance (Table 3).

We can see from Table 3 data that greens in nature differ with the qualitative indexes of usage of the raw materials and with data of gathered raw materials by one person. This is explained by the biological peculiarities of greens and local meteorological components, the number of not cold days, heat, moisture, etc.

The mentioned data are important as they have direct impact on the annual profitability of the greens. We have tried to find out how much every family of the Lori region spends for purchase of greens. According to average data (results of the survey) this

N	Name of the Herb	Usable part of the greens	Way of raw material usage		
			Salad or boiled	Marinated	Parched
1	Chaerophýllum	stem, sprout	+	+	-
2	Rumexacetosa	Leaf	-	-	+
3	Falcaria	Sprout	+	-	-
4	Malva	Sprout	+	-	-
5	Urticadioica	Sprout	+	-	-
6	Polygonátum	Sprout	+	-	-
7	Heracleum	Stem with ascending taper	-	+	-
8	Sempervivum	Sprout	+	-	-

index is from 15000 (the city of Vandazor) up to 30000 (village of Meghrut) drams (approximately 30-60 $).

Table 3 Time and quantity of usage of greens' raw materials

N	Greens	During the year the suitability of natural raw materials (days)		One-day gathering of raw materials by one person	
		Minimum	maximum	kg	Tuft
1	Chaerophýllum	34	60	8	-
2	Rumexacetosa	45	65	-	4 (packet)
3	Falcaria	38	48	-	32
4	Malva	60	72	4	70
5	Urticadioica	92	108	4	-
6	Polygonátum	38	52	-	68
7	Heracleum	42	62	-	70
8	Sempervivum	34	48	-	30

We have found out with the help of the same survey that the population attaches importance to the profit gained from wild greens. A part of the people who has passed the survey it also considered to be a part of livelihood that is a result of absence of vacancies and poverty. Surveys concerning reservation and usage of wild greens showed that the risk of elimination of certain greens is a fact (Table 4).

Table 4 Quantity of greens according to research area and date

N	Greens	Research areas	Dates							Reservation%
			2010	2011	2012	2013	2014	2015	2016	
1	Chaerophýllum	1	10	9	8	8	8	7	7	70.0
2	Rumexacetosa	5	36	36	36	35	34	33	33	91.7
3	Falcaria	4	18	15	12	12	12	12	12	80.0
4	Malva	4	26	26	25	25	25	24	24	92.3
5	Urticadioica	1	40	37	37	37	36	36	35	87.5
		2	14	14	13	13	13	13	13	92.8
		3	10	10	10	8	8	8	7	70.0
		5	86	80	80	79	79	79	78	90.7
6	Polygonátum	1	18	16	15	15	14	13	13	72.2
		3	26	22	21	18	17	16	16	61.5
		5	40	38	34	34	33	32	32	80.0
7	Heracleum	2	8	7	6	6	5	5	5	62.5
		3	4	3	3	3	3	3	3	75.0
		5	4	3	3	3	3	3	3	75.0
8	Sempervivum	1	10	9	8	8	8	7	7	70.0
		3	4	4	4	4	3	3	3	75.0
		5	104	84	84	84	83	83	83	79.8

As we can see in the presented data the diversity of vegetable greens in the flora of the research area is different. Year by year the number of greens reduces in all the research areas and especially Polygonátum is most vulnerable, and this is also proved by the study of the quantity of greens in the areas next to communities. The study was carried out taking into consideration of the distance of the greens from the inhabited areas.

For reservation of wild greens the regulations on their gathering are important, Problems on reservation of wild greens and the quantity of their gathering from nature are not regulated by both legislative and executive republic and local bodies. These problems are ignored and this fact is very dangerous. Their number reduces in the nature also due to their careless usage by the population. Nothing is done for the safety of raw materials either.

4. Conclusion

As a result of national traditions and customs, variety of greens, diverse structure of natural landscapes and flora, in different regions of Armenia the population gathers from nature different quantity of types of greens, different quantity of raw materials of herbs. The region of Lori is the first with the number of types of wild greens and the quantity of raw materials. The date of their usage is different for different greens.

No work is done for reservation, careful usage, stable development of wild greens though for separate families the money earned from their sale reduces poverty. It is purposeful to invest some vegetable plants in agriculture as crops. Problems of safety of raw materials usage are also ignored.

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HYDRAULIC JUMP IN A PIPE CANAL

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Keywords: hydraulic jump, pipe, slope.

Abstract
In different sections of gravity pipelines non-pressure and pressure mixed modes of fluid flow may occur. Transition from pressure mode to non-pressure one occurs when the curve of fluid surface is smoothly changed, while transition from non-pressure mode to pressure one is accompanied by rupture of the surface curve which from physical point of view is similar to the phenomena of hydraulic jump developing in open canals.
This paper intends to consider conditions under which a hydraulic jump develops, types of jumps and places where they occur, theoretical method of finding the relation between conjugate depths and to reveal its peculiarities, as well as its impact on the hydraulic regime of the pipeline operation.

1. Introduction
In the professional literature, the hydraulic jump has been little studied theoretically and, in particular, experimentally [1-3]. Possible cases of hydraulic jump formation depending on a flow change developed from the head unit and the magnitude of the buckup pressure at the lower end of the pipeline have not been studied. There are not studies on air entrainment by fluid stream depending on the jump type. Prof. Dikarevski suggested the initial depth of the jump to take equal to normal depth of the non-pressure section, which is obvious omission [4].
The present paper intends to built a simple theory of hydraulic jump occurrence in a closed canal. To define types of hydraulic jumps, ideas on depths of a boundary jump and conjugate jumps have been introduced.

2.Results and discussion
The gravitational water conduit belongs to the class of long pipelines of which hydraulic calculation making values of power local losses and kinetic energy losses are negligible small compared with power losses on transportation. On the entire length of a pipe of gentle slope (Fig.1) the flow will be pressure one, if the flow feeding its headworks is not smaller than gravitation flow of the conduit

$$Q \geq Q_g;\ \left(Q_g = \sqrt{H_0/S}\right) \qquad (1)$$

where S is the hydraulic resistance of the pipe.
If $Q < Q_g$ and there is a h_0 overpressure at the lower point of the pipeline, then a mixed mode of the fluid flow is developed in the pipe, at that on the $L - L_1$ length of the upper part of the pipe a non-pressure flow is developed, and on the L_1 length of the lower part a pressure flow is developed. If the pipe's own i_0 islope is greater than critical slope corresponding the Q flow, then at the pressure flow section b2 type curve sets up known in professional literature, therefore, at the pipe's entrance section a critical h_{cr} depth sets up and on the surface – atmosphere pressure. The depth along the flow direction decreasing tends to the depth of uniform flow. Thus, at the non-pressure section flow the fluid stream is rough, and at the initial pressure section – smooth. Therefore, the fluid flow passage from non-pressure mode to pressure one, actually is a passage from rough flow to smooth one, due to which transition phenomenon is a classic hydraulic jump.

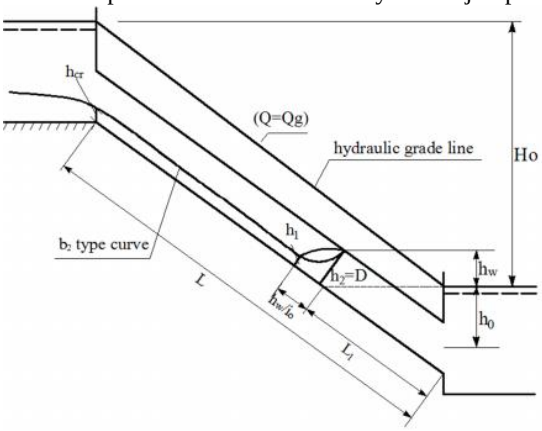

Fig.1 Mixed hydraulic mode (hydraulic free jump)

Conjugate depths of hydraulic jump developed in open-channel flow depending on the change of the downstream reach, always are conjugate, that is the increase of one of them is accompanied by a decrease of the other one, for functions of these depths jumps are equal.
It is obvious that the greater of the conjugate depths of the jump developed in the closed channel is determined by the diameter of the pipe $h_2 = D/i_0$ and is independent of the $Q < Q_g$ flow intensity. On the other hand at the lower point of the pipe h_0 pipe the change of retaining pressure is accompanied by displacement of the place where the jump occurs, during which the flow remains unchanged. Thus, if h_0 increases, the jump place travels upward along the length of the pipe, therefore, the smaller of conjugate depths of the jump grows, and the larger one remains unchanged. Therefore, in the general case of a hydraulic jump developed in a closed channel we arrive at a very important conclusion – conjugate depths can also be non conjugate, that is their jump functions can be unequal. This is an important peculiarity of the closed-channel hydraulic jump.
Another peculiarity of the closed channel hydraulic jump is its occurring place relatively large displacement depending on the flow release condition $Q_g > Q = const$ retaining pressure change and retaining pressure $h_* = const$ condition flow change.

Let us note that in this case the flow increase is accompanied with jump place upward travel and v.v. and the smaller of the jump conjugate depths can be changed in the range $(h_{cr} \dots h_0)$ between critical and normal depths. It is evident that in this interval there is a boundary h_0 depth which will be conjugation of the conjugated large depth $h_2 = {}^{D}/_{i_0}$. We now determine h_* depth by equality of its and conjugate jump functions.

The function of the jump function can be written as [5].

$$\Pi(h) = \frac{\alpha Q^2}{gA} + yA \qquad (2)$$

where A is the area of the effective cross-section, y is the distance from the gravity centre of the effective cross-section area to the free surface (Fig.2).

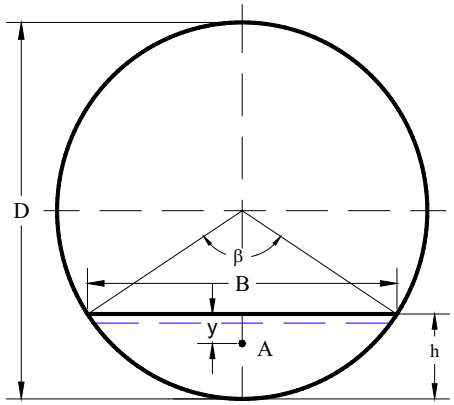

Fig.2 Cross section of the pipe

For the circular cross section, we have [6,7].

$$A = \frac{R^2}{2}(\beta - sin\beta); y = R\left(\frac{4}{3}\frac{sin^3\frac{\beta}{2}}{\beta - sin\beta} - cos\beta\right); (R = {}^{D}/_{2}) \quad (3)$$

Jump functions will have the following views

$$\Pi(h_*) = \frac{\alpha Q^2}{g\frac{R^2}{2}(\beta - sin\beta)} + \left(\frac{4}{3}\frac{sin^3\frac{\beta}{2}}{\beta - sin\beta} - cos\beta\right)\frac{R^3}{2}(\beta - sin\beta)$$

$$\Pi(D) = \frac{\alpha Q^2}{g\pi R^2} + yA \qquad (4)$$

Equalizing jump functions, we get

$$\frac{6\alpha Q^2}{gR^5} = \frac{[3\pi\beta - 3\pi sin\beta - 4sin^3\frac{\beta}{2} - 3cos\frac{\beta}{2}(\beta - sin\beta)]\pi(\beta - sin\beta)}{2\pi - \beta + sin\beta} \qquad (5)$$

It follows from the obtained equation that to each value of $Q < Q_g$ flow running by water conduit corresponds a certain β angle and therefore a certain h_* value of a boundary depth. That is, from Equation by the $Q < Q_g$ value is determined angle β

$$h_* = R\left(1 - cos\frac{\beta}{2}\right) \qquad (6)$$

The distance L_1 from the point where the jump occurs to the lower basin is determined geometrically by finding power loss of the pressure floe section.

It is not difficult to determine the above L_1 distance

$$L_1 = \frac{h_0}{i_0 - \frac{Q^2}{K^2}} \qquad (7)$$

where K is the pipe capacity.

It follows from (7) that increase of flow is accompanied by the jump point displacement to the head basin.

As far as the jump point depends on h_0 and Q moves along non-pressure section, then the boundary jump initiation probability is very small. Consequently, to determine the initial depth of the jump is reasonable to use the technique proposed in this paper which consists in finding the distance L_1 of the jump development place, then determining h_1 depth form free surface b_2 type curve equation.

Let us now differentiate to types of a jump which is developed in a closed channel. With this end in view we compare h_* boundary and conjugate small h_1 depths corresponding to to the given flow. If $h_1 > h_*$ then call the jump approached or suppressed, and if $h_1 < h_*$- remote, as in case of open channel.

It should be noted that dividing jumps into types is not an informal action. First, let us note that kinetic energy of a rough stream in a drowned section where jumps occur is smaller than that of the removed jump section and its dissipation caused by a quiet stream is more perfect. Continuous exchange of air and fluid particles takes place in the section of jump. In the place of the drowned jump feebly marked aeration is observed. Small air bubbles are developed in the lower part of the effective cross-section, then they moved to its top and with high velocity escape the fluid stream without being entrained by the stream, and dynamic equilibrium of air and fluid particles is established.

At the site of the remote jump relatively tiny particles of air in the form of bubbles are entrained by the fluid stream through the lower part of the live section, and in the upper part of the section the existing large bubbles move, gradually accelerating, in the opposite direction to the surface of the jump section. As a result removed jump possesses entrainment property.

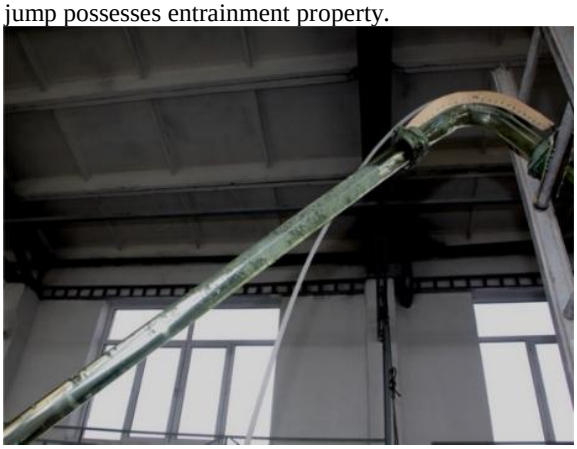

Fig.3 Siphon section of the experimental setup

Experimental study of the above described phenomena was carried out in the hydraulic research laboratory of the Institute of Water Problems and Hydro-engineering after Academician I.V. Eghiazarov, using an experimental setup made of glass pipes of 50mm diameter.

The hydraulic jump occurred in a closed canal is a dangerous phenomenon especially for open pipelines.

It causes jerks of the pipe which lead to destruction of free supports. Therefore, if a hydraulic jump is detected it is necessary to take special emergency measures to stop it. To provide smooth operation of a pipeline we suggest to equip the lower end of a pipeline by a valve which will gradually close the pipe by throttling and without changing the flow, establishing pressure flow along the entire length of the pipeline.

3.Conclusion

Conjugate depths of the closed canal hydraulic jump are conjugate only in case of boundary jump.

Depending on the value of existing overpressure at the lower end of the canal the point where a jump occurs undergoes an essential displacement along the canal.

In case of removed jump the fluid stream possesses entrainment property.

Jump elimination without the flow change is possible by throttling of the valve.

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HETERODYNE METHOD OF LINEAR MEASUREMENTS

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Keyword: range-finder, phase error, modem, modulation power

Abstract

In this article, the possibilities and a choice of ways of increase in accuracy of the linear measurement and reduction of the modulation power and their implementation in case of creation of a new high-precision laser range finder are considered. It is shown that for increase in accuracy of the linear measurements, for essential reduction of the modulation power and non-linear distortions it is necessary to realize transition to a new ways of the modulation method connected to transfer of operation point of the light modem on the average line section of the modulation characteristic of light. Dependence of amplitude-modulated light on distance is transferred from a Bessel function of the zero order J_0 (x) to a Bessel function of first order J_1(x). Such transition leads not only to reduction of power of modulation, but also non-linear distortions and dispersions of the modulation phase. At the same time, the accuracy of linear measurements increases by almost 10 times. The phase error of the linear measurements m_φ at the same time is evaluated by the 100-th shares of millimeter-0,015-0,02 mm.

Also schematic diagram of the heterodyne laser range finder allowing to realize work of a light range finder at circular polarization of light by use of the auxiliary generator – a heterodyne is provided in the offered article.

1. Introduction

In the development of high-precision optical meters and refractometers, the authors hoped to achieve too much in a very short time. Only at this stage, the technical achievements in the field of linear measurements on the modulation method are gradually approaching the required requirements for special engineering and geodetic work. In this regard, the main research should be aimed at reducing the modulation power on the linear electro-optical effect, which is potentially the only basis for the construction of modems (modulator-demodulator) of light on KDP crystals (potassium dihydrogen phosphate) for microwave range finders.

The increase in the requirements for the accuracy of linear measurements realized by high-precision light range finders, initially refers to the error in determining the residual part of the phase cycle with an error m_φ = 0.01-0.02 mm, which is possible by reducing the nonlinear distortions and phase variations of the modulation by 10 times.

The well-known high-precision light-measuring devices Geomensor 204 [1] and ДВСД-1200 [2] were provided the error in determining the residual part of the phase cycle, respectively, equal to 0.2 and 0.1 mm, but under real conditions, 0.35 and 0.25 mm were provided. Now the introduction of new solutions in the basis of the construction of microwave light range finders it is possible to construct the light range finders that provide an error in determining m_φ= 0.01-0.02 mm [3].

2. Methodology

At the first stage, the formation of two antiphase synchronously incoming optical signals in electro optical crystals of the light modem as a more accurate method for determining m_φ in the light range finder ДВСД-1200М at two photodetectors reached the value m_φ = 0.05 mm [4]. Periodic arrival of two antiphase signals to one photodetector in the light range finder CD-1200 was made possible to obtain m_φ = 0.03-0.035 mm [5].

Subsequent decrease in the value of m_φ is possible with a decrease in the modulation power, which at a frequency of 1200 MHz for the optimal modem on the compensation method is 160-180 W. This value is much more than the average permissible power for KDP crystals installed in the microwave modulators of the light range finders. Although the average value of the microwave power supplied to the modem of light does not exceed the permissible value of the power on the crystals, the applied microwave power of 150-160 W is a decisive factor, since during a pulse duration of 40 microseconds, it breaks the crystal structure, which is manifested by the presence of phase variations On the front of the modulated wave. At different areas of the light wave area, the modulation phase is different, and significantly. At a distance of 10-15 mm from the axis of the light flux, the phase difference leads to a spread in the linear value of 0.15-0.2 mm [6].

In this connection, a decrease in the microwave power on the crystals while maintaining the modulation efficiency to reduce the error m_φ = 0.01-0.02 mm becomes the main task. In this paper, we consider the possibilities of identifying and choosing ways to reduce the modulation power and their implementation in the construction of a new light range finder.

At microwave frequencies, a significant reduction in modulation power and nonlinear distortion is possible by switching to new manifestations of the modulation method associated with the transfer of the modem operating point from the onset of the modulation characteristic of light to the mid-line section. In this case, the dependence of the amplitude of the modulated light on

the distance is transferred from the Bessel function J_0 (x) to J_1 (x). Such a transition, in addition to reducing the modulation power, also reduces the nonlinear distortions and the spread of the modulation phase by a factor of 10.

For the function J_1 (x), the relative intensity of light is represented as the expression [7]

$$\tilde{I}/I_0 = \frac{1}{2}\left[1+2J_1\left(2\pi\frac{U}{U_\pi}\mathrm{Cos}\frac{2\pi D}{\lambda_m}\right)\cdot\mathrm{Cos}(\omega t+\varphi)\right] \quad \textbf{(1)}$$

Where U/U_π – power mode of the light modem; $U/U_\pi = \sqrt{P/P_\pi}$, λ_m – wavelength of the light modulation, D – Measured distance, ω– frequency modulation, φ – the phase depending on D.

According to expression (1), the ratio cannot have a value greater than one, i.e. at the maximum value of the function $J_1(x) = 0.58$ take place a over modulation. In this mode, the ratio $U / U_\pi = 0.3$ and most of the power consumed in the quadratic section depending $\tilde{I}/I_0$ on the U/U_π.

In mode $U/U_\pi = 0.3$, the modulation power reduction compared with the diphase mode, when $U/U_\pi = 0.54$ is the $(0{,}54/0{,}3)^2 = 3{,}2$ time. In practice, by a certain decrease in the ratio from one, which does not effect on the value of m_φ, it is possible to obtain a significant decrease in the modulation power. In mode $U/U_\pi = 0.18$ providing a ratio of $\tilde{I}/I_0 = 0.98$, when J_1 (x) = 0.48, the microwave power reduction is $(0{,}54/0{,}18)^2 = 9$ times

To estimate the possible decrease in power from an increase in the light intensity or modulation wavelength, we consider the differential of expression (1)

$$\frac{\partial\bar{I}}{\partial D} = -\frac{\gamma^2 I_0}{\lambda_m}\pi^2\frac{U}{U_\pi}\cdot J_1\left(2\pi\frac{U}{U_\pi}\mathrm{Cos}\frac{2\pi D}{\lambda_m}\right)\mathrm{Sin}\frac{2\pi D}{\lambda_m}$$

(2)

Values I_0, U/U_π and λ_m equally affect on the magnitude m_φ. By replacing the ratio U/U_π by $\sqrt{P_i/P_\pi}$, it is obtained that an increase in the light intensity on *n* time is possible to reduce the power of P_m on n^2 time. This is an effective way to reduce P_M and a small receiving optics increases the intensity of the receiving light by 2 or more times. As a result, a total power reduction of about 36 times is possible.

From the effective ways of reducing the modulation power, it is also necessary to stop the possibility of a buffer resonator providing a decrease in the power P_m by at least two times. Thus, taking into account the overall reduction, the total power is equal $P_i = 160/36 = 4{,}4$ W. If we also take into account the losses in the microwave circuits, we can take Pm = 5 W. With such powers, it is possible to combine the modem of light with an auto generator, since the frequency of the auto generator during the supply of the modulation pulse power to the modem crystals will not have appreciable departures and the accuracy of determining the modulation frequency will increase. Under such conditions, it makes sense to reduce the modulation frequency to 600 MHz, since the need for Optical delay line (ODL) is eliminated and the operation of the photomultiplier in the frequency conversion mode is facilitated. From the reduction in the frequency from 1200 to 600 MHz, the modulation power will be further reduced 2 times and becomes equal to Pm = 2.5 W.

However, for the operation of a light range finder in a continuous mode, such a reduction does not solve anything, because continuous mode of the modem is not yet possible, and heating the crystals at powers below 5 Watts no longer leads to noticeable errors.

Now let us consider the possibility of realizing an effective version of the receiving part of a heterodyne light range finder.

Since the modulation power is reduced, the departure of the resonant frequency of the modem is insignificant, i.e. it is possible to combine the modem with an auto generator with a smooth change in the modulation frequency, which allows us to exclude the strain-optical ODL.

With a smooth change in the modulation frequency, a heterodyne frequency must be synchronously changed so that the difference frequency remains in the low-frequency band. The difficulties of such a solution in matching the photocathode photomultiplier, as a complex resistance with the microwave field of the heterodyne resonator.

The second version of the application of the ODL at fixed frequencies of 1200 MHz at modulation powers of 5 W using electro optical frequency conversion without a phase plate by λ/2. For this variant, the second light modulator should operate in a nonlinear mode, so that the optical frequency conversion takes place, i. e. it requires high power for the heterodyne.

It is also possible to use the ODL at a frequency of 600 MHz with the use of a phase plate of λ/2 only to reduce the length of the ODL 2 times, using a microwave field on the photocathode of a photomultiplier for the photo heterodyning mode [8]. Two or three variants of matching the photocathode of a photomultiplier with the output of a heterodyne resonator are known.

With small diameters of the photocathode, the photomultiplier is installed inside the inner conductor

of the coaxial resonator. On the photocathode, the microwave field E of the resonator passes through the hole at the end of the inner conductor along the path of the receiving light. The efficiency of such control of the photocurrent is small, because a small part of the electric field E of the capacitive gap of the resonator is used.

At large diameters of the photocathode, the photomultiplier is mounted against the end of the inner conductor of the resonator and the matching is carried out by the photomultiplier, in which the constant component of the photomultiplier current decreases about 2 times.

It is more effective when the photocathode behind the resonator end, and the microwave field E of the resonator on the photocathode is fed through a capacitive connection.

With all the indicated variants of application of the field E of the heterodyne resonator to the photomultiplier photocathode, the areas of action of the field E and the light flux on the photocathode are almost not combined, and therefore the heterodyning efficiency is inadequate. In order to achieve this alignment, it is necessary to act on the photocathode of the photomultiplier in the magnetic field H of the heterodyne resonator.

The essence of this variant is given in the principle of the operation of the heterodyne light range finder, the functional diagram of which is shown in Fig.1.

The radiation of a linearly polarized laser 1 ЛГН-207А through mirrors 2 is directed to the light modem 4 at the output of which a phase plate 3 on λ/4 is mounted to convert linearly polarized light into circular light [9]. The use of a circularly polarized light source is not desirable, because after mirrors 2, the circular polarization becomes elliptically polarized, which causes the modem operating point to shift to the modulation characteristic of light.

The modulated light from the output of the modulation channel 5 of the light modem 4 is applied to a small transmission optics 6 reducing the opening of the laser beam approximately 2 times. After the mirror-lens reflector, the receiving light by the optics 7 is assembled into a parallel beam of 1.5 mm diameter and through the phase plate on λ/2 8 it is directed to the light demodulation channel 9. The demodulated light after the analyzer 10 by mirrors 11 is directed to the photo electronic multiplier (PEM) 14. The degree of demodulation is determined by the position of the modem in the path of light. By moving the modem

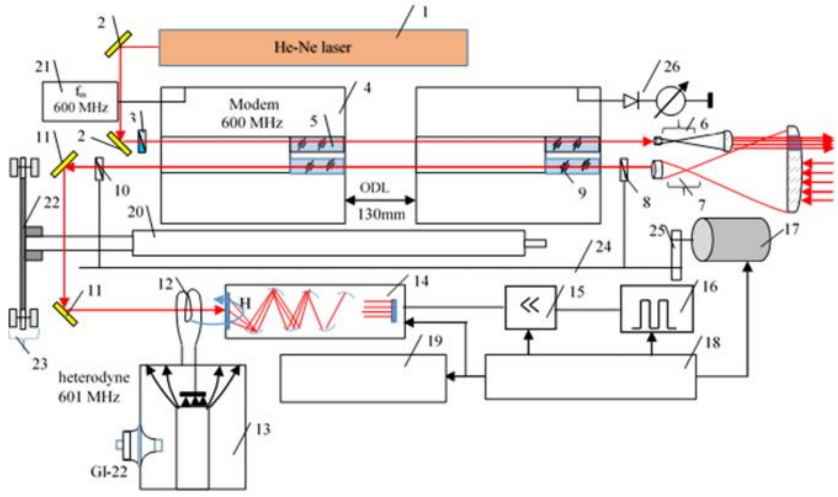

Fig.1 Functional scheme of heterodyne ligth rangefinder

within 130 mm, the position of the full demodulation of the light is found which corresponds to the point M indicated in Fig. 2, where the oscillation component of the modulation frequency is absent at half the intensity.

If within the movement of the modem the position of the point M does not occur, then by turning the phase plate λ/2 by 45º and the analyzer by 90º, the position of the point M is again sought for the minimum of the output signal of the PEM 14, the graph of which is shown in Fig.3.

To operate the photomultiplier in the photo heterodyning mode, an auto generator 13 is installed at a frequency of 601 MHz on a pulse microwave triode GI-22.

At the output of the resonator of the auto generator, through a capacitive transition in the form of a coil, a magnetic field loop 12 is established directly on the photocathode. The magnetic lines are focused on the axis of the loop in the form of a circular ring and penetrate the photocathode at the point where the light energy enters. The work of the exit of electrons from

the photocathode is the sum of the energy of the magnetic lines of force and the amplitudes of the oscillations of the light intensity. For the exit of electrons from the photocathode, the energy of the magnetic lines of force and the amplitudes of the oscillations of the light intensity are summed. Since the release of electrons on their movement, the frequencies of 600 and 601 MHz influence. At the output of the PEM anode, a difference frequency of 1 MHz is generated, whose amplitude depends on both the light intensity and the current through the loop 12. By choosing the distance between the photocathode of the photomultiplier and the loop, the maximum photo-heterodyne efficiency is established. The output signal of the photomultiplier is amplified in the resonant amplifier 15 and in the form of oscillations with a frequency of 1 MHz in a pulse with a duration of 40 microseconds is fed to the screen of the indicator 16. The repetition rate of pulses is increased to 1 kHz and this allows to estimate the operation of the MF generator and the scintillation in the reception light. The repetitions frequency of pulses is increased to 1 kHz and this allows us to estimate the operation of the microwave generator and the scintillation in the reception light.

If there are harmonics in the oscillator, amplitude emissions are observed at the vertices of pulses with a duration of 40 microseconds, and when the light is scintillated in the reception light for a period of 1 kHz, the amplitude of the signals changes.

Moving the modem as the SLS determines the number of pulses that are formed at the outputs of the photo-light diodes (PLD) 23 and are set by the slotted disk 22 directly connected to the screw of the modem movement 20. The rotation of the analyzer 10 is connected with the rotation of the phase transition by λ/2 through the axis 24, whose motion through the reducer 25 is realized by the motor 17.

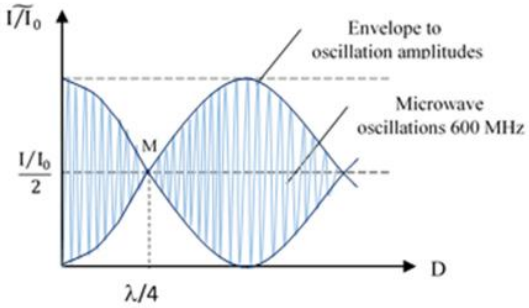

Fig.2 Dependence of the amplitude of the oscillation of light on the distance

The modem of light 4 is connected to the microwave generator 21 through a flexible coaxial cable. The position of the resonance of the modem is determined through the detector circuit 26. In the region where there is no modulation frequency, point M in Fig. 2, the sensitivity of the indicator device rises sharply in the form of the curve in Fig. 3 in the region of point M.

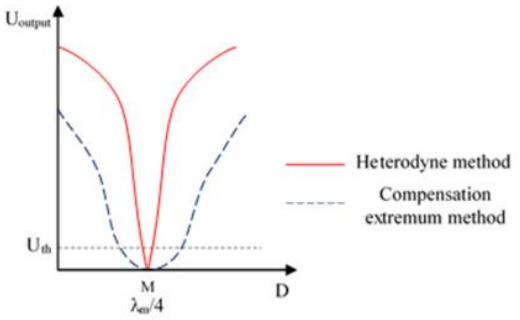

Fig.3 Dependence of the amplitude of the observed signal on the indicator

3. Results

On the heterodyne method, the calculated sensitivity is 2 times higher than on the diphase method. However, in the graphical comparison of the steepness of the heterodyne method with the diphase method, an increase in sensitivity is not noticeable. In Fig. 3, near the point M, the curve is more sensitive due to the amplitude of the heterodyne signal, the increase in the signal-to-noise ratio and for the large amplification after the photomultiplier in the resonant amplifier, which are not available on the diphase method.

In addition, the presence of phase spreads and nonlinear distortions do not allow on the diphase method to obtain the ultimate measurement accuracy of the modulation method.

For this period, the main nodes of the new light range finder are implemented - the microwave oscillator of scale oscillations, the light modem, heterodyne, ODL and certain electronic units. Work on the completion of a new optical range meter at the stage of the formation of a receiving path of a light range finder based on the process of photo heterodyne.

4. Conclusion

Figure 3 also shows the operation of the light-range finder, when the minimum of light is determined by fixing the receiving points M and N at the threshold level $U_{th.}$, which corresponds to the operation of the light-range finder at the compensation extremum method. Comparison of these curves indicates that results that are more accurate can be obtained in the photo heterodyning mode.

Although the calculated accuracy of phase fixation during the operation of the light-range finder in the photo heterodyning mode is significantly increased, in practice it is possible to provide an accuracy increase of 5-6 times, which leads to an accuracy of the phase measurement 0.015-0.02 mm.

5. Acknowledgments

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METHOD OF RECTANGULAR FINITE ELEMENTS WITH QUADRATIC LAW OF VARIATION OF COMPONENTS' DISPLACEMENTS ALONG THE CIRCUIT FOR SOLVING PLANE PROBLEMS OF MOMENT THEORY OF ELASTICITY

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Keywords: finite elements, displacements, quadratic programming, plane problem, cosserat theory of elasticity.

Abstract

Method of rectangular finite elements with quadratic law of variation of components' displacements along the circuit for solving plane problems of moment theory of elasticity is given. The first version of the cosserat theory of elasticity is used, wherein the elastic behavior of the new isotropic model is characterized by four elastic constants. Conditions for determining rotation and displacement of an element as an abstract rigid body, which do not act on the stress state of the element are derived.

1. Introduction

The method of rectangular finite elements with a quadratic law of variation of components' displacements along the circuit is considered to solve the plane problems of the moment theory of elasticity. The first version of the moment theory of elasticity is used, wherein the elastic behavior of the new isotropic model is characterized by four elastic constants [1,2]: E, ν, l and η, of which E is Young's modulus, ν - Poisson's coefficient, l and η - new constants of the material (l has the length dimension, η - dimensionless constant of Poisson's ratio type). Conditions for determining the rotation and displacement of an element as an abstract rigid body are derived, which do not act on the state of stress of the element.

2. Methodology

Let's designate the symmetrical component of the tangential component through, $\tau_c = (\tau_{xy} + \tau_{yx})/2$ and the antisymmetric component through. $\tau_a = (\tau_{xy} - \tau_{yx})/2$.

The plane problem of the moment theory of elasticity in the absence of volume forces and moments reduces to the integration of a system of equations in stresses σ_x, σ_y, τ_c, τ_α, moment stresses μ_x, μ_y [1, 2].

$$\begin{cases} \dfrac{\partial \sigma_x}{\partial x} + \dfrac{\partial \tau_c}{\partial y} - \dfrac{\partial \tau_a}{\partial y} = 0, \\ \dfrac{\partial \sigma_y}{\partial y} + \dfrac{\partial \tau_c}{\partial x} + \dfrac{\partial \tau_a}{\partial x} = 0, \\ \dfrac{\partial \mu_x}{\partial x} + \dfrac{\partial \mu_y}{\partial y} + 2\tau_a = 0 \end{cases} \quad (1)$$

and compatibility conditions

$$\begin{cases} \mu_x = 2l^2\{\dfrac{\partial \tau_c}{\partial x} - \dfrac{\partial}{\partial y}[\sigma_x - \nu(\sigma_x + \sigma_y)]\}, \\ \mu_y = 2l^2\{\dfrac{\partial}{\partial x}[\sigma_y - \nu(\sigma_x + \sigma_y)] - \dfrac{\partial \tau_c}{\partial y}\}, \\ \dfrac{\partial^2 \sigma_x}{\partial y^2} + \dfrac{\partial^2 \sigma_y}{\partial x^2} - \nu\nabla^2(\sigma_x + \sigma_y) = 2\dfrac{\partial^2 \tau_c}{\partial x \partial y}, \\ \dfrac{\partial \mu_x}{\partial y} = \dfrac{\partial \mu_y}{\partial x} \end{cases} \quad (2)$$

at the appropriate boundary conditions.

In [2] to determine the moment stresses μ_x и μ_y, it is proposed to use the following formulas:

$$\mu_x = \frac{2El^2}{1+\nu}\gamma_x, \quad \mu_y = \frac{2El^2}{1+\nu}\gamma_y, \quad (3)$$

where γ_x и γ_y - the curvatures, wherein

$$\gamma_x = \frac{\partial \omega_{\bar{z}}}{\partial x}, \gamma_y = \frac{\partial \omega_{\bar{z}}}{\partial y}, \omega_{\bar{z}} = \frac{1}{2}(\frac{\partial v}{\partial x} - \frac{\partial u}{\partial y}), \quad (4)$$

whereas

$$\gamma_x = \frac{1}{2}(\frac{\partial^2 v}{\partial x^2} - \frac{\partial^2 u}{\partial x \partial y}), \gamma_y = \frac{1}{2}(\frac{\partial^2 v}{\partial x \partial y} - \frac{\partial^2 u}{\partial y^2}). \quad (5)$$

Using Cauchy relations

$$\varepsilon_x = \frac{\partial u}{\partial x}, \quad \varepsilon_y = \frac{\partial v}{\partial y}, \quad \gamma_{xy} = \frac{\partial u}{\partial y} + \frac{\partial v}{\partial x} \quad (6)$$

and the curvature expressions (5), as well as the values for the moment stresses (3), we find

$$\mu_x = \frac{2El^2}{1+\nu}\left(\frac{\partial^2 v}{\partial x^2} - \frac{\partial^2 u}{\partial x \partial y}\right),$$

$$\mu_y = \frac{2El^2}{1+\nu}\left(\frac{\partial^2 v}{\partial x \partial y} - \frac{\partial^2 u}{\partial y^2}\right). \quad (7)$$

Using the generalized Hooke's law

$$\varepsilon_x = \frac{1}{E}(\sigma_x - \nu\sigma_y), \ \varepsilon_y = \frac{1}{E}(\sigma_y - \nu\sigma_x),$$

$$\gamma_{xy} = \frac{2(1+\nu)}{E}\tau_c \quad (8)$$

and the Cauchy relations (6) for tensions σ_x, σ_y and τ_c, we obtain

$$\begin{cases} \sigma_x = \dfrac{E}{1-\nu^2}\left(\dfrac{\partial u}{\partial x} + \nu\dfrac{\partial v}{\partial y}\right), \\ .\sigma_y = \dfrac{E}{1-\nu^2}\left(\dfrac{\partial v}{\partial y} + \nu\dfrac{\partial u}{\partial x}\right). \\ \tau_c = \dfrac{E}{2(1+\nu)}\left(\dfrac{\partial u}{\partial y} + \dfrac{\partial v}{\partial x}\right). \end{cases} \quad (9)$$

Assuming that within the final element the normal stresses σ_x^s and σ_y^s vary linearly, and the symmetrical component of the shear stress τ_c^s is constant, the general expressions for the stress components are given in the form

$$\sigma_x^s = \alpha_1 + \alpha_2 y, \sigma_y^s = \alpha_3 + \alpha_4 x, \tau_c^s = \alpha_5 \quad (10)$$

which contain five arbitrary parameters

$\alpha_i, i \in \{1,2,...,5\}$.

Substituting expressions (10) into differential equations of equilibrium (1), we obtain the remainder terms containing the antisymmetric component of the tangential stress, which determines the rotation of the element as an perfectly rigid body [2] and does not affect the state of stress of the element.

Taking

$$\tau_a = 0, \quad (11)$$

from the equilibrium equation (1), according to the ratio (10), the first two equations are identically satisfied, and from the third equilibrium equation (1) with regard to (7) we obtain:

$$\frac{\partial}{\partial x}\left(\frac{\partial^2 v}{\partial x^2} - \frac{\partial^2 u}{\partial x \partial y}\right) + \frac{\partial}{\partial y}\left(\frac{\partial^2 v}{\partial x \partial y} - \frac{\partial^2 u}{\partial y^2}\right) = 0. \quad (12)$$

We submit this equation in the form

$$\frac{\partial^3 v}{\partial x^3} - \frac{\partial^3 u}{\partial x^2 \partial y} + \frac{\partial^3 v}{\partial x \partial y^2} - \frac{\partial^3 u}{\partial y^3} = 0. \quad (13)$$

Equation (13) is satisfied if

$$\frac{\partial^3 v}{\partial x^3} + \frac{\partial^3 v}{\partial x \partial y^2} = 0 \ \text{ и } \ \frac{\partial^3 u}{\partial x^2 \partial y} + \frac{\partial^3 u}{\partial y^3} = 0. \quad (14)$$

Integrating equations (14), we obtain

$$\begin{cases} v\left(x^2 + y^2\right) = c_1 x^2 y^2 + c_2 x y^2 + c_3 y^2 + c_4 y + c_5 \\ u\left(x^2 + y^2\right) = c_6 x^2 y^2 + c_7 x^2 y + c_8 x^2 + c_9 x + c_{10} \end{cases}. \quad (15)$$

It is proposed

$$c_3 = c_4 = c_5 = c_8 = c_9 = c_{10} = 0. \quad (16)$$

$$c_1 = c_2 = c_5 = c_6 = c_7 = 1. \quad (17)$$

Conditions (16) determine the displacements of an element as an abstract rigid body and do not act on the state of stress of an element.

As the equilibrium equation is obtained from a unit element, and respectively the sides of the element in the direction x are equal a, and in the direction y are equal b, so denoting by $\xi = \frac{x}{a}$, and by $\eta = \frac{y}{b}$, from equalities (15) we obtain

$$\begin{cases} u\left(\xi^2 + \eta^2\right) = \xi^2\eta^2 + \xi\eta^2 + \eta^2 + \eta \\ v\left(\xi^2 + \eta^2\right) = \xi^2\eta^2 + \xi^2\eta + \xi^2 + \xi \end{cases}. \quad (18)$$

3. Conclusion

Substituting the values (18) into expressions (9) and (3), we find the values of the normal stresses σ_x and σ_y, the symmetric part of the tangential stress τ_c and the moments of the tension μ_x and μ_y.

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ABOUT WATER RESISTANCE OF PLASTERS Of TATEV MONASTERY COMPLEX AND GOVHAR AGHA MOSQUE IN SHUSHI

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Keywords: restoration, monument, plaster, composition, water resistance.

Abstract
The paper presents the results of studies of external and internal plaster samples taken from the surfaces of stone constructions of the auxiliary buildings of Tatev monastery complex , as well as the Iranian Govhar Agha mosque in Shushi.
Some of the basic physical and mechanical characteristics, including water absorption and solubility of plasters have been studied. Electron microscopic, X-ray and petrographic studies, as well as chemical analysis of ancient plasters were carried out.
The compositions of the investigated mortars are revealed. The dependences of the main qualitative indicators on structural features and mineralogical compositions of plasters are determined and some of the main causes of their damage are formulated.

1. Introduction
For the preservation of the cultural and historical values of ancient structures, modern rules of restoration do not allow an active interference with the appearance of the monuments. The restoration process supposes performances of the most necessary volume of strengthening and conservation activities using the most modern protective facilities and materials [1].
The life of architectural monuments, except the constructive solutions, largely depends upon the materials used. Since ancient times, thoroughly developed rules had strictly regulated the technology of preparation and use of building materials, including raw materials and formulation of compounding solutions, as well as their technologies. Despite this, extensive experience in the survey of ancient temple stone structures, including Armenian ones, showed that the most vulnerable part is the mortar component of the masonry [2].
For optimal selection of materials and formulations of new restoration solutions, it is necessary to take into account the conditions of their service in various structural components of the ancient structure. At the same time, they should be in reasonable combination with other used new materials, and also must correspond to the "old" mortars for the raw materials and basic quality characteristics. During the survey of the state of monuments and for the establishment of initial parameters of the used mortars, according to modern requirements a comprehensive study of mortars is also carried out.

2. Methodology
The surveys were carried out by means of preliminary visual inspection of the structure materials with a choice of places for taking samples and their subsequent detailed experimental study. The actual qualitative and quantitative characteristics of the "ancient" mortars were detected using a research technique based on an analysis of the results of determining certain basic properties - chemical composition, petrography, as well as micrography and X-ray.

3. Results
The ancient external and internal wall plasters were studied, samples of which were taken from two different monuments – from the auxiliary structure of Tatev monastery complex, as well as Govhar Agha mosque in Shushi (Fig. 1 and Fig. 2). The researches were carried out in the laboratories of the construction organization ,,Horizont 95'', in the Institutes of Chemical Physics and Geological Sciences of the Academy of Sciences of the Republic of Armenia. Some of the basic properties of plasters (Table 1), their chemical composition (Table 2) and petrography (Table 3) have been studied. Micrographs and X-ray photographs of the external and internal wall plasters of the auxiliary buildings of Tatev monastery complex are presented respectively in Fig. 3, and in Fig. 4 - the internal and external plasters of the Govhar Agha mosque.
Air-setting lime and local natural sands were used in the plasters of the auxiliary structure of Tatev monastery complex, which were formed as a result of natural weathering of slag rocks. Mortars have the following compositions: 1 part of lime and 2 parts of a mixture of mid size- and fine-grained sand for internal plaster and 1 part of lime and 1 part of fine-grained sand mixture for external plaster. Carbonate content of calcareous solutions is 33 and 45 percent, respectively. Free calcium hydroxide in the composition is missing. Carbonized calcium silicates and silicon oxide, recorded by X-ray photographs, indicate that the hydrate of calcium oxide from the calcareous solutions had interacted with siliceous components of the applied natural sands for a very long time. The main factor of longevity and air

resistance of lime mortars that do not contain pozzolanic additives appears to be an increased content of lime.

Fig. 1 Auxiliary buildings of Tatev monastery complex (External and internal plaster)

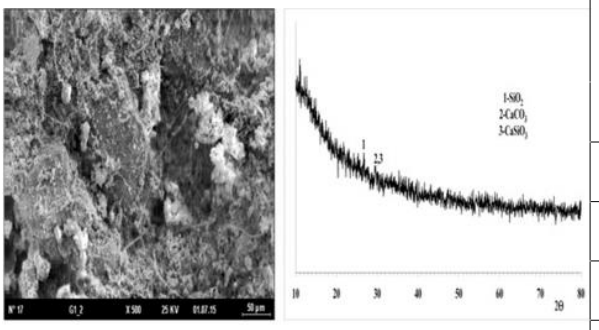
Fig. 2 The Iranian mosque Govhar Agha in Shushi (Interior and exterior plaster)

Table 1 Some physical and mechanical characteristics of plasters

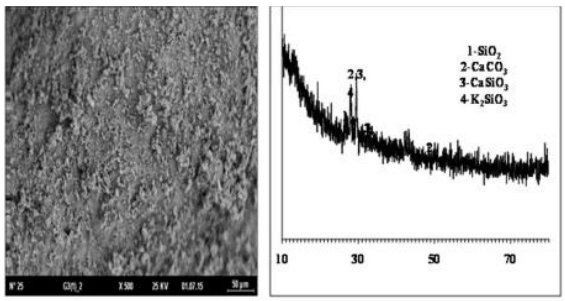
NN / п.п.	Reference designation	Material; Structure	Density, kg / m^3	Compressive strength, MPa	Solubility,% by weight	Water absorption,% by weight
1.	T_{ext}	External plaster of an auxiliary building of Tatev monastery complex	1695	1.2	35	10.5
2.	T_{int}	Internal plaster of an auxiliary building of Tatev monastery complex	1890	1.8	20	12.0
3.	Sh_{ext}	External plaster of the Iranian mosque Govhar Agha in Shushi	2570	5.7	-	11.1
4.	Sh_{int}	Internal plaster of the Iranian mosque Govhar Agha in Shushi	1870	3.5	-	20.0

Table 2 Results of chemical analysis of plasters

Number of samples accord. To Table 1	Reference designation	Content of components,%												
		SiO_2	Al_2O_3	TiO_2	Fe_2O_3	FeO	CaO	MgO	CO_2	P_2O_5	K2O	Na_2O	H_2O	SO_3
1.	T_{ext}	25.0	18,5	0.56	2.23	0.01	28.0	3.25	20.6	0.11	0.21	0.32	0.01	-
2.	T_{int}	18.0	10.2	0.42	2.4	0.02	41.4	2.06	22.4	0.08	0.09	0.32	1.33	-
3.	Sh_{ext}	9.60	2.80	0.05	1.85	0.14	30.4	1.50	2.50	0.04	0.40	0.50	5.20	45.0
4.	Sh_{int}	8.59	3.84	0.10	1.45	0.14	43.7	11.4	34.9	0.18	0.60	0.60	0.70	2.30

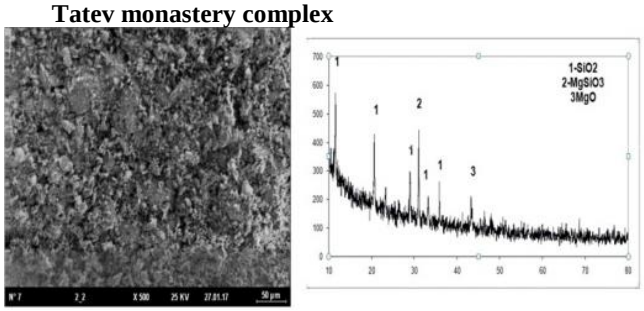


a – Text

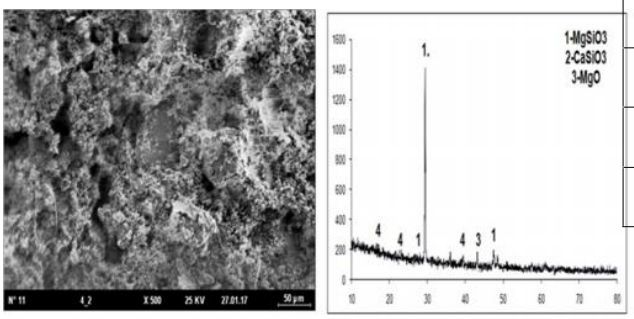


b – Tint

Fig. 3 Micrographs and X-ray patterns of external (a) and internal (b) plasters of auxiliary buildings of Tatev monastery complex



a-Shext



b-Shint

Fig.4 Micrographs and X-ray patterns of external (a) and internal (b) plasters of Iranian Govhar Agha mosque in Shushi.

Table 3 Results of petrographic analysis of plasters

Minerals	The quantitative content of minerals,% (numbers and designations of sample solutions according to Table 1)			
	N1 (T_{ext})	N2 (T_{int})	N3 (Sh_{ext})	N4 (Sh_{int})
Heavy fraction				
Magnetite	25.0	25.0	50.0	50.0
Hematite	25.0	25.0	-	-
Hypersthene	-	-	-	1.0
Diopside	0.5	-	4.0	2.0
Augite	17.0	14.0	10.5	10.0
Hornblende	10.0	12.0	12.0	12.0
Basaltic hornblende	20.0	20.0	20.0	18.0
Iron hydroxides	1.0	1.5	1.0	1.0
Actinolite	1.0	2.5	-	1.5
Epidote	Single grains	Single grains	1.0	2.0
Zircon	-	-	1.0	0.5
Biotite	Single grains	Single grains	0.5	1.5
Tremolite	0.5	-	-	0.5
Light fraction				
Weathered grains	65.0	45.0	54.0	30.0
Volcanic glass	25.0	40.0	35.0	Single grains
Quartz	-	-	1.0	-
Plagioclase	10.0	15.0	10.0	1.0
Muscovite	-	Single grains	-	1.0
chrysotile	-	-	-	68.0

The formation of calcium silicate is a primary process of long hardening of lime mortar in air:
Ca (OH) $_2$ + SiO_2 • aq → CaO · SiO_2 • aq.. During the further existence of the mortar, the residual free calcium hydroxide and the new formations under the influence of carbon dioxide in the air (CO2) were carbonized according to the following equations: $Ca(OH)_2+CO_2→CaCO_3+H_2O$;

$CaO{\cdot}SiO_2$ aq.+CO_2→$CaCO_3$+SiO_2 aq.

Calcium carbonate (CaCO3), as it is known, strengthens the mortar, and the silicic acid gel (SiO_2aq.) weakens it and can even lead to a complete loss of cohesion and strength. With an excessive amount of bindings from the latter, under the impact of carbon dioxide (CO_2), the opal gel of silica (SiO_2aq.) can be gradually dehydrated, crystallized and converted to chalcedony (SiO_2), i.e. return to crystalline anhydrous state. However, judging from the solubility of the studied plasters (35 and 20 per cent, respectively), this unfortunately did not happen.

The reason for the destruction of the plastering of the auxiliary structures of Tatev monastery complex is the lack of water resistance of the lime mortars . In mortars not containing hydraulic additives in case of abundant, permanent and long-lasting moistening, (annealed clay, natural pozzolans, etc.), the solubility of the materials increases, as water in combination with carbon dioxide has a destructive effect on calcium.

Wherein, calcium carbonate $CaCO_3$ is converted into acidic calcium bicarbonate Ca $(HCO_3)_2$, which, as it is well known, has a solubility of 100 times higher than calcium carbonate. In addition, dispersed straw-reinforcment, used in external plaster, during the accumulation of moisture on the surface becomes an additional cause of cracking and subsequent destruction of the plaster.

Plaster, taken from the internal surface of the mosque Govhar Agha in Shushi, was prepared with the use of three parts of gypsum binder (SO_3- 45%, CaO-30.4%) with the addition of one part of clay, slate and gneissic mixtures. The content of anhydrite can easily be seen on a micrograph in the form of twin crystals of a parallel-fibrous structure.

The material of the internal plaster is qualitatively similar to alabaster (clay gypsum), in which one-quarter of the quantitative composition are fine, very fine and powdery particles, mainly from chrysotile, which is a part of the serpentine group. The X-ray photographs of the solution showed the peaks of crystalline anhydrous oxides of silica (SiO_2) and magnesium (MgO). Layered magnesium silicates that make up the serpentine group $Mg_3(Si_2O_5)(OH)_4$ are the main minerals of clay shales and gneisses [3].

Additional components of gypsum plaster could get into the composition during their conscious introduction, as it is accepble in lime mortars, or with their initial content in the original rock in the form of natural contaminants.

Precisely, this kind of slightly burnt gypsum with variable contents of natural impurities were found in plasters, widely used for paintings in the palaces, temple structures and tombs in ancient Egypt.

And there are many such examples in other cultures of different historical periods. Starting from the second half of the 15th century, the so-called "kundal" raised pattern was a very popular ornamental decoration of mosques' interiors. It was a relief pattern of red clay with an admixture of vegetable glue and gypsum, which bulges were covered with gildings, and the background was filled with small floral patterns, painted blue. It is known that plaster for such finishing was applied with gypsum with a natural admixture of clay, as well as a certain amount of silica and carbonates. This natural material (ganch) was heated up to 130 ° C, crushed and sieved. Coarse-grained light gray ganch (tez-ganch) was used by old masters in clay mortars, and cleaned white fine-grinded ganch (gul-ganch) - in plasters for frescoes and carved knocking [4].

4. Conclusion

For plasters of Tatev monastery auxiliary structures, which do not contain pozzolonic additives, such as annealed clay, natural pozzolans, etc., the main factor of longevity and air resistance could be the increased content of lime mortar. It is known that the formation of calcium hydrosilicate is the primary process of hardening of the lime plaster in air - $Ca(OH)_2$+$SiO_2{\cdot}$aq.→$CaO{\cdot}SiO_2{\cdot}$aq. In case of further life of the plaster, the residual free calcium hydroxide and the new formations are carbonized under the influence of carbon dioxide (CO2), which is present in the air. At the same time, calcium carbonate ($CaCO_3$) is formed, which strengthened the mortar, and silica gel (SiO_2aq.) is formed as well, which weakens it and can even lead to a complete loss of connectivity and strength. With an excessive amount of bindings from the latter, under the impact of carbon dioxide (CO_2), the opal gel of silica (SiO_2aq.) can be gradually dehydrated, crystallized and converted to chalcedony (SiO_2), i.e. return to crystalline anhydrous state. However, judging from the solubility of the studied plasters, this unfortunately did not happen. Calcium carbonate (CaCO3), as it is known, strengthens the mortar, and the silicic acid gel (SiO_2aq.) weakens it and can even lead to a complete loss of cohesion and strength. With an excessive amount of bindings from the latter, under the impact of carbon dioxide (CO_2), the opal gel of silica (SiO_2aq.) can be gradually dehydrated, crystallized and converted to chalcedony (SiO_2), i.e. return to crystalline anhydrous state. However, judging from the solubility of the studied plasters, this unfortunately did not happen. In case of abundant, permanent and long-lasting moistening, the solubility of the materials increases, as water in combination with carbon dioxide has a destructive effect on calcium. Wherein, calcium carbonate $CaCO_3$ is converted into acidic calcium bicarbonate Ca $(HCO_3)_2$, which, as it is well known, has a solubility of 100 times higher than calcium carbonate.

In addition, dispersed straw-reinforcment, used in external plaster, during the accumulation of moisture on the surface becomes an additional cause of cracking and subsequent destruction of the plaster.

For the external plastering of Govhar Agha mosque in Shushi, the magnesian binder provided a more solid surface of the layer. Used as fillers of the same minerals of dolomitized limestones, from which a calcareous binder was obtained, caused the regeneration of calcite grains of the binder, that is, a gradual transition from the aggregate to the binder took place. This is due to the high density and uniformity of the structure, the good adhesion of the components and, as a consequence, the relatively low water absorption of the plaster. The usage of carbonate

fillers in the lime, in addition to the formation of a strong splice of crystals, permits usage of waste from conventional lime production (underburned lime, calcareous fine particales, over-burned lime, slagged parts). In case of absolute absence of solubility, partial destruction of the exterior plastering of the mosque Govhar Agha takes place. The mechanism of destruction is associated with the difference in temperature deformations, both for individual components of the mortar and masonry as a whole, and influence of periodic freezing, thawing, moistening, drying, and other aggressive chemical, physical and biological factors.

For the internal plasters of Govhar Agha mosque, the presence of layered minerals in the composition of the gypsum plaster determined (in case of absence of solubility) significant water absorption. Under complete absence of solubility - gypsum plaster has a significant water absorption (20%), which is due to layered minerals, which are present in its composition and during absorption of water molecules swells due to the expansion of inter-packet spaceing, but during their removal, they decrease in volume. Multiple repetitions of this process could lead to the appearance of surface cracks.

The hardening of the gypsum plaster is based on dissolution in water, hydration and subsequent crystallization into a strong splice of gypsum, containing two molecules of water ($CaSO_4 \cdot 2H_2O$). Separate particles of gypsum solution, adhering to each other and being smeared on the surface of the brickwork, do not possess a penetrating ability (diffusion). The destruction of gypsum plaster is associated with the mechanism of condensate formation on the surface of the brickwork, where moisture penetrates through the surface cracks of the plaster. Multiple freezing, defrosting, moisturizing and drying lead to the detachment and destruction of plaster. In addition, the process of destruction is also associated with the moisture of the brickwork due to the damage to the roof flashing, as evidenced by the absence of separate fragments of brickwork. Water, percolating through the interlayer of wedging lime mortar, accumulates on the interface of gypsum mortar and clay bricks, detaching from each other easily. Over time, moisture leads to the erosion of the lime solution, to fall of individual bricks, to loosening of the masonry and to acceleration of gypsum plaster further destruction process.

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ON A CLASS OF MIXED BOUNDARY-VALUE PROBLEMS OF CONTINUUM MECHANICS SOLVEABLY BY THE INTEGRAL EQUATIONS METHOD

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Abstract

This paper considers a class of mixed boundary-value problems of continuum mechanics, which closely related to each other ideological and the methodological aspects. Such problems rise in the mathematical theory of elasticity at antiplane deformation of an elastic medium, geomechanics in study of soil landslides, in the theory of steady fluid filtration in porous soils in calculation of hydraulic structures such as dams, structural mechanics, theory of stationary thermal conductivity and in other spheres. These problems as applied to the elasticity theory are formulated as follows:

1) on the half-infinite interval of the elastic layer upper face under antiplane deformation in direction of one of the coordinate axes, the constant displacement is given in the direction of the same axis; the tangential stresses are given at the subsequent finite interval and in the rest of the half-infinite interval the layer is rigidly fixed and the lower of the face layer is also rigidly fixed;

2) the same problem when in a half-infinite interval the constant displacement in the opposite direction of the coordinate axis is given instead of fixing;

3) the mixed problems presented in cases 1) and 2) are considered when the lower edge of the layer 1) free of external forces or 2) is rigidly fixed.

Using the Fourier integral transformations, solutions to the stated mixed boundary-value problem are reduced to solutions of Fredholm integral equations of the first kind. These equations admit closed, i.e. the exact solutions which are obtained by M.G. Krein method, as well as by the method of singular integral equations (SIE) or the Chebyshev of orthogonal polynomials method. In the solutions with their inherent singularities are explicit emphasized.

1. Introduction

This paper considers a class of mixed boundary-value problems of continuum mechanics, combined by a general mathematical method of research and commonality of physical ideas of their products. These problems are formulated by the mathematical theory of elasticity and the theory of steady-state filtration in the porous soils. But such and similar problems are often found in the structural mechanics, in the geomechanics, in the mechanics of composites, in the thermal conductivity and in the other areas. On the part of the theory of elasticity are considered mixed boundary-value problems for an elastic layer under the conditions of an antiplane deformation. Such problems are modeled many practical questions in geomechanics in the calculations of the stress-strain state of mountain massifs, subject to shear deformations, which in landslides of soils or earthquakes, greatly exceed other types of deformations. These problems are mathematically formulated as a mixed boundary value problems in the theory of harmonic functions and by the method of the integral Fourier transform are reduced to Fredholm integral equations of the first kind that admit exact solutions.

These solutions are constructed by the method of M.G. Krein or by the method of SIE or by the method of orthogonal polynomials and they are clearly identified the inherent characteristics of them. Thus, the class of mixed problems under discussion is directly connected with contact and mixed problems of the theory of elasticity, in which many results are summarized in [1-6].

On the part of the theory of filtration, the mixed boundary value problems under consideration arise in the calculation of hydraulic structures such as dams with a porous ground foundation [7-9].

2. Formulation of mixed boundary value problems.

Let the layer $\Omega = \{-\infty < x, z < \infty; -H \le y \le 0\}$ of the modulus of elasticity G and height H, referred to the right-handed coordinate system $Oxyz$, is in the direction Oz axis under antiplane deformation (longitudinal shear) (a basic plane Oxy). The following mixed boundary value problem for a layer Ω is considered.

I. Suppose on the upper edge of the layer Ω on the site $\omega_- = \{-\infty < x < -a;\ y = 0;\ -\infty < z < \infty\}$ constant displacement w_0 along positive axis Oz are given; the region $\omega_+ = \{a < x < \infty;\ y = 0;\ -\infty < z < \infty\}$ is rigidly fixed, on the central region $\omega_0 = \{-a < x < a;\ y = 0;\ -\infty < z < \infty\}$ tangential forces of $f(x)$ intensity are acting along negative direction Oz axis and the lower edge of the layer is free from external forces (Fig.1).

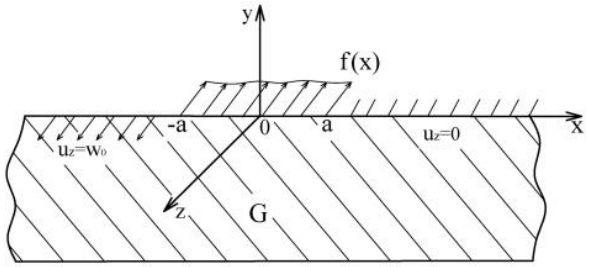

Fig.1 Elastic equilibrium of the layer at antiplane deformation for mixed boundary conditions

The governing equations of the theory of elasticity with a basic plane Oxy at an antiplane deformation consist of a single equilibrium equation and **Hooke's law**. They have the following view, respectively

$$\frac{\partial \tau_{xz}}{\partial x}+\frac{\partial \tau_{yz}}{\partial y}=0; \quad \tau_{xz}=G\frac{\partial u_z}{\partial x}; \quad \tau_{yz}=G\frac{\partial u_z}{\partial y},$$

where τ_{xz} and τ_{yz} are components of tangential stresses, $u_z=u_z(x,y)$ is a single nonzero component of the displacement along axis Oz. Introducing the function $\varphi(x,y)=G\,u_z(x,y)$, we find that this function, as well as the function $u_z(x,y)$ in the base layer $\omega=\{-\infty<x<\infty;\,-H<y<0\}$ are harmonic functions. Further, using the Cauchy-Riemann conditions, we get

$$\frac{\partial \varphi}{\partial x}=\frac{\partial \psi}{\partial y},\ \frac{\partial \varphi}{\partial y}=-\frac{\partial \psi}{\partial x} \qquad ((x,y)\in\omega). \qquad (1)$$

Introduce harmonic function $\psi(x,y)$ conjugate with $\varphi(x,y)$. Then,

$$\tau_{xz}=\frac{\partial \psi}{\partial y},\ \ \tau_{yz}=-\frac{\partial \psi}{\partial x}.$$

Obviously, the lines $\varphi(x,y)=const$ represent lines of equal displacement. The lines $\psi(x,y)=const$ represent the tangential stress trajectories. Indeed, we can write

$$\psi(x,y)=const \ \Rightarrow\ \frac{\partial \psi}{\partial x}dx+\frac{\partial \psi}{\partial y}dy=0 \ \Rightarrow$$
$$\Rightarrow\ -\tau_{yz}dx+\tau_{xz}dy=0 \ \Rightarrow\ \frac{dx}{\tau_{xz}}=\frac{dy}{\tau_{yz}}.$$

The function $\psi(x,y)$ in hydrodynamics is analog of the current function.

Now, the first mixed boundary value problem for the elastic layer Ω can be mathematically formulated as the following boundary value problem for a harmonic function $\varphi_1(x,y)=G\,u_z^{(1)}(x,y)$ in the base strip ω:

$$\begin{cases} \dfrac{\partial^2\varphi_1}{\partial x^2}+\dfrac{\partial^2\varphi_1}{\partial y^2}=0 & ((x,y)\in\omega) \\ \varphi_1(x,y)\big|_{y=-0}=w_0G & (-\infty<x<a); \\ \varphi_1(x,y)\big|_{y=-0}=0\ (a<x<\infty);\ \dfrac{\partial\varphi_1}{\partial y}\Big|_{y=-0}=-f(x)\ (-a<x<a); & \\ \dfrac{\partial\varphi_1}{\partial y}\Big|_{y=-H}=0 & (-\infty<x<\infty). \end{cases} \qquad (2)$$

The boundary value problem (2) is equivalent to the following mixed boundary value problem for the function $\varphi_1(x,y)$ conjugate to $\psi_1(x,y)$.

$$\begin{cases} \dfrac{\partial^2\psi_1}{\partial x^2}+\dfrac{\partial^2\psi_1}{\partial y^2}=0 & ((x,y)\in\omega); \\ \dfrac{\partial\psi_1}{\partial y}\Big|_{y=-0}=0 & (|x|>a); \\ \psi_1(x,y)\big|_{y=-H}=0 & (-\infty<x<\infty); \\ \psi_1(x,y)\big|_{y=-0}=\int\limits_{-a}^{x}f(s)\,ds+c_1 & (-a\le x\le a), \end{cases} \qquad (3)$$

where c_1 is a constant to be determined.

II. In the second mixed problem for the elastic layer Ω, the previous conditions on the sections ω_- and ω_0 will remain unchanged, and on the section ω_+ constant displacements w_0 are given along negative axis Oz and the edge of the layer $y=-H$ is rigidly fixed. This problem is formulated mathematically in the form of the following mixed boundary value problem for the function $\varphi_2(x,y)=G\,u_z^{(2)}(x,y)$:

$$\begin{cases} \dfrac{\partial^2\varphi_2}{\partial x^2}+\dfrac{\partial^2\varphi_2}{\partial y^2}=0 & ((x,y)\in\omega); \\ \varphi_2(x,y)\big|_{y=-0}=-G\,w_0\ signx & (|x|>a); \\ \dfrac{\partial\varphi_2}{\partial y}\Big|_{y=-0}=-f(x) & (|x|\le a); \\ \varphi_2(x,y)\big|_{y=-H}=0 & (-\infty<x<\infty). \end{cases} \qquad (4)$$

The boundary value problem (4) using conditions (1) is transformed into the following mixed boundary value problem for the conjugate function $\psi_2(x,y)$:

$$\begin{cases} \dfrac{\partial^2\psi_2}{\partial x^2}+\dfrac{\partial^2\psi_2}{\partial y^2}=0 & ((x,y)\in\omega) \\ \dfrac{\partial\psi_2}{\partial y}\Big|_{y=-0}=0 & (|x|>a); \\ \psi_2(x,y)\big|_{y=-0}=\int\limits_{-a}^{x}f(s)\,ds+c_2; & (|x|\le a); \\ \dfrac{\partial\psi_2}{\partial y}\Big|_{y=-H}=0 & (-\infty<x<\infty), \end{cases} \qquad (5)$$

where c_2 is a constant to be determined.

III. The mixed problem I is considered when the lower edge of the layer is rigidly fixed and the similarly mixed problem II is considered when the lower edge of the layer is free of external forces. These problems are formulated mathematically as completely analogous boundary value problems (2)-(4).

Let us proceed to the mixed problem theory of filtration. First, we give the basic equations of the plane theory of steady-state fluid filtration into a porous anisotropic soil mass, with referred to a rectangular coordinate system Oxy, and the vertical axis Oy is directed down. In this model, when the forces of resistance of the skeleton of the ground are accounted by

Darcy's law and taking into account anisotropy [7,8], we have

$$v_x = -k_1 \frac{\partial h}{\partial x},\ v_y = -k_2 \frac{\partial h}{\partial y};\ h = \frac{p}{\gamma} - y \quad (x, y \in S). \tag{6}$$

Here v_x, v_y are components of the velocity vector of filtration; k_1, k_2 are coefficients of fluid filtration in the ground, respectively, in the horizontal and vertical directions $(k_1 \neq k_2)$; $p = p(x,y)$ is a pressure; γ is the specific gravity of the fluid; $h = h(x,y)$ is a piezometric push; S is a filtering area. Substituting the expressions for the velocity components (6) in the continuity equation [7,8], will come to the generalized Laplace equation for the function $h(x,y)$

$$k^2 \frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} = 0 \quad \left((x,y) \in S;\quad k = \sqrt{k_1/k_2}\right). \tag{7}$$

Further, passing to the new variables $\xi = x/k;\ y = \eta$, equation (7) is transformed into the ordinary Laplace equation

$$\Delta h_0 = \frac{\partial^2 h_0}{\partial \xi^2} + \frac{\partial^2 h_0}{\partial \eta^2} = 0;\quad h = h_0(\xi,\eta) = h(k\xi,\eta). \tag{8}$$

Now consider the mixed boundary problem of the plane theory of steady-state fluid filtration in to a porous an isotropic soil mass in the form $\Omega_0 = \{-\infty < x < \infty; 0 < y < T\}$ of a power T, which associated with the calculation of hydraulic structures such as dams (Fig. 2).

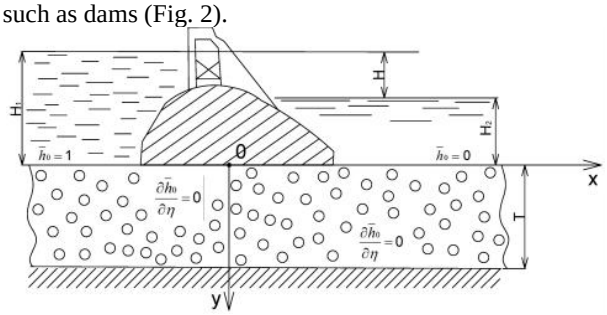

Fig.2 Fluid filtration under a hydrodynamic structure of dams type

Let the height of fluid at the upstream reach be H_1, and at the downstream reach H_2 $(H_2 < H_1)$ and the flute-underground base of the dam, together with the water proofing $y = T$ is watertight. Using (8), the described problem in the case of a flat base flute $\omega = \{y = 0; -b < x < b\}$ is mathematically formulated as the following mixed boundary value problem for a harmonic function $\bar{h}_0$ in the layer Ω:

$$\begin{cases} \dfrac{\partial^2 \bar{h}_0}{\partial \xi^2} + \dfrac{\partial^2 \bar{h}_0}{\partial \eta^2} = 0 \qquad (-\infty < \xi < \infty;\ 0 < \eta < T) \\ \bar{h}_0\big|_{\eta=0} = 1 \ (-\infty < \xi < -b_0);\quad \bar{h}_0\big|_{\eta=0} = 0 \quad (b_0 < \xi < \infty); \\ \left.\dfrac{\partial \bar{h}_0}{\partial \eta}\right|_{\eta=0} = 0 \quad (-b_0 < \xi < b_0);\quad \left.\dfrac{\partial \bar{h}_0}{\partial \eta}\right|_{\eta=T} = 0 \ (-\infty < \xi < \infty). \end{cases} \tag{9}$$

Here, as in [9], dimensionless piezometric pressures are introduced

$$\bar{h}_0 = \bar{h}_0(\xi,\eta) = \left(h_0(\xi,\eta) - H_2\right)/H,\ H = H_1 - H_2,\ b_0 = b/k.$$

It is required to determine the law of distribution of piezometric pressure h along the underground contour of the apron, the output velocity component $v_\eta = v_y$ along the line of the downstream reach lower creek, and the total filtration rate.

Further, in the context of applying the integral Fourier transform with the aim of constructing the solution of the boundary value problem (9), conveniently, as above, this problem should be reformulated for the function $\psi_0(\xi,\eta)$ conjugate to function $\bar{h}_0(\xi,\eta)$ to is analog of the current function, satisfying the Cauchy-Riemann conditions

$$\frac{\partial \bar{h}_0}{\partial \xi} = \frac{\partial \psi_0}{\partial \eta};\qquad \frac{\partial \bar{h}_0}{\partial \eta} = -\frac{\partial \psi_0}{\partial \xi}.$$

As a result, the problem (9) is transformed into the following boundary value problem for the function $\psi_0(\xi,\eta)$:

$$\begin{cases} \dfrac{\partial^2 \psi_0}{\partial \xi^2} + \dfrac{\partial^2 \psi_0}{\partial \eta^2} = 0 \quad (-\infty < \xi < \infty;\ 0 < \eta < T) \\ \left.\dfrac{\partial \psi_0}{\partial \eta}\right|_{\eta=0} = 0 \quad (b_0 < |\xi| < \infty); \\ \psi_0\big|_{\eta=0} = c \ (-b_0 < \xi < b_0); \\ \psi_0\big|_{\eta=T} = 0 \quad (-\infty < \xi < \infty). \end{cases} \tag{10}$$

Here c is till unknown constant.

3. Reduction of boundary value problems to integral equations

The solutions of problems (2)-(5) can be constructed by an integral Fourier transform. But it is convenient first to construct solutions of the boundary value problems (3) and (5) and then to find the conjugate harmonic functions $\varphi_k(x,y)$ $(k = 1,2)$ from the conditions (1).

Let us now proceed to the solution of the boundary value problem (3), reducing its solution to the solution of the integral equation. For this purpose, for function $\psi_1(x,y)$, previously, the following auxiliary boundary value problem is considered:

$$\begin{cases} \dfrac{\partial^2\psi_1}{\partial x^2}+\dfrac{\partial^2\psi_1}{\partial y^2}=0 \quad \big((x,y)\in\omega\big) \\ \left.\dfrac{\partial\psi_1}{\partial y}\right|_{y=-0}=g(x); \quad \psi_1\big|_{y=-H}=0 \quad (-\infty<x<\infty); \end{cases}$$

$$g(x)=\begin{cases} 0 & (|x|>a); \\ \chi_1(x) & (|x|<a), \end{cases}$$

(11)

where $\chi_1(x)$ is the desired calculation function. Now applying the Fourier integral transform to the boundary value problem (11), we arrive at the following one-dimensional boundary value problem:

$$\begin{cases} \dfrac{d^2\overline{\psi}_1}{dy^2}-\lambda^2\overline{\psi}_1=0 \quad (-H<y<0) \\ \left.\dfrac{d\overline{\psi}_1}{dy}\right|_{y=-0}=\overline{g}(\lambda); \quad \overline{\psi}_1\big|_{y=-H}=0; \end{cases} \tag{12}$$

$$\big\{\overline{\psi}_1(\lambda,y);\ \overline{g}(\lambda)\big\}=\int_{-\infty}^{\infty}\big\{\psi_1(x,y);\ g(x)\big\}e^{i\lambda x}dx.$$

The solution of the boundary value problem (12) has the form

$$\overline{\psi}_1(\lambda,y)=sh\big(\lambda(y+H)\big)\overline{g}(\lambda)\big/\lambda\, ch(\lambda H) \quad (-H\le y\le 0),$$

from which by the inverse Fourier transform formula we find

$$\psi_1(x,y)=\frac{1}{\pi}\int_{-1}^{1}K_1\big(|x-s|,y\big)\chi_1(s)\,ds \quad \big((x,y)\in\omega\big), \tag{13}$$

$$K_1(x,y)=\int_0^{\infty}sh\big(\lambda(y+H)\big)\cos(\lambda x)\,d\lambda\big/\lambda ch(\lambda H)=$$

$$=\frac{1}{2}\ln\left[\frac{ch(\pi x/2H)+\sin\big(\pi(y+H)/2H\big)}{ch(\pi x/2H)-\sin\big(\pi(y+H)/2H\big)}\right].$$

Hence,

$$\psi_1(x,0)=\frac{1}{\pi}\int_{-a}^{a}\ln\left(cth\left(\frac{\pi|x-s|}{4H}\right)\right)\chi_1(s)\,ds, \tag{14}$$

where the expression for the well-known Fourier cosine integral [10] (p. 530, formula 4.116.2) was used. Now using (14), realizing the third condition of the problem (3), relative to the function $\chi_1(x)$ we arrive at the next defining integral equation (IE) of Fredholm of the first kind:

$$\int_{-a}^{a}\ln\left(cth\left(\frac{\pi|x-s|}{4H}\right)\right)\chi_1(s)\,ds=h_1(x)=\pi h(x)+\pi c_1;$$

$$h(x)=\int_{-a}^{x}f(s)\,ds.$$

(15)

Now, the solution of IE (15) for the right-hand side of $h(x)$ we denote by $\omega_1(x)$, and for the right-hand side of the 1-th by $q_1(x,a)$. Then, we have

$$\chi_1(x)=\pi\omega_1(x)+\pi c_1\,q_1(x,a) \quad (-a<x<a)$$

and consequently, (13) can be written in the form

$$\psi_1(x,y)=\int_{-a}^{a}\big[\omega_1(s)+c_1\,q_1(s,a)\big]ds\times$$

$$\times\int_0^{\infty}\frac{sh\big(\lambda(y+H)\big)}{\lambda ch(\lambda H)}\cos\big(\lambda(x-s)\big)\,d\lambda \quad \big((x,y)\in\omega\big). \tag{16}$$

Now, by the first formula of the Cauchy-Riemann conditions (1), we obtain

$$\frac{\partial\varphi_1}{\partial x}=\frac{\partial\psi_1}{\partial y}=\int_{-a}^{a}\big[\omega_1(s)+c_1\,q_1(s,a)\big]ds\times$$

$$\times\int_0^{\infty}\frac{ch\big(\lambda(y+H)\big)}{ch(\lambda H)}\cos\big(\lambda(x-s)\big)\,d\lambda \quad \big((x,y)\in\omega\big).$$

After integrating this equality with respect to x, we have

$$\varphi_1(x,y)=\int_{-a}^{a}\big[\omega_1(s)+c_1\,q_1(s,a)\big]ds\times$$

$$\times\int_0^{\infty}\frac{ch\big(\lambda(y+H)\big)}{\lambda ch(\lambda H)}\sin\big(\lambda(x-s)\big)\,d\lambda+p_1(y)\ \big((x,y)\in\omega\big), \tag{17}$$

where $p_1(y)$ as yet is while an arbitrary function. From here

$$\varphi_1(x,0)=\int_{-a}^{a}\big[\omega_1(s)+c_1\,q_1(s,a)\big]ds\times$$

$$\times\int_0^{\infty}\sin\big(\lambda(x-s)\big)\frac{d\lambda}{\lambda}+p_1(0) \quad (-\infty<x<\infty). \tag{18}$$

Taking into account the value of the second integral (the discontinuous Dirichle integral), we discuss the following cases: 1) $-\infty<x<-a$. In this case $x-s<0$ and the basis of the first condition of the boundary value problem (2) from (18), we get

$$p_1(0)-(\pi Q_1/2)c_1=Gw_0+(\pi/2)\Omega_1;$$

$$Q_1=\int_{-a}^{a}q_1(s,a)\,ds, \ \Omega_1=\int_{-a}^{a}\omega_1(s)\,ds. \tag{19}$$

2) $a<x<\infty$. Then $x-s>0$ and by the second condition of the boundary value problem (2) from (18) we obtain

$$p_1(0)+(\pi Q_1/2)c_1=-(\pi/2)\Omega_1. \tag{20}$$

From (19) and (20) we find

$$p_1(0)=Gw_0/2, \ c_1=-(\pi\Omega_1+w_0G)/\pi Q_1. \tag{21}$$

It should be noted, that if the second boundary condition of the problem (2) is replaced by condition $\varphi_1(x,-0)=-w_0G\ (a<x<\infty)$, then instead of (21) we have $p_1(0)=0, \ c_1=-(\pi\Omega_1+2w_0G)/\pi Q_1$.

Continuing the problem of defining the function $\varphi_1(x,y)$, we turn to the second Cauchy-Riemann condition (1). Then by (16) and (17) we can write

$$\frac{\partial\varphi_1}{\partial y}=\int_{-a}^{a}\left[\omega_1(s)+c_1 q_1(s,a)\right]ds\times$$
$$\times\int_0^{\infty}\frac{sh\left(\lambda(y+H)\right)}{ch(\lambda H)}\sin\left(\lambda(x-s)\right)d\lambda+p_1'(y)=-\frac{\partial\psi_1}{\partial x}=$$
$$=\int_{-a}^{a}\left[\omega_1(s)+c_1 q_1(s,a)\right]ds\times$$
$$\times\int_0^{\infty}\frac{sh\left(\lambda(y+H)\right)}{ch(\lambda H)}\sin\left(\lambda(x-s)\right)d\lambda\quad\left((x,y)\in\omega\right).$$

It follows that

$$p_1'(y)=0\Rightarrow p_1(y)\equiv const\Rightarrow p_1(y)\equiv p_1(0)=Gw_0/2.$$

Consequently, we finally have

$$\varphi_1(x,y)=\int_{-a}^{a}\left[\omega_1(s)+c_1 q_1(s,a)\right]ds\times$$
$$\times\int_0^{\infty}\frac{ch\left(\lambda(y+H)\right)}{\lambda ch(\lambda H)}\sin\left(\lambda(x-s)\right)d\lambda+\frac{Gw_0}{2}\quad\left((x,y)\in\omega\right).$$

Turning to the boundary value problems (4) and (5), we arrive at the following defining IE in the same analogously way:

$$\int_{-a}^{a}\ln\left|1\Big/2sh\left(\frac{\pi|x-s|}{2H}\right)\right|\chi_2(s)ds=$$
$$=h_2(x)=\pi h(x)+\pi c_2 q_2(x,a),\quad h(x)=\int_{-a}^{x}f(s)ds.\tag{22}$$

We denote as above the solution of IE (22) for the right-hand side $h(x)$ by $\omega_2(x)$, and for the right-hand side of 1 through $q_2(x,a)$. Then we have

$$\chi_2(x)=\pi\omega_2(x)+\pi c_2 q_2(x,a)\quad(-a<x<a)$$
$$\psi_2(x,y)=\int_{-a}^{a}\left[\omega_2(s)+c_2 q_2(s,a)\right]ds\times$$
$$\times\int_0^{\infty}\frac{ch\left(\lambda(y+H)\right)}{\lambda sh(\lambda H)}\cos\left(\lambda(x-s)\right)d\lambda;\quad\left((x,y)\in\omega\right);$$
$$\varphi_2(x,y)=\int_{-a}^{a}\left[\omega_2(s)+c_2 q_2(s,a)\right]ds\times$$
$$\times\int_0^{\infty}\frac{sh\left(\lambda(y+H)\right)}{\lambda sh(\lambda H)}\sin\left(\lambda(x-s)\right)d\lambda=$$
$$=\int_{-a}^{a}arctg\left(tg\left(\pi(y+H)/2H\right)/th(\pi x/2H)\right)\times$$
$$\times\left[\omega_2(s)+c_2 q_2(s,a)\right]ds;$$
$$p_2(0)=0,\quad c_2=-\left(\pi\Omega_2+2Gw_0\right)/\pi Q_2;$$
$$\Omega_2=\int_{-a}^{a}\omega_2(s)ds;\quad Q_2=\int_{-a}^{a}q_2(s,a)ds,$$

where notation, corresponding to the first boundary-value problem is taken.

We note that the solution of the boundary value problem (10) also reduces to solution of the IE (15).

4. Solution of determining IE

Starting the solution of the IE (15) and (22), first we put in them

$$\xi=\pi x/H,\ \eta=\pi s/H;\ \alpha=\pi a/H;$$
$$\chi_k^0(\xi)=(H/\pi)\chi_k(H\xi/\pi);\ h_k^0(\xi)=h_k(H\xi/\pi)\ (k=1,2).$$

As a result, they will take, respectively, the form

$$\int_{-\alpha}^{\alpha}\ln cth\left(\frac{|\xi-\eta|}{4}\right)\chi_1^0(\eta)d\eta=h_1^0(\xi);\tag{23}$$
$$\int_{-\alpha}^{\alpha}\ln\left|1\Big/2sh\left(\frac{|\xi-\eta|}{2}\right)\right|\chi_2^0(\eta)d\eta=h_2^0(\xi);\ (-\alpha<\xi<\alpha);$$
$$h_k^0(\xi)=\pi h(H\xi/\pi)+\pi c_k;\ \chi_k^0(\xi)=\pi\omega_k^0(\xi)+\pi c_k q_k^0(\xi,\alpha);\tag{24}$$
$$\omega_k^0(\xi)=\omega_k(H\xi/\pi)\quad(k=1,2).$$

Here $q_1^0(\xi,\alpha)$ and $q_2^0(\xi,\alpha)$ are solutions of the IE

$$\int_{-\alpha}^{\alpha}\ln cth\left(\frac{|\xi-\eta|}{4}\right)q_1^0(\eta,\alpha)d\eta=1;$$
$$\int_{-\alpha}^{\alpha}\ln\left|1\Big/2sh\left(\frac{|\xi-\eta|}{2}\right)\right|q_2^0(\eta,\alpha)d\eta=1.$$

These functions are given by the formulas [11] (p.245)

$$q_1^0(\xi,\alpha)=\left[\pi Q_{-1/2}(ch\alpha)\sqrt{2(ch\alpha-ch\xi)}\right]^{-1};$$
$$q_2^0(\xi,\alpha)=\pi^{-1}M_0(\alpha)ch(\xi/2)\left[2(ch\alpha-ch\xi)\right]^{-1/2};\tag{25}$$
$$M_0(\alpha)=\left[\ln\left(1/sh(\alpha/2)\right)\right]^{-1},$$

where $Q_{-1/2}(x)$ is the function of the Legendre of the second kind. In the previous variables we have

$$q_1(x,\alpha)=\left[HQ_{-1/2}\left(ch(\pi a/H)\right)\sqrt{2\left(ch(\pi a/H)-ch(\pi x/H)\right)}\right]^{-1};$$
$$q_2(x,\alpha)=\left(M(a)/H\right)ch(\pi x/2H)\times$$
$$\times\left\{2\left[ch(\pi a/H)-ch(\pi x/H)\right]\right\}^{-1/2};$$
$$M(a)=-\left(\ln\left(sh(\pi a/2H)\right)\right)^{-1}.\tag{26}$$

Now, using (26), we can calculate the quantities Q_1 and Q_2

$$Q_1=P_{-1/2}\left(ch(\pi a/H)\right)\Big/Q_{-1/2}\left(ch(\pi a/H)\right);$$
$$Q_2=M(a)=\left[\ln\left(1/sh(\pi a/2H)\right)\right]^{-1},$$

where $P_{-1/2}(x)$ is the function of the Legendre of the first kind.

We can construct the solution of IE (23) and (24) by applying various methods. Based on (25), their solutions can be obtained using formulas of M.G. Kerin [11] (Ch. IV, §8). Further, by differentiating both sides of (23) and (24) with respect to ξ, we come to the SIE:

$$\int_{-\alpha}^{\alpha}\chi_1^0(\eta)d\eta\Big/sh\left((\eta-\xi)/2\right)=2\left[h_1^0(\xi)\right]';$$
$$\int_{-\alpha}^{\alpha}cth\left((\eta-\xi)/2\right)\chi_2^0(\eta)d\eta=2\left[h_2^0(\xi)\right]'.$$

The solutions of these equations are given in [12].

Solutions of these IE can also be found using the method of orthogonal polynomials. Specifically, the solution of the IE (23) by this method can be obtained using the spectral relationships established in [13].In order to solve the IE (24) we apply the same method using known spectral relationships established in [14].

$$\frac{1}{\pi}\int_{-1}^{1}\ln\frac{1}{|x-s|}\frac{T_n(s)\,ds}{\sqrt{1-s^2}}=\begin{cases}\dfrac{1}{n}T_n(x)\ (n=1,2,...);\\ \ln 2\quad (n=0),\end{cases}\quad(-1<x<1) \tag{27}$$

where $T_n(x)$ are Chebyshev polynomials of the first kind. These polynomials are transformed into the corresponding relationships for the kernel of the IE (24). For this purpose in (27) with $n=2m\ (m=0,1,2,...)$ we put

$$x=sh(\xi/2)/sh(\alpha/2),\ s=sh(\eta/2)/sh(\alpha/2)\ (-\alpha<\xi,\eta<\alpha),$$

and when $n=2m+1\ (m=0,1,2,...)$ -

$$x=th(\xi/2)/th(\alpha/2),\ s=th(\eta/2)/th(\alpha/2)\ (-\alpha<\xi,\eta<\alpha).$$

After simple transformations, the following spectral relationships are obtained:

$$\frac{1}{\pi}\int_{-\alpha}^{\alpha}\ln\left[1\Big/2sh\left(\frac{|\xi-\eta|}{2}\right)\right]\frac{T_{2m}\left(\dfrac{sh(\eta/2)}{sh(\alpha/2)}\right)ch(\eta/2)\,d\eta}{\sqrt{2(ch\alpha-ch\eta)}}=$$

$$=\begin{cases}\dfrac{1}{2m}T_{2m}\left(\dfrac{sh(\xi/2)}{sh(\alpha/2)}\right)\quad(m=1,2,...);\\ \ln\dfrac{1}{sh(\alpha/2)}\qquad(m=0);\end{cases} \tag{28}$$

$$\frac{1}{\pi}\int_{-\alpha}^{\alpha}\ln\left[1\Big/2sh\left(\frac{|\xi-\eta|}{2}\right)\right]\frac{T_{2m+1}\left(\dfrac{th(\eta/2)}{th(\alpha/2)}\right)d\eta}{ch(\eta/2)\sqrt{2(ch\alpha-ch\eta)}}=$$

$$=\frac{T_{2m+1}\left(\dfrac{th(\xi/2)}{th(\alpha/2)}\right)}{(2m+1)ch(\alpha/2)}\quad(m=0,1,2,...). \tag{29}$$

Under these conditions, the orthogonal polynomials Chebyshev of the first kind with the indicated modified arguments have the form

$$\int_{-\alpha}^{\alpha}T_{2m}\left(\frac{sh(\eta/2)}{sh(\alpha/2)}\right)T_{2n}\left(\frac{sh(\eta/2)}{sh(\alpha/2)}\right)\frac{ch(\eta/2)\,d\eta}{\sqrt{2(ch\alpha-ch\eta)}}=$$

$$=\begin{cases}\pi & (m=n=0);\\ \dfrac{\pi}{2} & (m=n\neq 0);\\ 0 & (m\neq n);\end{cases} \tag{30}$$

$$\int_{-\alpha}^{\alpha}T_{2m+1}\left(\frac{th(\eta/2)}{th(\alpha/2)}\right)T_{2n+1}\left(\frac{th(\eta/2)}{th(\alpha/2)}\right)\frac{d\eta}{ch(\eta/2)\sqrt{2(ch\alpha-ch\eta)}}=$$

$$=\begin{cases}\dfrac{\pi}{2}ch(\alpha/2) & (m=n);\\ 0 & (m\neq n).\end{cases} \tag{31}$$

Then, we expand the solution of the IE (24) into symmetric and skew-symmetric parts:

$$\chi_2^0(\xi)=\chi_+(\xi)+\chi_-(\xi),\ h_2^0(\xi)=h_+(\xi)+h_-(\xi);$$
$$\chi_\pm(-\xi)=\pm\chi_\pm(\xi);\quad h_\pm(-\xi)=\pm h_\pm(\xi).$$

As a result, we come to the following IE:

$$\int_{-\alpha}^{\alpha}\ln\left[1\Big/2sh\left(\frac{|\xi-\eta|}{2}\right)\right]\chi_\pm(\eta)\,d\eta=h_\pm(\xi)\ (-\alpha<\xi<\alpha). \tag{32}$$

Now, the solutions of IE (32) can be represented in the form of infinite series $(-\alpha<\xi<\alpha)$

$$\chi_+(\xi)=\frac{ch(\xi/2)}{\sqrt{2(ch\alpha-ch\xi)}}\sum_{m=0}^{\infty}X_m^+T_{2m}\left(\frac{sh(\xi/2)}{sh(\alpha/2)}\right); \tag{33}$$

$$\chi_-(\xi)=\frac{1}{ch(\xi/2)\sqrt{2(ch\alpha-ch\xi)}}\sum_{m=0}^{\infty}X_m^-T_{2m+1}\left(\frac{th(\xi/2)}{th(\alpha/2)}\right) \tag{34}$$

with unknown coefficients $X_m^\pm\ (m=0,1,2,...)$. Substituting (33) into the IE (32) with the sign "+", and (34)— into the IE (32) with the sign "-" and using the spectral relationships (28) and (29), as well as the orthogonality conditions (30) and (31), after simple transformations we have

$$X_0^+=-h_0^+/\pi^2\ln sh(\alpha/2);\ X_m^+=4mh_m^+/\pi^2\ (m=1,2,...);$$
$$X_m^-=2\pi^{-2}(2m+1)ch(\alpha/2)\,h_m^-\quad(m=0,1,2,...); \tag{35}$$

$$h_m^+=\int_{-\alpha}^{\alpha}h_+(\xi)\frac{T_{2m}\left(sh(\xi/2)/sh(\alpha/2)\right)ch(\xi/2)\,d\xi}{\sqrt{2(ch\alpha-ch\xi)}}$$

$$h_m^-=\int_{-\alpha}^{\alpha}h_-(\xi)\frac{T_{2m+1}\left(th(\xi/2)/th(\alpha/2)\right)d\xi}{ch(\xi/2)\sqrt{2(ch\alpha-ch\xi)}}\ (m=0,1,2,...).$$

Accordingly, the exact solution of IE (24) has been obtained in the form of infinite series (33) and (34) with coefficients (35).The coefficients (35) can easily be computed by Gaussian quadrature formulae.

5. Results

The following primary results have been obtained in the paper:

a) exact solutions of a class of mixed boundary-value problems of the theory of elasticity for a layer and the theory of fluid filtration into a porous soil massif in the form of a layer are obtained by a general method of IE solving;

b) the solutions of IE are represented by explicit analytical formulae, convenient for numerical analysis.

6. Conclusion

Used in the present paper method of solving IE for characteristic problems enables to get explicit analytical expressions that have theoretical and practical interest. This method can be successfully used for effective solution of other boundary value problems of continuum mechanics.

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STUDY OF THE STABILITY OF ANNULAR ROUND PLATES BY THE USAGE OF HERMITE'S FUNCTION

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ABSTRACT

The paper presents an approximate method designed to study stability of thin-wall annular slab of constant thickness when it's under arbitrary axial symmetry external influences. Unlike our proposed for another approximate method based on the hypothesis of rigid section, in this case the restrictions are eliminated plowed kinematic conditions. As a result, the basic equation of stability has been derived, at the same time using was applied Bubnov-Galerkin method,the relations of critical values determination of external influences.

1. Introduction

The design process of building stractures, mechanical systems and machines is carried out according to calculations of the strength and stability of rods, plates and membranes. Moreover, for the production of semiconductor devices and integrated circuits, the initial material, semiconductor platforms in the technological process are subjected to mechanical, chemical and thermal processing. As a result, there occur residual tenions which cause a loss of stability of the form of flat platforms.

In the work [1] observed by us, being based on the hypothesis of rigid cross sections [2], we got the intensive critical values of the equally distributed volumetric forces for marginal limited kinematic conditions. The obtained results, in terms of assessment, can be applied in the process of installation of semiconductor equipment and creation of concentration of optimal technological methods and devices.

2. Method

The approximate method proposed by our recently published work [3] on the stability of large round plates needs to be generalized for circular plates, by excluding the restrictions that still exist in the research done [1,2]. The Fig.1 shows the cranked shape of the circular plate of average plain in the hour-circle plain. Let us present the function describing it by the help of normalized coordinate system

of Hermite functions (Fig. 2) as it was done in [3] work,

$$w(\eta) = \psi_c w_c + \sum_{k=i,j} [Э_к] \cdot \{\delta_k\} \qquad (1)$$

where $\{\delta_k\} = [w_k, \theta_k]^T$ – are generalized displacements of base points

$[Э_к] = [\psi_k(\eta), \Phi_k(\eta)]$- are Hermite functions corresponding to base points.

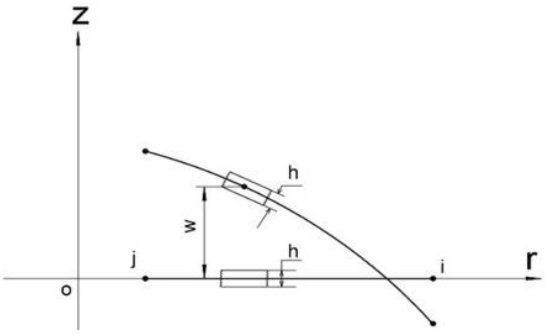

Fig.1 Bent plate

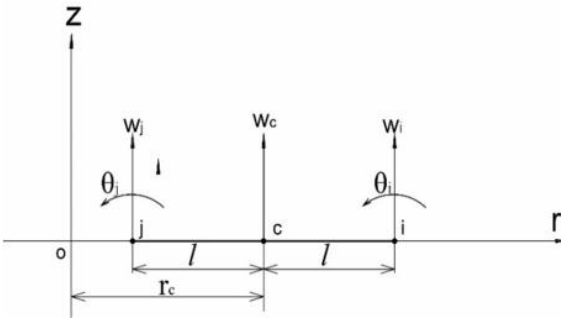

Fig.2 Base points generalized displacements

Note that the choice of the base points is optional. To simplify the calculations, it is convenient that they include not only the points corresponding to the plate outline, but also the points where kinematic conditions are given. On the other hand you have to take into consideration the consequence that the construction of Hermite relevant functions depends on the suitable choice of the base points.

In the normalized coordinates, the functions describing the shape of the circular cranked plate illustrated in Fig. 2 ($-1\le\eta\le1$) will be the following [3]

$$\psi_k=\frac{1}{4}\eta_0(\eta_0+1)(3+\eta_0-2\eta^2) \quad (2)$$

$$\Phi_k=\frac{1}{4}l\eta(\eta_0-1)(\eta_0+1)^2\ ,\ (k=i,j) \quad (3)$$

$$\psi_c=(1-\eta^2)^2 \quad (4)$$

where such assignments are made

$$\eta_0=\eta_k\cdot\eta\ ,\ \eta=\frac{r-r_c}{l}\ ,\ (k=i,j) \quad (5)$$

In addition, it should be noted that if c point is not included in the list of base points illustrated in Fig. 2, then according to Hermite the functions will be

$$\psi_k=\frac{1}{4}(\eta_0+1)(2+\eta_0-\eta^2), \quad (6)$$

$$\Phi_k=\frac{1}{4}l\eta(\eta_0-1)(\eta_0+1)^2\ ,\ (k=i,j) \quad (7)$$

$$\psi_c=0 \quad (8)$$

The following relations of determination of internal force factors are known from the theory of bending thin-walled solid plates [4]

$$M_r=-D\left(\frac{d^2w}{dr^2}+\frac{\mu}{r}\frac{dw}{dr}\right), \quad (9)$$

$$M_\varphi=-D\left(\frac{1}{r}\frac{dw}{dr}+\mu\frac{d^2w}{dr^2}\right), \quad (10)$$

$$Q_r=D\frac{d}{dr}\left[\frac{1}{r}\frac{d}{dr}\left(r\frac{dw}{dr}\right)\right], \quad (11)$$

where D-is the rigidness of the plate, μ-is Puasson coefficient.

Considering (1) – (8) relations, (9)-(11) internal force factors η immeasurable parameters depending will be presented in the following form

$$M_r=-\frac{D}{\ell^2}\{(\psi_c''+\mu\beta\psi_c')\cdot w_c+\sum_{k=i,j}[(\psi_k''+\mu\beta\psi_k'),(\Phi_k''+\mu\beta\Phi_k')]\{\delta_k\} \quad (12)$$

$$M_\varphi=-\frac{D}{\ell^2}\{(\beta\psi_c'+\mu\psi_c'')\cdot w_c+\sum_{k=i,j}[(\beta\psi_k'+\mu\psi_k''),(\beta\Phi_k'+\mu\Phi_k'')]\{\delta_k\} \quad (13)$$

$$Q_r=-\frac{D}{\ell^3}\{(-\beta^2\psi_c'+\beta\psi_c''+\psi_c''')\cdot w_c+\sum_{k=i,j}[(-\beta^2\psi_k'+\beta\psi_k''+\psi_k'''),(-\beta^2\Phi_k'+\beta\Phi_k''+\Phi_k''')]\{\delta_k\} \quad (14)$$

where it is assigned $\beta=\frac{1}{\eta+m}$, $m=\frac{r_c}{l}$ (Fig. 1) derivatives are marked with strokes.

Suppose a annular round plate in an average plane is loaded with optional redial q(η) distributed load, and the contours – with equally distributed radial forces P_j and P_i. by observing (Fig. 3)

the balance of the half plate, we will get the basic equations describing stability.

$$\int_F\sigma_\varphi^0 dF-R_y=0 \quad (15)$$

$$\int_S q_z(r)\,ds+\int_{L_k}P_{kz}\,dL+Q_z-R_z=0 \quad (16)$$

$$2\int_{r_j}^{r_i}M_\varphi dr+\int_F\sigma_\varphi^0 w dF+\bar{M}=0 \quad (17)$$

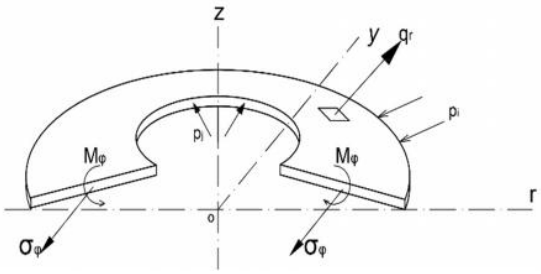

Fig.3 Half of the plate in stability loss state

where F is the surface of the plate meridian section, $F=4lh$, $S=2\pi r_c l$ - is the average plane surface of the plate, $L_k=\pi r_k$ $(k=i,j)$ –is the length of the contour of the half plate.

Static equivalents of external forces and communication counteractions $\bar{R}(R_y,R_z)$ and $\bar{M}$, Q_z- communication counteraction, $q_z(r)$ and P_z- are the components of the relevant external force after the loss of stability (when the forces are tracking). In case of dead forces (16) will satisfy similarly, (15) also satisfies similarly, which is immediately derived from a smooth resolution of the problem. Consequently, in case of dead force impact, the primary equation of the stability will

be presented in (17) finite form. Inserting (1) and (13) relations in (17) and doing corresponding mathematical calculations, as a result we will have the system of linear homogeneous equations towards generalized transfers of base points.

where $x = \sum_{k=c,i,j} \{(\frac{h\cdot\sigma_r^0}{D}\cdot\frac{l^2}{\beta^2} - 1)\cdot[\vartheta_K']\cdot\{\delta_k\} + \frac{1}{\beta}\cdot[\vartheta_K'']\cdot\{\delta_k\} + \frac{1}{\beta^2}\cdot[\vartheta_K''']\cdot\{\delta_k\}\}$

σ_r^0 – is the radial tension, which is determined by a smooth solution of the problem. The critical value of external influences is determined (18) by the condition of the main determinate of the equation system being zero. Finally, note that each problem can be solved with the proposed two methods to verify the accuracy of the solution.

3. Conclusion

By introducing base points and Hermite functions the calculations procedure becomes essentialy simple.

By adding the number of base points the accuracy of solution considerably increases.

In the formula of critical pressure of a plate fixed by its edges in calculation of m coefficient.

Poisson ratio does not take part.

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ON DETERMINATION OF VALUES OF THE VELOCITIES OF TRANSVERSE SEISMIC VIBRATIONS OF SOIL PARTICLES IN STRONG EARTHQUAKES

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Keywords: rupture, soil speed, earthquake magnitude

Abstract

Considering a strong earthquake as an instantaneous rupture of the earth's crust (ripping), the task is to determine the maximum value of the initial velocity of transverse seismic vibrations of soil particles on the earth surface near the rupture and at a certain distance from it, depending on the magnitude of the earthquake. In the case of short-term mechanical discontinuities of the medium (earthquakes), the main parameter of the effect on the earth's surface is the values of the initial velocities that the ruptured parts of the medium acquire after the rupture. The paper considers the problem of determining the values of these velocities in strong earthquakes, depending on the magnitude of the earthquake. Using theoretical research of J. N. Brune set the maximum values of accelerations and ground velocities in the vicinity of a point source (hearth), and based on the works of G. Reed C. Lomnitz, S. Singh, K. Kasahara, T. Rikitake, D. Wells, K. Copperschmidt, E. Khachiyan, an empirical relationship was obtained between the maximum velocity of soil particles near the rupture and at a certain distance from it and the magnitude of the predictable earthquake. The effect of the duration of the rupture, in comparison with its instantaneous case, on the values of the initial rates of soil vibration is estimated. The limiting values of displacements, velocities and accelerations are calculated for elastic vibrations of rocky soils. The values of rates and intensities of earthquakes, established by the developed method, are compared with the intensities established on the basis of the MSK-64 seismic scale, showing a sufficient correlation for earthquakes with intensity V-X intensity and magnitudes $6.0 \leq M \leq 8.0$. Based on the results obtained, a method for determining synthetic accelerograms of soils of predicted strong earthquakes with a magnitude $M \geq 6.0$ for different seismic building sites was developed. The method was applied for solid and loose basements.

1. Introduction

One of the main tasks of seismology and the theory of seismic stability of structures is the identification of the characteristic regularities of soil variations (building sites), their amplitude-frequency characteristics, and the establishment of standard values for the movement of velocities and the acceleration of various soils in strong earthquakes.

Given the lack of a sufficient number of real strong earthquake records for most seismically active territories of the earth, allowing them to conduct a full statistical analysis of them for specific construction sites, the task of finding ways to artificially construct seismograms and accelerograms of strong ground motions that adequately reflect the properties of events actually recorded provides a certain theoretical and practical interest.

According to modern concepts, the earthquake is a consequence of a mechanical rupture - the ripping of the environment due to the contact of two geological blocks (plates) with rough edges with their long slow movement in opposite directions. Before the earthquake in some area of the earth's crust, there are continuously increasing mechanical shearing stresses and deformations with the accumulation of a huge amount of potential deformation energy. When the strain reaches its limit values, a break occurs (earthquake), the accumulated potential energy quickly turns into the kinetic energy of seismic waves propagating throughout the earth. On the surface of read mechanical oscillations of soil particles whose recording in the form of seismograms and accelerograms allows you to estimate the magnitude and intensity of the earthquake.

2. Initial Parameters

According to studies of J.N. Brune [1] in strong rocks near the zone at instantaneous break (earthquake) can accelerate soil exceeding 1g and velocities exceeding 100 cm/s. The upper limit of the initial velocity of soil particles, according to him, for most of strong earthquakes reach 150cm/s. Proceeding from these conclusions, from the point of view of practical application, the problem arises: how to determine the values of the initial velocity of oscillation of the fracture soil particles and at a certain distance Δ for earthquakes with a given magnitude M. If it is assumed that the initial velocity of the soil particles is equal to $v_{max} = v_{rup}$

= 100 cm/s, as indicated by J.N. Brune, corresponds to the

Table 2. The values of the velocities (cm/s) of soil particles' oscillation depending on the earthquake intensity on the scale MSK-64

Intensity	V	VI	VII	VIII	IX	X
Velocity	1÷2	2.1÷4	4.1÷8	8.1÷16	16.1÷32	32.1÷64

Figure 1 shows the v(Δ) dependences for some magnitudes M and earthquake intensity on the MSK-64 scale.

3. Influence of non-instantaneous rupture

In case of non-instantaneous destruction fracture, the initial value of soil particles' velocity, according to J. Brune [1] can be represented as:

$$v = v_{inst} e^{-t/\Delta t}, \quad (6)$$

where: v_{inst} is the above obtained value of the initial velocity at instantaneous destruction of the medium, Δt is the time of complete break, including the time of rupture, it can be written as:

$$\Delta t = \frac{L/2}{v_s}, \quad (7)$$

where: L is the length of the future rupture, vs is the speed of the transverse wave. The duration of destruction can last up to tens seconds. The ripping begins in the last stage of complete destruction, when a small part (within tens to hundreds meters) of the rupture line remains unbroken. Until this point, the dynamic effect of the earthquake - the generation of inertial forces in the ground has not yet manifested itself. The last stage of destruction (the beginning of the earthquake) -wrecking, according to experts [1,2], occurs with supersonic speed and lasts a fraction of a second, within the range t = 0.1 to 0.3 seconds (depending on the strength of rocks). Therefore, by formula (6), for example, we have:

at M = 7.0, L = 30 km, v_s = 4000 m/s,

$$\Delta t = \frac{15 \cdot 1000}{4000} = 3.75 \text{ s},$$

at M = 8.0, L = 80 km, v_s = 4000 m/s, $\Delta t = \frac{40 \cdot 1000}{4000} = 10$ s.

Assuming the ripping of the rupture, respectively, t = 0.1 sec and t = 0.3 s, according to the formula (6), we obtain:

$$v = e^{-\frac{0.1}{3.75}} v_{inst} = 0.975\, v_{inst} \quad \text{in case M=7.0,}$$

$$v = e^{-\frac{0.3}{10}} v_{inst} = 0.970\, v_{inst} \quad \text{in case M=8.0.}$$

As we see, it is not the instantaneous discontinuity that leads to an insignificant decrease in the initial velocity of oscillation of soil particles and, consequently, to the same mitigation of the earthquake effect on the earth's surface.

4. Synthetic Seismograms and Accelerograms of the Soil

case of a strong earthquake with a magnitude of M = 8.5, to take into account that according to A. Esteva [2] the recorded values of the maximum velocity of soil particles are proportional to e^M, where M is an earthquake magnitude, the value $v_{max} = v_{rup}$ in other magnitudes can be represented as follows: $v_{max} = 100\, e^{M-8.5}$ cm /s, (1)

The earth, seismic waves cause widesp according (1) to which at M = 8.5 - v_{max} = 100 cm/sec. Since, according to "The static displacement field, which determines the double dipole, without moment decreases with a distance proportionally to Δ^{-2} " [2], the dependence of the maximum value of v_{max} on the distance to the discontinuity Δ, by analogy with the studies of [4], can be represented as:

$$v(\Delta) = v_{max}\left(1 - \frac{\Delta^2}{R^2}\right). \quad (2)$$

In formula (2), R is the extent of the region in the direction of the earthquake deformed during the preparation process, perpendicular to the rupture. The deformation of the medium after Δ>R is assumed to be neglected in comparison with the deformation at the rupture. The values of R (in meters) are taken from the formula [4]:

$$R = 5(\bar{u} + 3)\cdot 10^3 \quad (3)$$

where $\bar{u}$ (in meters) is the value of the average (mean) slip [5] on the surface of the earth with an earthquake with a magnitude M≥6.0. The value $\bar{u}$ is determined by the formula [6]:

$$\lg\bar{u} = 0.55M - 3.71 \quad (4)$$

Thus, for the initial velocity of the soil particles vibration near the rupture at a distance Δ from the rupture, for quakes of M≥6.0, we get (sm/s):

$$v(\Delta) = 100\, e^{M-8.5}\left(1 - \frac{\Delta^2}{R^2}\right). \quad (5)$$

In Table 1 the values of $\bar{u}$ (m), R (km), v_{max} and v(Δ) (cm/s) calculated by formulas (1) to (5) for different Δ<R epicentral distances, depending on the magnitude of earthquakes.

In Table 2 are listed the values of the ground vibration velocities (cm/s), depending on the earthquake intensity on the MSK-64 scale. Comparison of Table 1 and Table 2 shows that the values of the ground particle velocities calculated by the formula (5) are comparable with their MSK-64 values for VII-X-point intensities in the epicentral zone, and in areas outside the zone with V-X-point intensity and6.0 ≤ M ≤ 8.0 magnitudes earthquakes.

Table 1. The values of the velocities of soil particles vibration in cm/sec, depending on the magnitude M and the distance from the rupture Δ in km.

Earthquake magnitude M	Mean slip $\bar{u}$, m	Value of R from (3), km	Value velocities for rupture v_{max}, cm/s	Value velocities v(Δ) in cm/s, depending on the magnitude M and distance from the rupture Δ in km																				
				5	10	15	20	25	30	35	40	45	50	55	60	65	70	75	80	85	90	95	100
6.0	**0.39**	**16.90**	**8.2**	**7.5**	**5.3**	**1.7**																	
6.25	**0.54**	**17.70**	**10.5**	**9.7**	**7.1**	**3.0**																	
6.50	**0.73**	**18.60**	**13.5**	**12.5**	**9.6**	**4.7**																	
6.75	**1.00**	**20.00**	**17.4**	**16.3**	**13.1**	**7.6**																	
7.0	**1.38**	**21.90**	**22.3**	**21.1**	**17.7**	**11.8**	**3.7**																
7.25	**1.90**	**24.50**	**28.6**	**27.4**	**23.8**	**17.9**	**9.5**																
7.50	**2.60**	**28.00**	**36.8**	**35.6**	**32.1**	**26.2**	**18.0**	**7.5**															
7.75	**3.57**	**32.80**	**47.2**	**46.1**	**42.8**	**37.3**	**29.7**	**19.8**	**7.7**														
8.0	**4.90**	**39.50**	**60.6**	**59.6**	**56.7**	**51.9**	**45.1**	**36.3**	**25.6**	**13.0**													
8.25	**6.73**	**48.60**	**77.9**	**77.1**	**74.6**	**70.5**	**64.7**	**57.3**	**48.2**	**37.5**	**25.1**	**11.1**											
8.50	**9.23**	**61.10**	**100**	**99.3**	**97.3**	**94.0**	**89.3**	**83.3**	**75.9**	**67.2**	**57.1**	**45.8**	**33.0**	**19.0**	**3.6**								
8.75	**12.68**	**78.40**	**128.4**	**127.9**	**126.3**	**123.7**	**120.0**	**115.3**	**109.6**	**102.8**	**95.0**	**86.1**	**76.2**	**65.2**	**53.2**	**40.1**	**26.0**	**10.9**					
9.0	**17.38**	**102.00**	**164.9**	**164.5**	**163.3**	**161.3**	**158.6**	**155.0**	**150.6**	**145.5**	**139.5**	**132.8**	**125.3**	**117.0**	**107.8**	**97.9**	**87.2**	**75.7**	**63.5**	**50.4**	**36.5**	**21.9**	**6.4**

On the basis of the above results, for the arbitrary vertical section (lithological column) in Fig. 2a [8] the displacement function of the layers U_k (z, t), (k = 1 ... n), satisfying the wave equations are plotted [7]:

$$G_k \frac{\partial^2 U_k}{\partial z^2} - \rho_k \frac{\partial^2 U_k}{\partial t^2} + \eta_k \frac{\partial^3 U_k}{\partial z^2 \partial t} = 0 \quad . \qquad (8)$$
$$k = 1,2...n$$

Initial conditions of the problem:

$$\text{at } t = 0 \qquad U_k(z,0) = 0$$
$$\text{at } t = 0 \qquad \frac{\partial U_k(z,0)}{\partial t} = v(\Delta). \qquad (9)$$
$$k = 1,2...n$$

Boundary and 2n-2 conditions for the continuity of displacements and shear stresses at the levels of separation of layers:

$$\text{at } z = 0 \qquad \frac{\partial U_1(0,t)}{\partial z} = 0, \qquad (10)$$
$$\text{at } z = H \qquad U_n(H,t) = 0,$$

$$U_k(h_k,t) = U_{k+1}(h_k,t)$$
$$G_k \frac{\partial U_k(h_k,t)}{\partial z} = G_{k+1} \frac{\partial U_{k+1}(h_k,t)}{\partial z}. \qquad (11)$$
$$k = 1,2,3...n-1$$

In the work [7], the following expressions for the displacements (seismograms) and accelerations (accelerograms) of oscillations of soil particles on the earth surface have been obtained by solving the system of equations (8), taking into account conditions (9) to $[U_1(0,t) = U(0,t)]$ (11):

$$U(0,t) = 100e^{M-8.5}\left(1-\frac{\Delta^2}{R^2}\right)\sum_{i=1}^{\infty}\frac{T_{0i}}{2\pi}\delta_i e^{-\frac{\theta_i}{T_{0i}}t}\sin\frac{2\pi}{T_{0i}}t$$
$$U'(0,t) = 100e^{M-8.5}\left(1-\frac{\Delta^2}{R^2}\right)\sum_{i=1}^{\infty}\frac{2\pi}{T_{0i}}\delta_i e^{-\frac{\theta_i}{T_{0i}}t}\sin\frac{2\pi}{T_{0i}}t \qquad (12)$$

where: T_{0i} is the period of free oscillations in the i-th form of the oscillation, [8], θ_i is the energy absorption coefficient for the i-th form of the oscillation of the earth's thickness, and by δ_i and h_k the following is denoted:

$$\delta_i = \frac{\sum_{k=1}^{n}\int_{h_{k-1}}^{h_k}\rho_k U_{ki}(z)dz}{\sum_{k=1}^{n}\int_{h_{k-1}}^{h_k}\rho_k U_{ki}^2(z)dz}, \qquad (13)$$

$$h_k = \sum_{i=0}^{k} H_i, \qquad h_0 = 0, \quad h_1 = H_1, \quad h_n = H$$

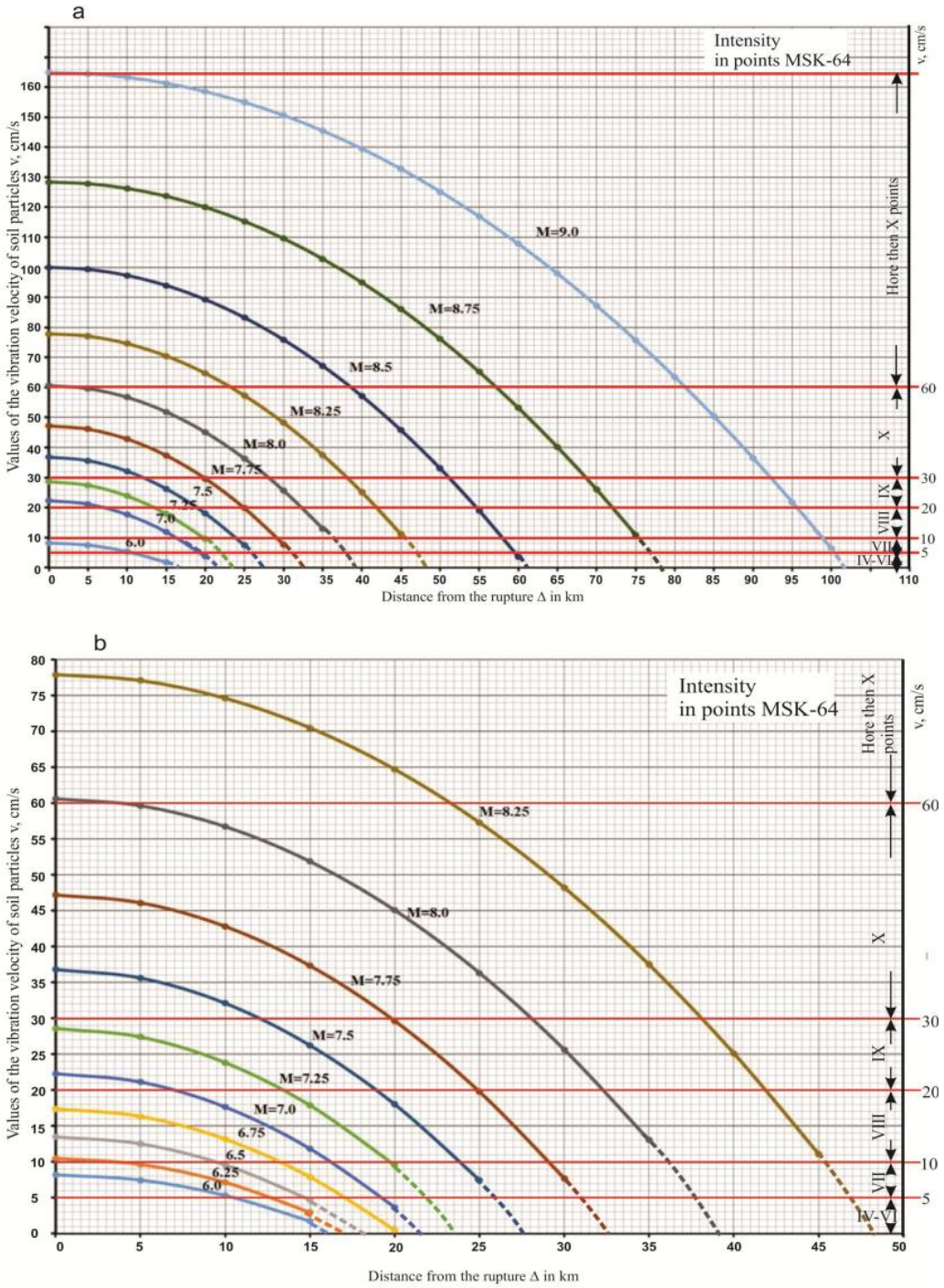

Fig. 1. Dependences of the rates of velocity of soil particles from the epicentral distance Δ and intensity on the scale MSK-64, for different magnitudes.
a.-for magnitudes 6.0≤M≤9.0, b.- for magnitudes of 6.0≤M≤8.25.

The method for determining functions $U_{ki}(z)$ and T_{01} is given in [8]. As an example in Fig. 3. the calculated according to formula (12) for single-layer (Fig.2.b) solid (T_{01}=0.25 s) and loose (T_{01}=0.8 s) bases, taking into account the three forms of oscillations: at M = 7.0, Δ = 5 km, V_{max}=22.3cm/s, $\theta_1 = \theta_2 = \theta_3 = 0.3$, for solid soils, $\theta_1 = \theta_2 = \theta_3 = 0.2$, for loose soils, and $T_{0i}=T_{01}/(2i-1)$, i=2.3… Using the same formula (12) and taking into account tree forms of vibrations at T_{01}=0.45 s we have U_{max}=2.0 cm, U''_{max}=0.88 g, and at T_{01}=1.2 s.- U_{max}=5.5 cm, U''_{max}=0.36 g.

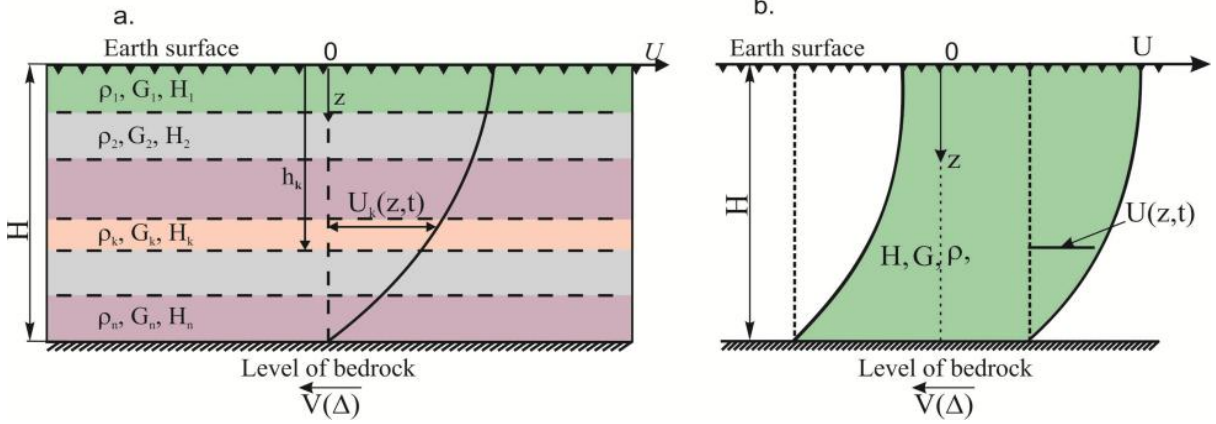

Fig. 2. Schematic representation of the deformed medium after the formation of a break - earthquake. H is the total thickness of the subsurface thickness, v (Δ) - is the initial velocity of blocks motion at distance Δ from the rupture, U_k (z, t) is the horizontal displacement function in the direction perpendicular to the rupture, ρ_k, G_k, H_k is the density, the shear modulus and the power of the k-th layer, respectively; a. In the form of a heterogeneous multi-sided base, b. In the form of a homogeneous single-sided base.

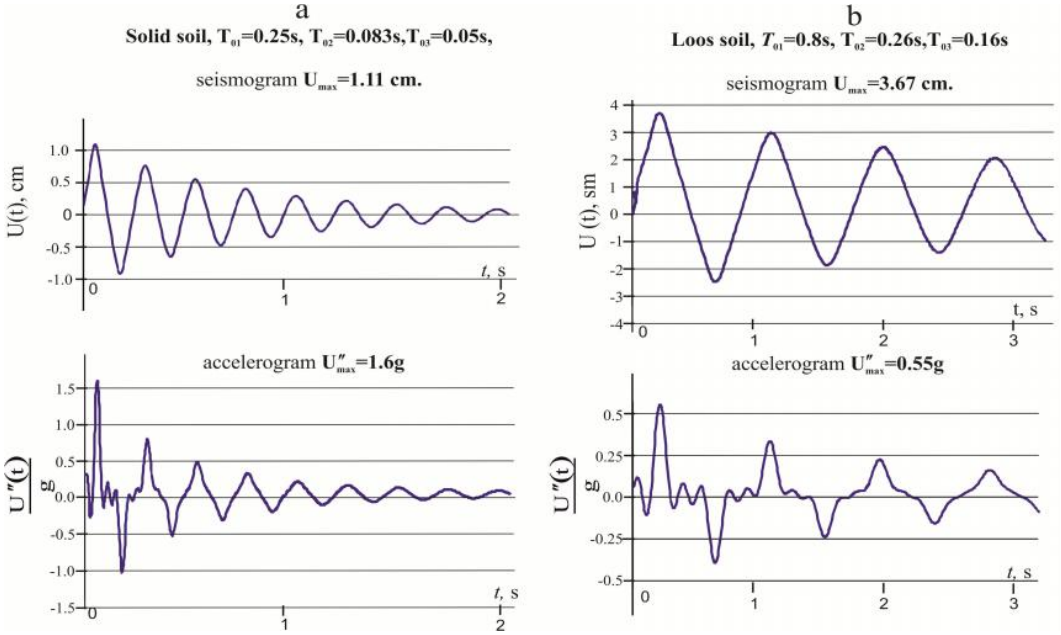

Fig.3. Seismograms and accelerograms for solid and loose soils calculated using formula (12), taking into account three forms of oscillations for M = 7.0, Δ = 5 km, v_{max} = 22.3 cm/s, $\theta_1 = \theta_2 = \theta_3 = 0.3$ for solid soils, $\theta_1 = \theta_2 = \theta_3 = 0.2$ for loose

soils.

5. Conclusion

1. A technique was developed for determining the initial velocities of transverse seismic vibrations of soil particles near a rupture and at a certain distance from it, depending on the magnitude, when considering an earthquake as an instantaneous rupture of the earth's crust.
2. The influence of the rupture duration on the values of the ground vibration velocities was estimated in comparison with the case of instantaneous rupture.
3. The values of the ground velocities established by the developed method are compared with their values on the MSK-64 seismic scale, which showed a sufficient correlation for earthquakes with an intensity of V-X scores and magnitudes $6.0 \leq M \leq 8.0$.

4. The ways of using the obtained values of ground motion velocities for obtaining synthetic seismograms and accelerograms are shown taking into account the local soil conditions of the construction site and depending on the earthquake magnitude and epicentral distance.

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EXISTING RESIDENTIAL BUILDINGS' MANAGEMENT IN ARMENIA AND APPLICATION OF BIM

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Abstract

This article is an overview of the ways of introduction and operation of Building Information Modeling (BIM) in Industrial countries with big heritage of old multi-apartment residential buildings needing systematic and capital maintenance. Particular attention is being paid to the experience of the former Soviet States that has similar background, but different paths of development followed by the collapse of the Soviet Union. The issue is particularly critical for the Republic of Armenia that has big heritage of residential buildings necessitating management and enhancement of seismic resilience. Six main components of the issue were introduced that are critical for development of institute of residential building management and may be sustained and developed by the means of information technologies and introduction of BIM.

1. Introduction

Many industrial countries in the world have huge inventory of the buildings built in mid decades of 20th century. There is a common problem for these countries to maintain safe operations of these buildings and infrastructure [1] and at the same time save public funds. This article focuses on the use of Information Technologies and management mechanisms for resolving maintenance issues of the old residential buildings. This problem is particularly important for the Republic of Armenia (RA), where earthquake resilience is critical requirement. This concern is significant for RA and from the urban development point of view, and from the national security.

After independence the Government of RA has privatized significant part of state owned residential building. However, it was not clarified whether the State or the private owners should carry out the current and capital repairs of the buildings (basement, roofs, commonly used spaces and technologies). In accordance with current urban development standards (which were set up after the 1988 Spitak earthquake) the old residential buildings have to be reinforced to comply the requirements of the new standards; the repair regularity and priority have to be set up based on the life cycle (LC) studies; responsibility for the reinforcement should be clarified; etc. Of course this list could go on expanding, covering all the aspects of the old building Facility Management (FM).

We see introduction of BIM-Building Information Modeling as a necessary component for facilitating process of FM for storing and communicating easily big amount of data about constructions in general and old residential buildings in particular. Digitalization and applications of information and communication technologies (ICT) are now in almost every aspects of our life. ICT as mean for assisting the FM processes, enhancing efficiency in communication between stakeholders and reducing costs is in the core of the multi-direction measures forming the field. We propose following six components (fig.1) as the above mentioned vital measures that constitute aspects of the old residential building issue.

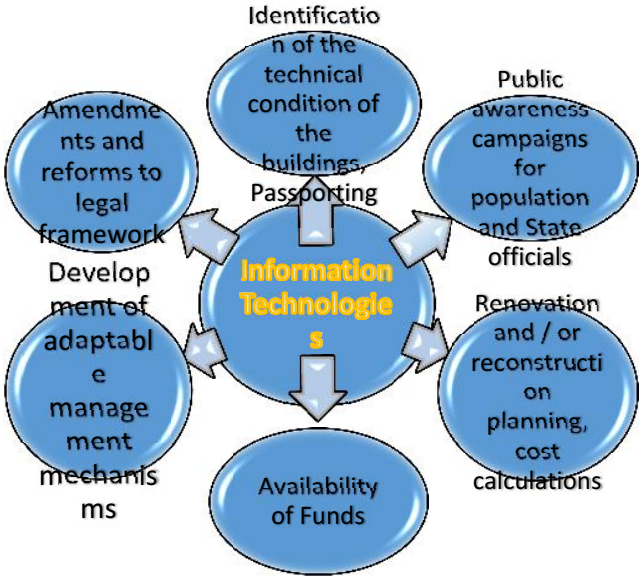

Fig. 1 Six constituents of Residential Building Management

2. Methodology

As mentioned the problem is common for many countries and some countries have found very educational, smart and interesting solutions. As a first step we present the overview of the international experience of old residential building management in former three soviet states (Latvia, Lithuania and

Russia) for: (a) adapting the best solution in RA, and / or (b) developing new ways and solutions.

Second part will be developed introduction of BIM as an application to construction business in general.

3. Overview Of International Experience

The issues connected with the management of the old multi-apartment residential buildings as: (1) planning and implementation of capital and periodic repairs, (2) data collection and registration of technical condition of the buildings, (3) management of the residential buildings, and other issues are common for the former Soviet republics as inheritors of the former practice and understanding of FM. Here we present the experience of three former soviet states: Latvia, Lithuania and Russia, their approaches and solutions to the Residential Building Management (RBM).

3.1 Latvia -Sandra Geypelen in her research paper devotes to the Republic of Latvia has accurately presented that the understanding of ownership is not being perceived only by the transfer of the certificate verifying ownership, but by the understanding of the great responsibility, which in realty the real estate ownership includes. [2] "Taking into account the fact that during the Soviet times, there was no ownership of the apartment, and then quickly offering apartments to a planned privatization, in the society there did not develop an understanding and responsibility for the property, especially for the property in multi-storey apartment buildings. Similarly, the understanding of the property ownership did not facilitate the transfer of the ownership rights of residential property using privatization certificates, without explaining the potential owner the burden of responsibility of having the property in the ownership." This statement is true and for the case of the RA as the adoption of the ownership doesn't assume acceptance and perception of the entire spectrum of its responsibilities arising from the ownership. The authors of the article highlight the importance of the institute of management for the residential buildings under the laws and regulations of the State. This institute in Latvia has considerable influence in initiating the repair and reconstruction work [3], but it is still a matter of discussion the method of assessment and evaluation of the repair works and sustainable development of the emerging residential buildings' managers market [4]. Iveta Pukiten and card Geypelen articles [5] point out that the problem of residential buildings has a major impact on the whole society. Latvia has developed chaotic management of these building process [3] and had a great impact on the economic and social spheres, and thus clarify that the issue should take into account not only the interests of the private owners, but entire society to ensure healthy and safe accommodation requirement.

3.2 Lithuania-In Republic of Lithuania the issues of management, maintenance, repair and reconstruction residential buildings are included into the program of energy efficiency improvement for the buildings, which [6] was implemented in three phases. The first phase (1996-2004) [7], the World Bank and the donor community conducted $ 28.6 million US dollars investment for energy efficiency improvement pilot project. In addition, technical assistance was provided for forming and development of energy audit markets, establishment of owners unions, for bank officials training and formation of the so-called development housing agency officials, the goal of the latest was planned to be promotion of energy efficiency investments in housing. Thus, the elements of infrastructure were created for management of residential building in Lithuania.

The second phase [7] (2005-2010) began by adoption of the State strategic plan on repairs of multi-apartment buildings in 2004 after the adoption of the Lithuanian Housing Strategy Project. The project includes commercial banks for investments in the sector in engagement and in support with subsidies provided by the State. The subsidies rose from 15 percent to 30 percent and then reached the 50% threshold. Dramatical increase of the State subsidies was conditioned by the outbreak of the financial crisis. At this stage, banks were reluctant to invest own resourced taking and risks for providing credit facility.

The third stage [7] (2010-2013), Joint European Support for Sustainable Investment in City Area-JESSICA program was introduced, which provided € 227 million of EU structural funds and the state budget. Initially, the project was developing slowly. Implementation of the new program EnerVizija model accelerated introduction of launching a large-scale repairs (a) by the municipalities and (ii) managers/administrators of residential buildings who initiated repairs and became borrowers of loans, instead of apartment owners. The project was supported by state subsidies of up to 15 percent. Further accelerating the process of promotion new tools were introduced, including an additional 25 percent subsidized by the Climate Change Fund and 3% fixed interest rate of JESSICA funds. These changes accelerated the process of modernizing buildings in Lithuania. Prior to the third phase gains momentum 70 building were renovated annually in Lithuania. The taken measures increased up to 490 renovated buildings annually. In May 2013, the subsidy procedure for low-income homeowners was revised. All this simplifies the process of decision making for repairs, especially low-income homeowners. Summarizing the experience of the Republic of Lithuania should be noted that mechanisms and institutions have been developed with the combination

of public, private and business resources. 97 percent of Lithuanian residential buildings are privatized and only 3% is of the municipal property. [7] Only 4 percent of the 35,000 residential buildings built before 1993, were repaired within the framework of this reform

3.3 Russia-The European Bank for Reconstruction and Development and the International Finance Corporation conducted a study on capital repairs of the residential buildings of Russian Federation (RF) [8] titled Financing Capital Repairs and Energy Efficiency Improvements in Russia (1), where is stated that the capital repairs are urgent priority for residential buildings in Russia. About 60 percent of residential buildings of RF need urgent repairs according to the regulations of the country on urban planning. International experience shows that the sustainable housing renovation and modernization policy of the government depends on the precise definition of goals and objectives, long-term strategic plan. And concluded that "the problems of maintenance and repairs of apartment buildings should be settled based on the principles of private property and the rule of law" [8].

This brief review allowed us to conclude that in financial support can be serious measure for solving residential building issues along with innovative management methods.

4. Introduction of BIM

The modern urban development has the trend of transition from industrial development of the cities, which is recognized as non-sustainable development phase, to the smart and green cities as sustainable development stage [9]. This assumes modernization of the urban development functions, restructuring and reorganization of the complete cycle: design, construction, operations, maintenance and demolition, with the applications of ICT.

This development trend of the industry has been adopted in some extent by 60-70 countries and was titled BIM (Building Information Management/ Modeling). BIM is the innovation in the urban development concepts by introducing new technologies, polices and regulations, which changes the results of the work, the relationships in the industry and beyond. Building Information Modeling (BIM) is the process that creates and manages buildings' physical and functional characteristics by ICT. BIM are files, which can be exchanged in the network, putted into circulation, as support for decision-making. It involves the dissemination of information at different levels and sectors. There are many definitions of BIM. Here are a number of them:

"Building Information Modeling (BIM) is a digital representation of physical and functional characteristics of a facility. A BIM is a shared knowledge resource for information about a facility forming a reliable basis for decisions during its life-cycle; defined as existing from earliest conception to demolition". *National BIM Standard- United States* [10]

"BIM describes the means by which everyone can understand a building through the use of a digital model which draws on a range of data assembled collaboratively, before during and after construction. Creating a digital Building Information Model enables those who interact with the building to optimize their actions, resulting in a greater whole life value for the asset." *National Building Specifications, UK* [11]

Besides these two [10, 11] official definitions, there are many other definitions of BIM. Some definitions pay great attention to the environmental impact of the buildings or the buildings' owners or inhabitants needs in information during the whole life cycle. BIM is defined as a way of ensuring a smooth process throughout the LC of the building with the use of information technology. As a result the residents will have more safe and more effective management with less environmental impact [12]. In other words BIM is the informational infrastructure for the buildings, which may be available to interested parties linked to the structure. BIM is a methodology which integrates a digital description of the building components and their relationships (fig.2). Accurate and complete information about the buildings will help the users to execute queries, electronic experimental simulations, calculations and other activities, examining the effects on the facility during life cycle. Consequently, BIM can provide, building owners and residents with valuable information for decision making, for increasing LC and operational stability.

BIM can be seen from a narrow and a broader perspective. BIM in a narrow sense comprises solely the digital building model itself in the sense of a central information management hub or repository and its model creation issues [14]. From the broader perspective organizational and legal issues, functional and LC issues, informational issues are included in the BIM. Commercial BIM platforms offer integrated data management, component libraries and general functionalities [15]. Widespread differentiations of BIM are 3D (spatial model with quantity takeoff), 4D (plus construction scheduling) and 5D (plus cost calculation) BIM [15].

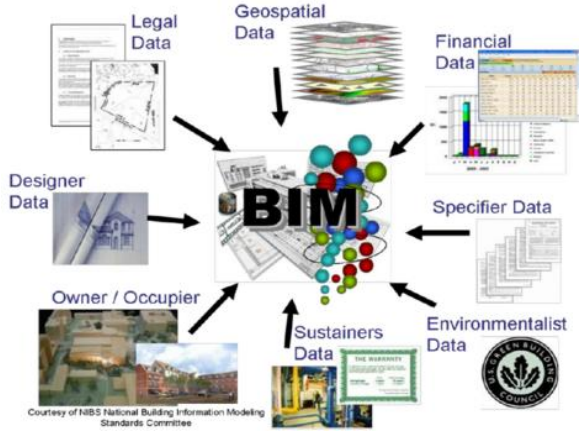

Fig. 2 Communication, collaboration and visualization with BIM model (NIBS, 2008) [13]

5. Conclusion

BIM adoption in the Construction Industry is seen as an important step and facilitating tool for resolving the particular, but vital and huge issue existing in Armenia on residential building management. Due to the big number of buildings that were built before the modernization of the seismic standards, and based on the recent programs developed by the Urban Development Committee on passporting the residential buildings we see BIM implementation as an important and inevitable constituent of the industry developments. The BIM usage and implementation in new structures (without even titling it as BIM) is still limited, but exist is some extent. However, the structured BIM implementation in the new buildings is still missing, as after construction stakeholders access to information is limited yet. Owners, facility managers and related consultants are hardly involved in BIM functionality development [16] worldwide.

Although on the one hand, implementation of BIM both in new and existing buildings induces profound changes of processes and information flows, on the other hand it accrues considerable advantages [14]. Potential BIM functionalities and benefits in existing buildings are numerous. Calculation of alternatives and optimizations seem promising to enhance project management and risk mitigation or to limit costs and duration of FM or deconstruction measures, e.g. in complex buildings or infrastructures [14].

It has to be mentioned that to our knowledge there isn't research on comprehensive cost-benefit analysis of BIM implementation for existing buildings.

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ON INTERACTION OF CRACK TYPE STRESS CONCENTRATORS IN AN ELASTIC PLATE AND AN ABSOLUTELY RIGID THIN INCLUSION INDENTED INTO THE PLATE TAKING INTO CONSIDERATION THEIR ADHESION

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Abstract

A plane problem of the mathematical theory of elasticity for the study of stress concentration in an infinite elastic plate containing a crack of finite length and an absolutely rigid thin inclusion, indented into the lower edge of the crack, is considered. We assume that the inclusion has the form of a strongly oblate along its length ellipse or a thin rectangle with height much shorter than its length. We consider that the inclusion is indented into the central part of the crack's lower edge without reaching the crack's tips; adhesion forces in the contact zone are taken into account.

Under these assumptions, using the Fourier integral transform, solution of the stated problem is reduced to the solution of a singular integral equation (SIE). The kernel of this equation is represented as a sum of the Cauchy kernel and regular kernel. The governing SIE is solved by a well-known method based on Gauss quadrature formulas.

We investigated effects of interactions of crack type stress concentrators and the thin inclusion, when inclusion endpoints are approaching the crack's tips. In this case, stress intensity factors significantly increase.

The problem main physical characteristics: contact stresses under inclusion, the crack opening and stress intensity factors at the crack tips are represented by explicit formulas of simple structure.

1. Introduction

The need to study the interaction of various types of stress concentrators, especially cracks and thin inclusions, with massive deformable bodies is dictated by their theoretical and practical significance.

Around these concentrators in the bodies, local stress concentration fields are formed with large and intensively changing gradients. Theoretically, their modeling leads to new classes of mixed boundary-value problems of the mathematical theory of elasticity, representing the development and generalization of classical boundary-value problems. Practically, stress concentration problems are often encountered in calculating the strength characteristics of various engineering structures in construction mechanics, in mechanical engineering, especially in aircraft building, and in calculating composite materials in composite mechanics. Stress concentrations often arise because many construction materials contain, by their technological and constructive manufacturing standards, foreign inclusions, which during the operation of structures can come off from the elastic matrix, forming a crack. Similar stress concentration problems also arise in geomechanics in the study of rock displacements, when landslides of soils occur.

Because of its topicality, the problems of stress concentration in deformable bodies caused by different types of stress concentrators have become the subject of research by many authors. In this connection, we indicate the works of Selvadurai and Singh [1], Hwu, Liang, and Yen [2]. In the papers of Berezhnitsky and Staschuk [3, 4], the problem on the crack growth in the vicinity of the rigid inclusion vertex was investigated and SIFs near the crack on the continuation of the linear rigid inclusion were determined. Various cases of a combination of cracks and rigid inclusions in a homogeneous and piecewise-homogeneous elastic matrix were considered in monograph by Hakobyan [5]. Many results on this research area are summarized in Cherepanov's monographs [6, 7], as well as in the handbooks edited by Murakami [8], and of Savruk [9].

In monograph by Muskhelishvili [10], as well as earlier in Sherman's paper [11], an exact solution of a mixed boundary-value problem for an elastic plane with an arbitrary finite number of collinear cracks (gaps) is constructed based on complex potentials when on the upper edges of the cracks the components of external stresses are given and on the lower edges – the displacement components. A special case of one crack is considered when the crack upper edge is free from external stresses, and an absolutely rigid strip-inclusion is indented into the crack lower edge along its entire length under the action of a centrally vertical force [10]. In the present paper, we consider this problem for the case of a crack of finite length and normal discontinuity from the point of view of fracture mechanics. It is assumed that a thin absolutely rigid inclusion is indented into the central part of the crack lower edge along the symmetric segment,

which is shorter than the crack full length, and adhesion forces are taken into account in the contact zone.

We have studied the posed problem by the mathematical methods of the integral Fourier transform, the theory of SIEs in combination with the well-known numerical-analytical method for solving the SIF [12, 13] based on quadrature formulas Gauss equations for computing ordinary and Cauchy-singular integrals. The main mechanical characteristics are represented by explicit analytical formulas containing integrals of oscillating functions. An effective way of calculating such integrals is proposed. The regularities of the change in the SIF and J –integral are studied when the end points of the inclusion are approaching the crack tips. Meantime, a phenomenon is revealed: applying this approximation or at the limiting transition, SIF and the J – integral become infinitely large reaching their critical values, at which a crack initiation and its subsequent spontaneous propagation occur. As a result, at such a limiting transition, the crack will spread earlier than the end-points of the inclusion will reach the crack tips. Therefore, it is necessary to clarify the previous formulation of the problem and, from the point of view of fracture mechanics, to consider incomplete contact of an inclusion with the lower edge of the crack, as done in the present paper.

We note that the problem on a single crack similar to the considered in this paper was previously studied in the paper by Bardzokas and Mkhitaryan [14] for the case when there are no tangential forces in the contact zone.

2. Formulation of the problem and derivation of basic equations

Assume that an infinite elastic plate with the elastic modulus E, the Poisson's ratio ν and the height h, referred to a rectangular coordinate system $0xy$, has a crack (mathematical cut) on the horizontal axis along the line segment $[-a,a]$ (Fig.1). Let normal forces of intensity $p_+(x), (p_+(-x)=p_+(x))$, act on the upper (+) edge of the crack, the tangential forces be equal to zero, a rigid thin inclusion be indented into the crack lower edge (-) along its central part $[-b,b](b<a)$ under the action of the centrally vertical force, P. The contacting surface of inclusion is described by the equation $y=f(x)$, where $f(x)>0$, $f(-x)=f(x)$ and $\max f(x) \ll b(-b \le x \le b)$. This can be, in particular, the central part of the half-ellipse, which is strongly oblate in the vertical direction

$$y=f(x)=d-\frac{a}{c}\sqrt{c^2-x^2} \quad (d \ll c; -b \le x \le b; b<c<a),$$

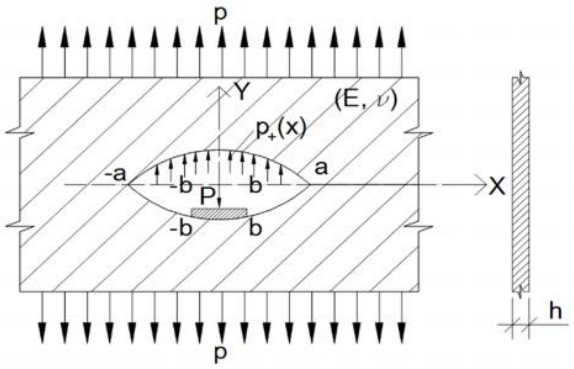

Fig. 1 Equilibrium of an elastic infinite plate containing a crack and an absolutely rigid thin inclusion

or a strip of constant height δ, i.e. $f(x)=\delta$ $(\delta \ll b, -b \le x \le b)$. In addition, we assume that the plate at infinity is stretched in the vertical direction perpendicular to the crack line by uniformly distributed forces of intensity p.

Assuming that the plate is in the generalized plane stress state, the inclusion is adhered to the plate along the crack lower edge segment $[-b,b]$, that means adhesion forces act in the contact zone, it is required to determine: the tangential and normal contact stresses, $\tau_-(x)$ and $p_-(x)$ $(-b \le x \le b)$, respectively; the density of dislocations on the crack edges; the opening of the crack; tangential $\tau(x)$ and normal breaking $\sigma(x)$ stresses outside the crack on its line of location, i.e. when $|x|>a$ and $y=0$; SIF and J – integral. Note that unknown functions $\tau_-(x)$ and $p_-(x)$ must satisfy the condition of adhesion

$$|\tau_-(x)/p_-(x)|<\rho \quad (-b<x<b),$$

where ρ is the coefficient of dry friction between contact bodies.

Having begun to derive the basic equations of the posed problem, we cut the infinite plate along the horizontal axis into the upper (+) and lower (-) elastic half-planes and introduce the following notation

$$\sigma_y\big|_{y=\pm 0} = -\sum\nolimits_{\pm}(x) = \begin{cases} -p_{\pm}(x) - p & (|x|<a); \\ -\sigma(x) & (|x|>a); \end{cases}$$

$$\tau_{xy}\big|_{y=\pm o} = -T_{\pm}(x) = \begin{cases} -\tau_{\pm}(x) & (|x|<a); \\ -\tau(x) & (|x|>a), \end{cases}$$

where σ_y and τ_{xy}, are, respectively, the normal and tangential stresses in the plate, referred to the unit of its height h, $\tau_{+}(x)\equiv 0$ when $|x|<a$, and $\tau_{-}(x)=p_{-}(x)\equiv 0$ when $b<|x|<a$. Further proceeding from complex potentials for the case of presence of concentrated forces on the boundary of the elastic half-plane [10, p.343] and from the linear superposition principle, we get the following expressions for complex combinations of derivatives of boundary point horizontal $(u_{\pm}(x))$ and vertical $(\nu_{\pm}(x))$ displacements corresponding to the upper and lower elastic half-planes:

$$w_{\pm}(x)=u'_{\pm}(x)+iv'_{\pm}(x)=\vartheta_1\int_{-\infty}^{\infty}\frac{\Omega_{\pm}(s)ds}{s-x}+ \\ +2i\vartheta_2\Omega_{\pm}(x)-Q \quad (-\infty<x<\infty) \tag{1}$$

$$\Omega_{\pm}(x)=T_{\pm}(x)+i\sum\nolimits_{\pm}(x);\ Q=\nu p/E;$$

$$\vartheta_1=2/\pi E;\ \ \vartheta_2=(1-\nu)/2E.$$

Now in (1) we introduce the functions

$$w(x)=\frac{w_{+}(x)+w_{-}(x)}{2};\ \Phi(x)=\frac{w_{+}(x)-w_{-}(x)}{2};$$

$$\Omega(x)=\frac{\Omega_{+}(x)+\Omega_{-}(x)}{2};\ \mathrm{X}(x)=\frac{\Omega_{+}(x)-\Omega_{-}(x)}{2}$$

and note that due to the continuity of stresses and displacements outside the crack, i.e. for $|x|>a$ and $y=0$, $\Phi(x)=\mathrm{X}(x)\equiv 0$. After introducing these functions in (1), we apply to this equation the integral Fourier transform with respect to the variable x and get the system of linear algebraic equations. We express the Fourier images of functions $w(x)$ and $\Omega(x)$ from this system through the Fourier images of the remaining functions $\Phi(x)$ and $\mathrm{X}(x)$. Then we apply the Fourier inversion formula and use the inversion formula for a SIE with a Cauchy kernel on a finite interval. As a result, after simple transformations and using the contact condition of the inclusion with the elastic matrix, we arrive at following governing SIE of posed problem with respect to the complex combination of unknown contact stresses under the inclusion in the dimensionless form:

$$\frac{1}{\pi}\int_{-k}^{k}\frac{\omega(\eta)d\eta}{\eta-\xi}-2i\nu_0\omega(\xi)+\frac{1}{\pi\sqrt{1-\xi^2}}\int_{-k}^{k}\frac{\sqrt{1-\eta^2}\,\omega(\eta)d\eta}{\eta-\xi}= \\ =g(\xi)\ \ (-k<\xi<k);\ g(\xi)=\frac{\nu_0\left(P_0-P_0^{+}\right)+2i\pi p_0\xi}{\pi\sqrt{1-\xi^2}}-p_0- \\ -if'(a\xi)+\frac{i}{\pi}\int_{-1}^{1}\frac{(\eta+\xi)p_{+}^{0}(\eta)d\eta}{\sqrt{1-\xi^2}\left(\sqrt{1-\eta^2}+\sqrt{1-\xi^2}\right)}\ \ (\nu_0=(1-\nu)/2). \tag{2}$$

Here we introduce dimensionless quantities $\xi=x/a$, $\eta=s/a$; $\omega(\xi)=\tau(\xi)+ip(\xi)$;

$\tau(\xi)=\tau_{-}(a\xi)/E$, $p(\xi)=p_{-}(a\xi)/E$; $p_{+}^{0}(\xi)=p_{+}(a\xi)/E$;

$p_0=p/E$; $P_0=P/aE$; $P_0^{+}=P^{+}/aE$; $k=b/a$,

where P_0^{+} is the resultant of normal forces $p_{+}^{0}(x)$. Note that the solution of SIE (2) must satisfy the inclusion equilibrium condition

$$\int_{-k}^{k}\omega(\xi)d\xi=iP_0. \tag{3}$$

Now the main characteristics of the problem can be expressed through the solution of the governing SIE (2)-(3). Namely, for the density of dislocations on the crack edges $\varphi_0(\xi)=\varphi(x)=\varphi(a\xi)$ we get

$$\varphi_0(\xi)=\frac{1}{\pi E\sqrt{1-\xi^2}}\int_{-k}^{k}\frac{\sqrt{1-\eta^2}\,\omega(\eta)d\eta}{\eta-\xi}-i\nu_0\omega(\xi)- \\ -\nu_0 p_{+}^{0}(\xi)-\frac{\nu_0\left(P_0-P_0^{+}\right)+2i\pi p_0\xi}{\pi\sqrt{1-\xi^2}}+ \\ +\frac{i}{\pi\sqrt{1-\xi^2}}\int_{-1}^{1}\frac{\sqrt{1-\eta^2}\,p_{+}^{0}(\eta)d\eta}{\eta-\xi}\ \ (-1<\xi<1). \tag{4}$$

For dimensionless crack opening $\Psi_0(\xi)=\Psi(a\xi)/a$ we have

$$\Psi_0(\xi)=\frac{1}{\pi}\int_{-k}^{k}K(\xi,\eta)\omega(\eta)d\eta+\frac{i}{\pi}\int_{-1}^{1}K(\xi,\eta)p_{+}^{0}(\eta)d\eta- \\ -i\nu_0\int_{-k}^{k}\mathrm{sign}(\xi-\eta)\omega(\eta)d\eta-\nu_0\int_{-1}^{1}\mathrm{sign}(\xi-\eta)p_{+}^{0}(\eta)d\eta- \\ -\frac{2\nu_0}{\pi}\left(P_0-P_0^{+}\right)\left(\frac{\pi}{2}+\arcsin\xi\right)+4ip_0\sqrt{1-\xi^2}\ \ (|\xi|\le 1); \tag{5}$$

$$K(\xi,\eta)=\ln\frac{1-\xi\eta+\sqrt{\left(1-\xi^2\right)\left(1-\eta^2\right)}}{1-\xi\eta-\sqrt{\left(1-\xi^2\right)\left(1-\eta^2\right)}}.$$

For dimensionless breaking stresses $\tau_0(\xi)=\tau(a\xi)/E$ and $\sigma_0(\xi)=\sigma(a\xi)/E$ we get:

$$\tau_0(\xi)+i\sigma_0(\xi)=\frac{1}{\sqrt{\xi^2-1}}\left[\frac{\operatorname{sign}\xi}{2\pi}\int_{-k}^{k}\frac{\sqrt{1-\eta^2}\,\omega(\eta)d\eta}{\eta-\xi}+\right.$$
$$+\frac{i\operatorname{sign}\xi}{2\pi}\int_{-1}^{1}\frac{\sqrt{1-\eta^2}\,p_+^0(\eta)d\eta}{\eta-\xi}-\frac{\nu_0\left(P_0-P_+^0\right)}{2\pi}\operatorname{sign}\xi- \qquad (6)$$
$$\left.-\frac{ip_0}{|\xi|+\sqrt{\xi^2-1}}+\frac{\nu_0}{2\pi}\sqrt{1-\xi^2}\int_{-1}^{1}\frac{p_+^0(\eta)d\eta}{\eta-\xi}\right]\quad(|\xi|>1).$$

SIFs are defined by the following formulas in terms of the functions $\varphi(x)$ from (4)

$$K_{II}^{\pm}+iK_I^{\pm}=\mp\frac{2G}{\chi+1}\lim_{x\to a\mp 0}\left[\sqrt{2\pi(a\mp x)}\;2\varphi(x)\right].$$

After introducing dimensionless SIFs, these formulas can be represented in the form

$$\bar{K}_{II}^{\pm}+i\bar{K}_I^{\pm}=\int_{-k}^{k}\sqrt{\frac{1\pm\eta}{1\mp\eta}}\;\omega(\eta)d\eta+i\int_{-1}^{1}\sqrt{\frac{1\pm\eta}{1\mp\eta}}\;p_+^{(0)}(\eta)d\eta\pm$$
$$\pm\nu_0\left(P_0-P_0^+\right)+2\pi ip_0 \qquad (7)$$
$$\bar{K}_I^{\pm}=\frac{2\sqrt{\pi}}{E\sqrt{a}}K_I^{\pm};\quad \bar{K}_{II}^{\pm}=\frac{2\sqrt{\pi}}{E\sqrt{a}}K_{II}^{\pm}$$

Finally, the expressions of J - integrals (the rates of release of the elastic body potential energy during the propagation of a crack) are given as

$$J_{\pm}=\frac{1+\chi}{8G}\left[\left(K_I^{\pm}\right)^2+\left(K_{II}^{\pm}\right)^2\right]\quad\left(\chi=\frac{3-\nu}{1+\nu}\right)$$

and in dimensionless form as

$$J_{\pm}^0=\left(\bar{K}_I^{\pm}\right)^2+\left(\bar{K}_{II}^{\pm}\right)^2;\quad J_{\pm}^0=\frac{4\pi J}{aE}. \qquad (8)$$

3. The numerical-analytical solution of the governing SIE (2)-(3)

In order to apply the above-mentioned numerical-analytical method for solving SIE to (2) - (3), we write them in standard form. For this purpose we put

$$\xi=kt,\quad \eta=ku;\ \nu_0=th(\pi\mu)\quad\left(\mu=\frac{1}{2\pi}\ln\frac{3-\nu}{1+\nu}\right)$$

and transform SIE (2) into

$$-ith(\pi\mu)\omega_0(t)+\frac{1}{\pi}\int_{-1}^{1}\frac{\omega_0(u)du}{u-t}-\frac{k^2}{2\pi}\int_{-1}^{1}K_0(t,u)\omega_0(u)du=$$
$$=g_0(t)\ (-1<t<1);\ \omega_0(t)=\omega(kt),g_0(t)=\frac{1}{2}g(kt), \qquad (9)$$
$$K_0(t,u)=\frac{u+t}{\sqrt{1-k^2t^2}\left(\sqrt{1-k^2t^2}+\sqrt{1-k^2u^2}\right)}\quad(-1<u,t<1)$$

and condition (3) – into

$$\int_{-1}^{1}\omega_0(t)dt=iP_0/k. \qquad (10)$$

The solution of SIE (9) can be represented [10] in the form

$$\omega_0(t)=(1-t)^{-\frac{1}{2}-i\mu}(1+t)^{-\frac{1}{2}+i\mu}X_0(t)\quad\left(\mu=\frac{1}{2\pi}\ln\frac{3-\nu}{1+\nu}\right) \qquad (11)$$

where $X_0(t)$ is the function of the Hölder class on an interval, and, generally speaking, $X_0(\pm1)\neq0$. When $t\to\pm1$ the function $\omega_0(t)$, oscillating an infinitely number of times, changes its sign, i.e. when $t\to\pm1$ oscillation of dimensionless contact stresses under the inclusion occurs.

Let us separate the oscillation part of the solution

$$\omega_0(t)=\varphi_\mu(t)X_0(t)=\left[X_0(t)-At-B\right]\varphi_\mu(t)+$$
$$+(At+B)\varphi_\mu(t);\ \varphi_\mu(t)=(1-t)^{-\frac{1}{2}-i\mu}(1+t)^{-\frac{1}{2}+i\mu}\quad(|t|<1), \qquad (12)$$

where A and B are constants. We choose these constants in such a way that the following conditions will be fulfilled: $\left[X_0(t)-At-B\right]\Big|_{t=\pm1}=0$,

whence

$$A=\frac{1}{2}\left[X_0(1)-X_0(-1)\right],\quad B=\frac{1}{2}\left[X_0(1)+X_0(-1)\right].$$

Further assuming

$$\varphi_0^*(t)=X_0(t)-At-B\qquad(\varphi_0^*(\pm1)=0);$$
$$\psi_0(t)=\varphi_0^*(t)(1-t)^{-i\mu}(1+t)^{i\mu}\quad(-1<t<1),$$

we write formula (12) in the form

$$\omega_0(t)=\frac{\varphi_0(t)}{\sqrt{1-t^2}}+\frac{(At+B)(1-t)^{-i\mu}(1+t)^{i\mu}}{\sqrt{1-t^2}}\quad(-1<t<1). \qquad (13)$$

It is evident that oscillations of the function $\psi_0(t)$ decay when $t\to\pm1$. Now oscillations enter only into the second summand of formula (13).

Further, we substitute the representation (13) in the SIE (9) and in the condition (10). After calculating the elementary integrals associated with the second term in (13) and simple transformations, we arrive at the following SIE for function $\psi_0(t)$:

$$-ith(\pi\mu)+\frac{\sqrt{1-t^2}}{\pi}\int_{-1}^{1}\left[\frac{1}{u-t}-\frac{k^2}{2}K_0(t,u)\right]\frac{\psi_0(u)du}{\sqrt{1-u^2}}=$$
$$=\sqrt{1-t^2}\left[g_0(t)+Ah_1(t)+Bh_2(t)\right](-1<t<1), \qquad (14)$$

where $g_0(t)$, $h_1(t)$, $h_2(t)$ and $K_0(t,u)$ are known functions. The condition (10) will take the form

$$\int_{-1}^{1}\frac{\psi_0(u)du}{\sqrt{1-u^2}}=\frac{iP_0}{k}-\frac{\pi}{\operatorname{ch}(\pi\mu)}(2i\mu A+B). \qquad (15)$$

Now, following the procedure of the above-mentioned numerical-analytical method for solving SIE, we reduce

equations (14) - (15) to the following system of linear algebraic equations (SLAE):

$$\begin{cases} -i\mathrm{th}(\pi\mu)\psi_0(t_r)+\sum\limits_{m=1}^{M}\dfrac{\sqrt{1-t_r^2}}{M}\left[\dfrac{1}{u_m-t_r}-\dfrac{k^2}{2}K_0(t_r,u_m)\right]\times \\ \times\psi_0(u_m)=\sqrt{1-t_r^2}\left[g_0(t_r)+Ah_1(t_r)+Bh_2(t_r)\right]\ \left(r=\overline{1,M-1}\right); \\ \sum\limits_{m=1}^{M}\dfrac{\pi}{M}\psi(u_m)=\dfrac{iP_0}{k}-\dfrac{2\pi i\mu}{\mathrm{ch}(\pi\mu)}A-\dfrac{\pi}{\mathrm{ch}(\pi\mu)}B \end{cases} \quad (16)$$

$$u_m=\cos\left[\frac{(2m-1)\pi}{2M}\right]\left(m=\overline{1,M}\right);\ t_r=\cos\left(\frac{\pi r}{M}\right)\ \left(r=\overline{1,M-1}\right).$$

Here M is any natural number, u_m and t_r are Chebyshev nodes, i.e. roots of Chebyshev polynomials of the first kind $T_M(u)$ and of the second kind $U_{M-1}(t)$, respectively.

The values of $\psi_0(t_r)$ are determined from the Lagrange interpolation polynomial at Chebyshev nodes, which gives the best approximation of function $\psi(t)$ [15, p.596].

Now all the characteristics of the problem under consideration (4) - (8) with the help of Gaussian quadrature formulas at the Chebyshev nodes can easily be expressed in terms of the SLAE solution (16).

In the conclusion of the section, we again turn to the first integrals in the formulas for SIF (7) and, in accordance with (12), we represent $\omega_0(t)$ in the form

$$\omega_0(t)=\frac{\Omega_0(t)}{\sqrt{1-t^2}},\ \ \Omega_0(t)=\mathrm{X}_0(t)(1-t)^{-i\mu}(1+t)^{i\mu}\ \ (-1<t<1).$$

Then we write these integrals in the form

$$G_{\pm}(k)=k\int_{-1}^{1}\sqrt{\frac{1\pm ku}{1\mp ku}}\,\frac{\Omega_0(u)\,du}{\sqrt{1-u^2}}\qquad\left(\Omega_0(\pm1)\neq0\right)$$

and pass to the limit $k\to1$. It is evident that $\lim\limits_{k\to1}G_{\pm}(k)=\infty$.

4. Results

The following main results are obtained in the paper:

a) an effective solution of the mixed boundary-value problem on the interaction of stress concentrators of inclusion and crack types with an elastic infinite plate is constructed when incomplete contact of the inclusion with the elastic matrix occurs;
b) the main characteristics of the problem are represented by explicit analytical formulas convenient for numerical analysis;
c) it is shown that the SIF and J – integrals increase infinitely, when the end points of inclusion tend to the crack tips.

5. Conclusion

In paper:

a) the regularities of the change in SIF and J – integrals are studied from the point of view of fracture mechanics;
b) the effect is revealed: when the end points of inclusion infinitely approach the crack tips the SIF and J – integrals increase infinitely, reaching their critical values at which the crack initiation and its subsequent unsteady propagation occur;
c) the formulation of the mixed boundary-value problem discussed earlier by other authors was refined from the point of view of fracture mechanics by consideration of incomplete contact.

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THE STUDY OF CRACK PROPAGATION IN THE BODY OF CONSTRUCTION USING THE METHOD OF SECTIONS

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Keywors: disintegration, tension, deformation, crack, sections.

Abstract

The coefficient of stress intensity plays a strong role in the study of fracture propagation existing in the body of structure, by goal of receiving it the study problem of determination of body stress-deformation state becomes priority. The solution of problem, the theory of elasticity sometimes is very difficult by analyzing methods and sometimes even impossible which depends on the shape of the edge of discussed domain and border conditions. This way it's very modern to work out methods for solving concrete applied problems by application of approximate methods. The study of plain problem of border crack propagation of massive concrete dam in conditions of large-scale cavity is observed in the work which was provided by hydro-energetic reason inside the body. In the case of the flat problem of endless body, by the method of Vestergard, the image of the stressed state in the area of the tip crack was received, taking into account the gravity force, and by presenting the direction of crack propagation by vertical direction in dangerous section by similarity of continuous functions chosen for tensions. By application of the method of sections and satisfying the condition of corresponding balance, the length of stimulated section in the area of crack tip is received. By means of the similitude of the coefficient of tension intensity, the graph of its dependence upon it, is built by means of which in case of the concrete crack size length of crack propagation can be determined. The veracity of received results is well-grounded for several cases by means of comparison of data for several cases.

1. Introduction

It is known that all the construction materials, including the concrete, have different types of preliminary cracks inside their body and surface, which during the work of the structure can achieve a chance of distribution, or disintegration of structure takes place and a state of emergency may appear. In the point of view of disintegration of mechanics the singular parameter of tension intensity in the tip of the crack is observed as a coefficient of tension intensity, the determination of which is possible to solve by using approximate methods.

2. Methodology

As a first important condition in the study of crack propagation, the solution of problem decision of tension-deformation body is put. This problem in some cases is solved by an analytical method but, more often, by using different digital methods and by opportunities of computer techniques. It should be noted that the application of digital methods requires a lot of work. In the point of view of solving applied problems is more practical the application of approximate methods. Among those methods, the method of section is more practical, which was applied during the realization of this work.

3. Results

We have the following marginal conditions for solution of the flat problem of the concrete dam.

a) in the pressure edge of dam

$$\sigma_x = -\gamma_w \cdot y, \tau_{yx} = 0 \quad (x = 0, 0 \le y \le H) \tag{2}$$

b) in the level of base

$$U = 0, \; v = 0 \quad (0 \le x \le b, \; 2b \le x \le B)$$

c) on the walls of cavity

$$\sigma_x = \tau_{xy} = 0$$

$$(x = b, 0 \le y \le h; \; x = 2b, \; 0 \le y \le h)$$

$$\sigma_x = \tau_{xy} = 0 \quad (b \le x \le 2b, \; y = h), \tag{3}$$

d) $\sigma_x = \tau_{xy} = 0 \quad (0 \le x \le 2b, \quad y = H),$ (5)

in the unpressed, oblique session of dam

$$\sigma_\vartheta = \tau_\vartheta = 0 \tag{4}$$

The acting in the base section of dam will emerge from water pressure.

$$M = \gamma_w \cdot \frac{H^3}{6} : \tag{6}$$

The normal tension, acting in the level of the dam base, will be the sum of the components emerging from the torque and the powers of body weight. (7)

$$\sigma = \sigma\,(M) + \sigma(Q):$$

Here Q - is the power of own weight of the body dam.

$$Q = S \cdot \gamma c$$

S-is the material surface of the dam section. So, the normal tension, acting in the level of the dam base, will be. (8)

$$\sigma = \frac{M}{I_z} \cdot x - \frac{Q \cdot X_Q}{I_z} \cdot x - \frac{Q}{A} \quad (9)$$

A-is the surface of section base, I_z-is the moment of inertia of that axe section.

4. Conclusion

The coefficient of tension intensity is determined by the following formula [1].

$$K_i = \sigma\sqrt{\pi l} \cdot \zeta\left(l/B\right), \quad (10)$$

where l-is the length of the crack, B-is the width in the direction of propagation, the coefficient ζ is determined by the table [1]. σ_y tensions is determined in the direction of the crack are determined by [2]

$$\sigma_y = \sigma \cdot \frac{x}{\sqrt{x^2 - l^2}}, \quad x > l;$$

σ_y normal tension acting on the y=0 line we can write like this:

$$\sigma_y(x,0) = \frac{K_I}{\sqrt{\pi l} \cdot \zeta\left(\frac{l}{B}\right)} \cdot \frac{x}{\sqrt{x^2 - l^2}}, \quad x > l:\text{f}$$

For applying the method of sections, first of all, we determine th the oblique moment acting in the y=0 section, taking into account the water pressure and the power of own weight of dam. The sizes of structure are accepted as B=4b, B=0.8H, b=0.2H, h=0.1H:

While choosing the link of measures we have taken into consideration the existence of the cavity inside, because of which B was increased, by reason of securing the horizontal sliding stability of dam. [3].

The formula of tension determination in the arbitrary point of dam base, gets the following final view.

$$\sigma = \frac{\gamma_q}{0.3138} \cdot x - 1.2832 \cdot \gamma_c \cdot x - 0.666 \cdot H \cdot \gamma_c \quad (13)$$

The normal power lost because of the cleft of the crack, is compensated by additional efforts emerging in the stimulated domain of the area of the top.

$$\int_0^l \sigma dx = \int_0^a \frac{K_I}{\sqrt{\pi l} \cdot \zeta\left(\frac{l}{B}\right)} \cdot \frac{x}{\sqrt{x^2 - l^2}}, \quad x > l \quad (14)$$

Here, σ written in the left part, is the tension determined by the formula (13), a-is the length of the stimulated range emerged in the area of cracks' tip, which is determined by the following. (16)

$\frac{K_I}{\sqrt{\pi a}} = \sigma | x = 0.433H - l$It is received

$$a = \frac{K_I^2}{\pi \cdot A^2(l)},$$

where by A(l) the expression of the equation is appointed in the right part.

Forming the balance condition in the base level for determination of K_I, the expression, besides other parameters, is received which depends also upon the preliminary greatness of the crack size.

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GEOMETRICAL ANALYSIS OF CAVITY DEFECTS BEHIND LINING IN HIGH-SPEED RAILWAY TUNNEL

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Keywords: tunnel lining, lining defect, lining thickness, cavity geometry cavity, probability of cavity

Abstract

Analysing the quality defects on-site data of tunnel through the probability and mathematical statistics analysis, the distribution quality defects and diseases of lining are found, the general law of lining behind contact state is statistically analyzed, and the geometric distribution characteristics of cavity behind lining and the safety of the structure are obtained. The conclusion is that the strength of lining concrete is not the main defect of lining, and lack of thickness and presence of cavity behind it are important factors affecting the safety of the tunnel structure. The conclusion of this paper is meaningful to study the safety effect of tunnel structure.

1 Introduction

With the advance of the urbanization process, the demand for monitoring and monitoring of the tunnel traffic network is increasing. In the face of the huge test data, it is an important research topic to analyze the scattered data. Therefore, it is necessary to systematically and comprehensively analyze the lining defects of tunnel lining structure in order to formulate targeted and reasonable measures for tunnel lining disposal, so as to effectively control the occurrence and development of tunnel defects and prolong the service life of tunnels, Master the actual operating state of the tunnel.

2 The main types of tunnel lining defects in high - speed railway

When the lining is in close contact with the concrete, the lining structure can withstand the pressure of the surrounding rock, and it can produce sufficient formation reaction force, so that the lining structure is in a good state of force, and when the lining is in poor contact, the stress of the lining structure is deteriorated Conditions, making the lining of the local stress concentration, serious lead to the lining of the structure of cracking, loss of pieces and other diseases [1]. In order to fundamentally correct the tunnel, it is important to make clear the main types of tunnel lining contact. The internal types of the lining of the tunnel include: insufficient lining strength, insufficient thickness and voids.

3 Statistical analysis of lining concrete strength

The strength of the concrete lining is measured and the statistics are shown in Table 1:

Table 1.Statistical table of lining concrete strength

Item / Name	Number	Lining concrete strength		
		Qualified	Unqualified	Pass rate
Tunnel 1	9	8	1	88.89%
Tunnel 2	23	21	2	91.30%
Tunnel 3	17	16	1	94.12%
Tunnel 4	11	10	1	90.90%
Tunnel 5	16	14	2	87.50%
Tunnel 6	19	17	2	89.47%
Tunnel 7	14	13	1	92.86%
Tunnel 8	9	9	0	100.00%
Tunnel 9	6	5	1	83.33%
Tunnel10	7	6	1	85.71%
Tunnel11	12	11	1	91.67%
Total	143	130	13	90.91%

It can be concluded that the lining of the tunnel is better, and the strength of the lining concrete is basically 90.91%. Obviously, the lining concrete strength is not the main defect factor to detect the lining state of the tunnel.

4 Statistical analysis of the thickness of lining and the geometrical characteristics of voids behind lining

Tunnel design specifications require tunnel lining and the surrounding rock close fit, in fact, there is often between the lining and the surrounding gap or between the two backfill is not dense, this phenomenon is called the lining behind the cavity. Professor ZHANG in the Ministry of Railways key scientific research project "railway operation tunnel disease comprehensive treatment technology research and safety evaluation" support, access to the Beijing-Kowloon line, Tan Long line, Beijing-Guangzhou line, the sea line, Chengdu line, South Xinjiang and other railway lines on more than 100 tunnel geological radar non-destructive testing data, statistical analysis of defects in the tunnel with the defect in the detection of the proportion of the tunnel from 11.56% [2]. It should be pointed out that in the tunnel construction process, due to the lack of lining thickness, the design thickness and the actual thickness of the difference, the construction of the collapse of the disposal is not complete, and left behind the lining of the phenomenon is quite common.

4.1 Statistical analysis of the thickness of lining and the geometrical characteristics of voids behind lining

The mean average u_k, standard deviation σ_k and coefficient of variation δ_k of the k (a) ~ (c) of the tunnel lining are analyzed:

$$u_k = \frac{1}{n}\sum_{i=1}^{n} u_i \quad \text{(a)}$$

$$\sigma_k = \frac{1}{n-1}\sqrt{\sum_{i=1}^{n}(u_i - u_k)^2} \quad \text{(b)}$$

$$\delta_k = \frac{\sigma_k}{u_k} \quad \text{(c)}$$

According to the statistical results, the calculation is shown in Table 8:

Table 2. Statistical table of lining length characteristic result in each defect level for Zhang Wu lining

Characteristics / Defect levels	u_k			σ_k			δ_k		
	vault	hance	sidewall	vault	hance	sidewall	vault	hance	sidewall
1	21	22	17	5.657	7.071	1.414	0.269	0.321	0.083
2	13	7	1	8.485	1.784	1.350	0.653	0.202	1.510
3	2.5	0.5	0	2.121	0.707	0.707	0.849	1.204	1.312
4	0	0	0	0.000	0.000	0.000	0.000	0.000	0.000

Table 3. Statistical table of lining length characteristic result in each defect level for Gao Shan lining

Characteristics / Defect levels	u_k			σ_k			δ_k		
	vault	hance	sidewall	vault	hance	sidewall	vault	hance	sidewall
1	20.9	13	7.8	5.798	9.899	3.253	0.277	0.761	0.417
2	13.8	12.5	2.5	1.697	3.536	3.536	0.123	0.283	1.414
3	4	1.5	1.85	1.414	2.121	2.616	0.354	1.414	1.414
4	0	0	0	0.000	0.000	0.000	0.000	0.000	0.000

Analysis of the tunnel lining defects can be made as follows:

(1) Located in the vault and arch at the lining of the lining of a defect level 1, defect 2 and defect 3 level more, should focus on monitoring and remediation.

(2) The general rule of the mean of the length of the defective lining is located at the top of the vault > arch side > wall, and the position of the lining is the most serious.

(3) The general rule of variation coefficient of length of defective lining basically satisfies the variability of vault > arch side > wall, which indicates that the variation range of defective lining is larger in the dome.

4.2 Statistical analysis of lining thickness

Lining thickness is not associated with the back of the hole is usually linked, according to ensure the normal use of lining and driving safety requirements, the lining thickness is divided into minor, more serious, serious and very serious four categories.

Table 4 Table of lining imperfect defect quantitative indicators

Defect levels / Severity	1 Slight	2 More severe	3 Serious	4 Very seri
$1 > h_l / h \geq 0.9$	L		-	-
$0.9 > h_l / h \geq 0.75$	$L < 5$	$L \geq 5$	-	-
$0.75 > h_l / h \geq 0.60$	-	$L < 5$	$L \geq 5$	-
$h_l / h < 0.60$	-	-	$L < 5$	$L \geq 5$

Among them: h_l for the measured the thickness of the lining; h for design thickness; L for detect the in sufficient thickness of lining section of the line length in a row.

The first detection of the first Lao Ling tunnel were detected left and right dome, left and right arch waist and left and right, the edge of the six lines. Due to the need to avoid the tunnel within the catenary and the next anchor and other ancillary facilities, the length of each side line is not the same. On the lining thickness of the defect level analysis, the survey results of the test line in Table 5 to Table 7:

Table 5. Table of left vault lining thickness

Length of the line / Defect levels	Cumulative length/m	Percentage
1	17	21.79 %
2	19	24.36 %
3	1	1.28 %
4	0	0.00 %

Table 6. Table of left arch waist lining thickness

Length of the line / Defect levels	Cumulative length	Percentage
1	27	17.88 %
2	6	3.97 %
3	1	0.66 %
4	0	0.00 %

Table 7. Table of left wall lining thickness

Length of the line / Defect levels	Cumulative length	Percentage
1	18	11.76 %
2	2	1.31 %
3	1	0.65 %
4	0	0.00 %

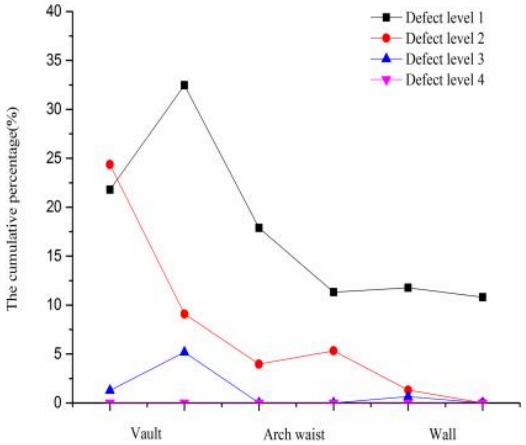

Fig.1 Diagram of defect statistical for ZhangWu tunnel lining

From Table 2 to Table 7, we can draw the following understanding:

(1) The first and second Lao Ling tunnel vault at the location of the lining behind the situation is relatively serious, the edge of the wall at the location of the lining behind the situation, the arch at the location of the lining behind the empty situation is less Detection of 4 defects.

(2) The lining defects of the third ridge tunnel are mainly concentrated in the vault and arch position, and the measured thickness of the dome is less than 60% and the design thickness is less than 60%.

4.3 High probability analysis of voids behind lining under different positions

Fig.2 Height characteristic statistical graph of cavity at the vault position

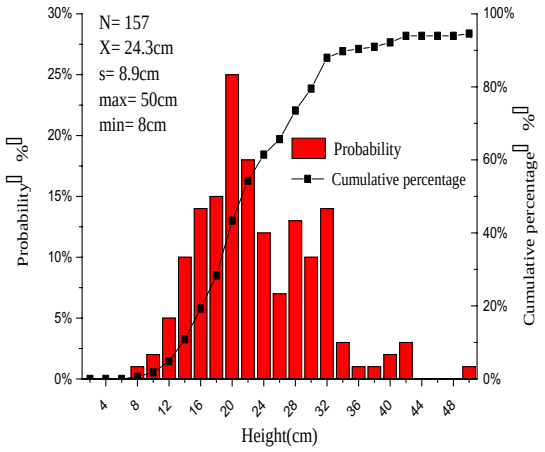

Fig.3 Height characteristic statistical graph of cavity at the arch waist position

From Figure 2 to Figure 3, the following conclusions can be drawn:

(1) The distribution of lining thickness at each location showed a more obvious normal distribution in the range of 8cm ~ 24cm.

(2) 69% of the arch measuring points, 18% of the wall measurements are found empty. This shows that the side wall is fit, not compact degree of small.

5 Analysis of Cavity Probability in Lining Holes under Different Positions

According to the current test data, tunnel lining defects are prevalent in the position of vault, arch and side wall, and the length of defect of tunnel lining is 12.16% of the total length of tunnel. Therefore, there is a hole in the tunnel lining is one of the common quality defects.

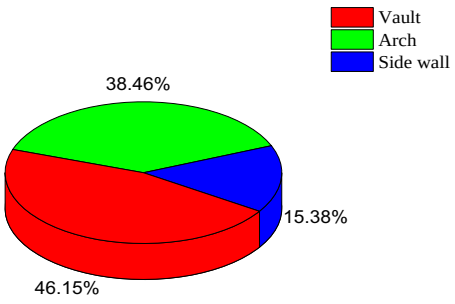

Fig.4 The quantity distribution ratio graph of cavity in different position

From Figure 4 can be drawn, the vault position behind the lining of the situation is relatively serious, arch waist position lining the back of the hole after the situation, while the edge of the wall lining the situation behind the lightest situation, therefore, should focus on the arch lining to do the focus of remediation.

6 Conclusion and suggestion

The study of more than 10 railway operating tunnel lining contact state, from the mathematical statistics analysis obtained the following understanding:

(1) The existence of voids in the tunnel lining is a more common quality defect phenomenon. The lack of lining thickness and the existence of holes are important factors that affect the safety of tunnel structure.

(2) The probability and geometric mean, standard deviation and discrepancy of the contact defects in the back of the lining are gradually reduced by the order of the vault, arch and side wall.

(3) The distribution characteristics of the lining thickness are related to the distribution characteristics of the lining behind the lining.

(4) Obviously, the ratio of lining thickness or concrete strength to its design value, if smaller, the greater the degree of impact on the lining capacity of the tunnel lining. Similarly, the longer the length of the lining line is longer (or the longer the continuous length of the lining of the concrete is less), the greater the degree of impact on the lining capacity of the tunnel lining.

Acknowledgements

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ASSESSMENT OF ENERGY SAVING POTENTIAL THROUGH IMPLEMENTATION OF THERMAL INSULATION OF RESIDENTIAL APARTMENTS

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Keywords: thermal insulation materials, residential buildings, environment impact.

Abstract

The implementation of energy saving thermal insulation alternatives is crucial aspect for Armenia. It is conditioned by the fact of absence of mineral energy sources in the country and lack of potential to develop and introduce new technologies of alternative source of energy.

Such an important sector which is energy efficiency of buildings, in comparison with other sectors of economy has biggest share of social-economic impacts (according to statistic that is up to 40%) as well as great potential for energy and resource saving. Under the present conditions of increasing tariffs over energy sources and necessity for reduction of energy consumption, the introduction of efficient thermal insulation of technologies becomes priority issue.

The paper considered application of alternative thermal insulation material for the construction of residential buildings.

Residential buildings account for 74% of global building energy use and non-residential buildings (notably commercial buildings) for 26%. From 2002 to 2012, however, energy use in non-residential buildings grew at a faster rate (22%) than in residential buildings (12%).

Energy saving potential of RA is estimated over 1 mln. tons of equivalent fuel from which only 16.5% come from basic sectors of economy. By implementation of thermal insulation of the buildings the 40% of energy saving potential will be applied.

For the thermal insulation purposes the research considers the application of aerated concrete based on the waste material accumulated from mining and cutting of volcanic rocks (tuff).

The use of aerated concrete with 500-600 kg / m^3 density, allows constructing buildings in average climate zone with the wall thickness 20-24 cm and a mass 120-180 kg /m^2, which is up to two times less in comparison with lightweight concrete of 1100 kg / m^3 density, applied in production of prefabricated houses.

Case study: Assessment of the energy saving potential for households in Erebuni district of Yerevan city.

Estimate of energy consumption has been conducted for residential apartments (1, 2, 3 rooms)

Paper present calculation of energy consumption in winter seasons for per capita based on findings of survey and energy saving calculation for residential apartments with application alternative thermal insulation material like AAC. Research result shows that appropriate thermal insulation of buildings will reduce energy consumption for heating purposes by having positive impact of income for per capita.

1. Introduction

Increasing energy efficiency in all consuming sectors plays a vital role in coping with future energy challenges.

Residential buildings are among the main consumer of final energy. While only some new buildings benefit from model energy performance, the majority of saving potential must be realised in the vast building stock already in existence.

Worldwide, countries and regions have embraced increasingly forceful measures and support programmes to help improve the performance of the existing building stock. However, the level of success is far from satisfactory as a comprehensive approach to building refurbishment is frequently not aimed at.

The implementation of the measures for increasing energy efficiency in the building sector will result in lower energy consumption of the building and, at the same time, better comfort of living and performance of the building. The preference of measures depends on the energy performance and type of building, its purpose and location, and the best option would be to implement several measures ensuring their synergistic effect and to achieve more notable energy savings [1].

Global energy efficiency investment in buildings is projected to increase to over USD 125 billion (excluding appliances) by 2020. This level, however, falls far short of the estimated USD 215 billion needed by that time to achieve the climate change mitigation goals set out in the 2-Degree Scenario (2DS) of the International Energy Agency (IEA).

A key challenge for the future is that the global buildings market is large and highly disaggregated. Decisions on energy efficiency are taken by multiple players – governments, industry and consumers – based on different needs and goals, and reflect diverse incomes, climate conditions, habits, etc.

Residential buildings account for 74% of global building energy use and non-residential buildings (notably commercial buildings) for 26%. From 2002 to 2012, however, energy use in non-residential buildings grew at a faster rate (22%) than in residential buildings (12%). Non-residential buildings often provide a good opportunity for energy efficiency investment, as the commercial activities undertaken typically require more active management of revenues and expenses, and decisions about per-unit energy consumption are more centralized [2].

The implementation of energy saving thermal insulation alternatives is crucial aspect for Armenia from the prospective of energy efficiency and economic impact on energy production sector.

In general, buildings in Armenia were built before 1987 and as such they do not have adequate thermal protection. As many as 83% of the buildings do not satisfy even the 1987 technical regulations, they have high heat losses with the average energy consumption ranging from 150 to 200 kWh/m^2, which classifies them in energy efficiency class E! Increased energy consumption implies higher emissions of CO2 in the atmosphere and it is, therefore, essential to implement the required measures in order to reduce excessive consumption and streamline the utilisation of available energy generating products.

The energy efficiency approach for building sector with regard to technology level however considers 2 main aspects:

- the thermal properties of the building envelope components that reflect its enhanced thermal performance level and is defined by high thermal resistance values -R [m^2 ^oC /W] ;
- efficiency and management of the energy end use systems.

The paper focused on the first aspect of energy efficiency approach for building sector through application of alternative thermal insulation material for the construction of residential buildings. For this purpose the research considers the application of aerated concrete based on the waste material accumulated from mining and cutting of volcanic rocks (tuff).

Assessment of the energy saving potential for households in Erebuni district of Yerevan city has been conducted, i.e. the results of thermal performance design with the enhanced energy efficiency have been compared for the prototype building designed with the old Soviet approach – an un-insulated house version that represents the current construction practice in Armenia.

2. Thermal insulation alternative - Aerated concrete based on the waste material

In general, aerated concrete is quite simple for exploitation and easy to handle, so it facilitates building of walls of complex configuration. Besides its head insulating features aerated concrete also has quite strong sound insulation qualities, it is fireproof and environmentally friendly material so it will not burn or give off toxic fumes. The cost of aerated blocks sufficiently competitive with brick, wood and other wall materials costs.

It has been refined into a high-thermal insulating concrete-based material used for both internal and external construction. Currently, aerated concrete items are the most common building materials with over 300 plants worldwide. In CIS countries the technology of AAC production was introduced in 1950s and led to large-scale manufacturing in Belarus, Ukraine and Russia.

2.1 Advantages of the proposed AAC

- For the production of proposed AAC the wastes of volcanic rocks (tufo) mining will be applied.

The question of integrated use of non-metallic raw materials especially volcanic rocks (tufo) is quite urgent nowadays. It conditioned with large volume of tufo extraction and production waste formed in process of mining and treatment of this volcanic rocks. In Armenia the volume of volcanic waste exceeds 2 million m^3/p.a. and make of 60% of overall mined rock mass. The most of production waste is formed basically due to natural fissuring of rocks (up to 40%) and fines formation that take place in process of rocks cutting during its mining and treatment (20%-25%). Currently, over 100 million m^3 volcanic production wastes occupy a large territory of Armenia that deteriorates ecological and economic conditions of the country. The occupation of usable lands for disposal of non-metallic industry wastes may lead to irrecoverable consequences [2].

- The use of aerated concrete with 500-600 kg /m^3 density, allows constructing buildings in average climate zone with the wall thickness 20-24 cm and a mass 120-180 kg/m^2, which is up to two times less in comparison with lightweight concrete of 1100 kg/m^3 density (Table 1), applied in production of prefabricated houses. In result the cost of one m^2 produced wall is greatly reduced [3].
- AAC blocks weigh lesser, the dead load is lesser on the structure and hence the structure can be designed more efficiently for a lower load which has seismic significance

Table 1. Required tackiness of wall to ensure 2.2 thermal resistances for various materials

Raw material	Density, kg/m^3	Compressive strength, MPa	Thermal conductivity coefficient W/ m.^{0}C	Thickness of wall, m
Slag concrete	1400	10.0	0.405	0.88
	1300	9.0	0.407	0.77
	1200	7.5	0.349	0.66
Granulated concrete	950	15.0	0.211	0.42
	850	13.0	0.177	0.35
	780	7.5	0.162	0.31
	700	5.5	0.151	0.29
Aerated concrete	1000	15.0	0.326	0.61
	900	12.0	0.256	0.48
	800	9.0	0.209	0.40
	700	7.5	0.175	0.33
	600	5.0	0.116	0.22
	500	3.5	0.105	0.20
	400	2.0	0.093	0.18
Artik tufo	1500	-	0.465	0.88
Kosh tufo	1900	-	0.640	1.21
Hoktemberyan tufo	1800	-	0.582	1.10

3. Methodology and case study

Assessment of the energy saving potential for households in Erebuni district of Yerevan city has been conducted. Particularly, research present calculation of energy consumption in winter season for per capita [4] based on findings of survey and energy saving calculation for residential apartments with application alternative thermal insulation material like AAC.

Household survey has been implemented in accordance with number of rooms, in particular, dealt with 1, 2, 3 bedroom apartments

As research tools we have the results of social polls. Data analysis has been done based on handbooks of statistics of National Statistical Service of Armenia. Applied data cover the period from January 2015 to December 2015.

121 sample of questionnaires have be reviewed, 4% incomplete

Households have been selected up to 5th floor.

3.1 Determination of energy performance – basic principle of calculation [5, 6]

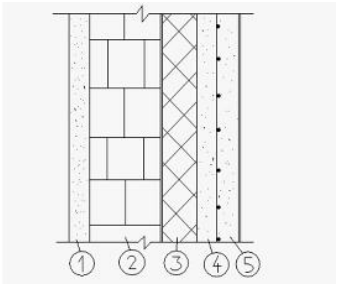
For the assessment of energy consumption in the residential buildings the application of three alternatives methods of building envelopment has been considered (Figure 1):

(i) one layer wall, Artik tuff (old Soviet approach – an un-insulated house version that represents the current construction)

(ii) one layer bearing wall with AAC

(iii) one layer bearing wall with polystyrene

For the comparison of consumed energy in the buildings



with application of abovementioned enveloping alternative, the technical characteristics of the building should be calculated.

Description of the layer	Calculated coefficient of thermal conductivity λ, Vt/m^2.°C	Thickness of the layer δ, m
Concrete	0,87	0,02
Artik tufo	0,55	1,49 or 0.4
Expanded polystyrene	0,042	0,13
Thermal insulation, AAC	0,093	0,18
Protective mortar	0,87	0,006

The coefficient of heat resistance for walls and ceilings have been calculated:

$$K_m = K_m^{\inf} + K_m^{tr} = 0,65 + 0,21 = 0,86 \; Vt/m^2 \cdot °C$$

$$R_w = \frac{1}{\alpha_в} + \sum \frac{\delta}{\lambda} + \frac{1}{\alpha_н} = \frac{1}{8,7} + \frac{0,22}{2.04} + \frac{0,1}{0,04} + \frac{0,01}{0,17} + \frac{0,02}{0,93} + \frac{0,005}{0,11} + \frac{0,03}{0,23} + \frac{1}{23} = 3.01 m^2 \cdot °C/Vt$$

Coefficient of thermal conductivity of the building is:

$$R_w = \frac{1}{\alpha_в} + \sum \frac{\delta}{\lambda} + \frac{1}{\alpha_н} = \frac{1}{8,7} + \frac{0,02}{0,87} + \frac{0.5}{0,55} + \frac{0,006}{0,87} + \frac{1}{23} = 1.1 m^2 \cdot °C/Vt$$

$$K_m^{inf} = \frac{0,28 \cdot c \cdot n_a \cdot \beta_v \cdot V_h \cdot \rho_a^{ht} \cdot k}{A_e^{sum}} = \frac{0,28 \cdot 1 \cdot 0,2 \cdot 0,85 \cdot 128.3 \cdot 1,3 \cdot 0,9}{89} = 0,08 \ Vt/m^2 \cdot °C$$

3.2 Definition of energy consumption characteristics

1. General building heat loss through the exterior building structures during the heating season (over 214 days) is:

$$Q_h = 0,0864 \cdot K_m \cdot D_d \cdot A_e^{sum} = 0,0864 \cdot 0,86 \cdot 4000 \cdot 3117,8 = 687600 MJ$$

2. Domestic heat gain during the heating season:

$$Q_h^y = [Q_h - (Q_{int} + Q_s) \cdot \nu \cdot \zeta] \cdot \beta_h = 140112 MJ$$

3. Heat gain from solar radiation for heating season:

$$Q_h^y = [Q_h - (Q_{int} + Q_s) \cdot \nu \cdot \zeta] \cdot \beta_h = 729,009 MJ$$

Thus thermal energy consumption of the building is:

$$Q_s = \tau_F \cdot k_F \cdot (A_{F.1} \cdot I_1 + A_{F.2} \cdot I_2 + A_{F.3} \cdot I_3 + A_{F.4} \cdot I_4) = 0,75 \cdot 0,83 \cdot (141 \cdot 71 + 123 \cdot 429) = 39\,079 \ MJ$$

Technical indicators				
№ п.п.	Description	Unit	Normative value of the index	Factual value of the index
1	Thermal resistance of external structures:	R_0^r, м^{2}·°C/Vt	-	
	Wall	R_w	3,16	4.01
	Windows and balcony doors	R_F	0,55	0,54
	Entrance doors and gates	R_{ed}	0,6	0,6
	Attic floors	R_c	4,12	4,18
	Above the basement floor	R_f	4,55	3.01
2	Resulted heat transfer coefficient of the building	K_m^{tr}, Vt/(m^{2}·°C)	-	0,4
3	Conditional heat transfer coefficient of the building	K_m^{inf}	-	0,2
4	The total heat transfer coefficient of the building	K_m	-	0,6

Based on the calculation, Table 3 provided winter time energy consumption for 1, 2, 3 bedroom apartments accordingly, with application proposed enveloping alternatives for the buildings.

Obtained results of the energy consumption in residential buildings have been applied for assessment of average cost of electricity in the winter time for Erebuni district with consideration of proposed alternatives.

Erebuni covers significant area of the city Yerevan, which is one of the reasons to conduce survey in this area (Tables 4, 5).

One-room apartment			
	One layer wall (tuff)	One layer bearing wall with AAC	One layer bearing wall with polystyrene foam
Energy consumption, Kvt/hour			
	2900	2180	2050
Two-room apartment			
	One layer wall (tuff)	One layer bearing wall with AAC	One layer bearing wall with polystyrene foam
Energy consumption, Kvt/hour			
	5890	4300	4300
Three-room apartment			
	One layer wall (tuff)	One layer bearing wall with AAC	One layer bearing wall with polystyrene foam
Energy consumption, Kvt/hour			
	9400	6730	6700

following numbers of apartments have be selected (Table 6):

Table 6 Number of apartments selected for assessment

One room apartment	29	24.2%
Two-room apartment	34	28.3%
three-room apartment	44	36.7%

Thus the average cost of electricity in the winter time and calculated savings from thermal insulation of building with application AAC for per capita will decrease 24-25% in comparison with old Soviet approach – an un-insulated house version that represents the current construction practice in Armenia (Table 7,8).

Table 7 Thus the average cost of electricity in the winter time and calculated savings from thermal insulation of building with application AAC

	Electricity (kVt)	Electricity (USD)	Savings (USD)	Saving with application of stone-walls and AAC, %
1	437	44	12.5	24.8%
2	600	61	15	27%
3	1009	100	29	28.4%

Conclusion

Paper present calculation of energy consumption in winter seasons for per capita based on findings of survey and energy saving calculation for residential apartments with application alternative thermal insulation material like AAC.

Research result shows that appropriate thermal insulation of buildings can reduce energy consumption for heating purposes by having positive impact of income for per capita.

Table 8 The average monthly cost of electricity per capita vs. average monthly net salary

The average monthly net salary (USD) 270	The average monthly cost of electricity per capita in winter (USD)	The average monthly cost of electricity per capita vs. average monthly net salary
One room apartment	12.5	4.7%
two-room apartment	15	5.6%
Three room apartment	21	7.7%

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THE FORECAST OF GROUNDWATER LEVEL UNDER INFLUENCE OF NATURAL AND ARTIFICIAL FACTORS OF STRUCTURES AND AREAS

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Keywords: Capillary rise, groundwater, moistening.

Abstract

Artificial groundwater recharge from utilized construction and reclamation areas was investigated. The main purpose of this paper was prediction and regulation of groundwater level. By comparing and investigating the results from the various theories, the theoretical expressions were obtained which is giving an opportunity to determine the level of groundwater recharge. This can be milestone in justification of drainage construction.

1. Introduction

The high level of groundwater or capillary water in aquifer is the threat for the safe operation of buildings and facilities. Due to the water losses from water mains (plumbing systems), pumping stations, water reservoirs and other water sources, the underground water level rises and often there is an emergence of capillary water. Finally emergence of water within underground parts of structures can cause groundwater flooding.

Nowadays determination methods of groundwater level is well elaborated in case of hydraulic constructions located in river basins, but not well developed in case of losses from underground water communications in residential areas and industrial construction sites.

The high level of groundwater has negative effect on industrial sites, urban and agricultural areas as for new constructions as for existing ones in terms of operation and maintaining. In case when engineering protection measures in not implemented properly the humidification of underground parts of constructions is occurring. In many cases the high level of groundwater causes decrease of structural strength of structures. The purpose of this paper is the elaboration of prediction methods for groundwater high level in case of infiltration feeding.

After the utilization of construction and reclamation areas extra percolation feeding is appearing from accumulated precipitations inside construction pits under the bases of constructions, losses from water mains and etc., which often lead to formation of techno-genic aquifers in aeration zone of grounds. From mentioned sources water percolation is appearing into the permeable grounds of aeration zone (lenses), the accumulated water is forming groundwater hump, which is leaking out from the surface of weak permeable layer. At the end of the accumulation process the groundwater stationary table is established, which is necessary to determine in the scope of prediction. The negative impact of the high level of ground water is usually required to take protective engineering measures. In order to protect the basis of construction from negative effect of groundwater (under the influence of natural and artificial factors) the theoretical prediction method is suggested and also for justification of drainage construction.

2.Methodology

The formation of techno-genic aquifer in aeration zone on weak permeable bases due to infiltration has the appearance of 2L wide zone in plan. In this case the groundwater formation process (Fig.1) can be explained with following expressions [2-4]:

$$k\frac{\partial}{\partial x}\left(h\frac{\partial h}{\partial x}\right)+w-\frac{k_p}{m_p}(h+m_p)=\mu\frac{\partial h}{\partial t};\qquad (0\le x\le L),\quad (1)$$

$$k\frac{\partial}{\partial x}\left(h_1\frac{\partial h_1}{\partial x}\right)-\frac{k_p}{m_p}(h_1+m_p)=\mu\frac{\partial h_1}{\partial t};\qquad (x>L);$$

(2)

$$\frac{\partial h(0;t)}{\partial x}=0;\qquad h(x;0)=h_1(x;0)=h_1(\ell;t)=0;\qquad (3)$$

$$h(L;t)=h_1(L;t);\qquad \frac{\partial h(L;t)}{\partial x}=\frac{\partial h_1(L;t)}{\partial x};\qquad \ell(0)=L;$$

$$\mu\frac{d\ell}{dt}=-k\frac{\partial h_1(\ell;t)}{\partial x},\qquad (4)$$

where the h and h_1 are the groundwater level corresponding I and II zones, k and k_p are filtration coefficients for the permeable and weak permeable grounds, w is the intensity of infiltration feed, μ is the coefficient of ground saturation deficit.

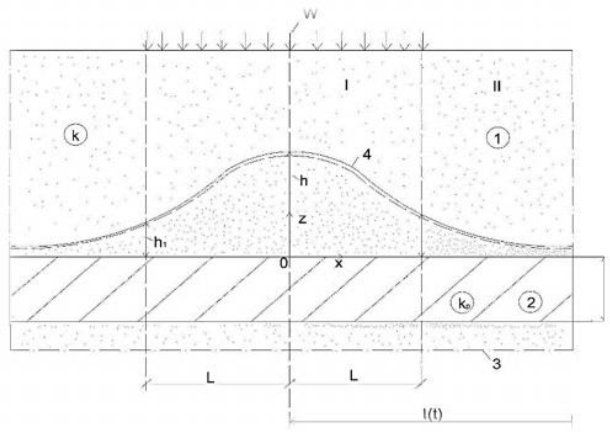

Fig.1 Scheme of groundwater emergence in aeration zone

1- aeration zone, 2 - weak penetrable layer, 3 – groundwater level, 4 - groundwater in aeration zone

In case of non-liner and undefined movement boundaries of equations, obtaining the right solutions for (1) and (2) are difficult. Accepting, the change in pressure for zone I in the form of full-time determined by the change of pressure on the boundary $x = L$, it means $\partial h_1 / \partial t = 0$ is taken into the equation (1). In zone II accepting that the increase of groundwater in aeration zone everywhere is same, so finally the pressure does not depend on the coordinates $\partial h_2 / \partial t = F(t)$.

After the linearization of equations (1) and (2) using N.N. Verigin methods[1], they will have following shape:

$$\frac{d^2h}{dx^2} - \lambda h = \varphi_0 , \qquad (5)$$

$$\frac{d^2h_1}{dx^2} - \lambda^2 h_1 = \varphi_1 : \qquad (6)$$

Given the above admissions for (1) - (4), the solution of equations of the system is as follows:

$$h(x) = \frac{\varphi_0}{\lambda}(ch\lambda x - 1) - \left[\frac{\psi_0}{\lambda}\frac{ch\lambda(\ell - L) - ch\lambda\ell}{ch\lambda\ell} + \frac{\varphi_1}{\lambda}\left(\frac{ch\lambda\ell - 1}{ch\lambda\ell} + \frac{2sh\lambda\ell sh\lambda L}{ch\lambda\ell}\right)\right]ch\lambda x \qquad (7)$$

$$h_1(x) = \frac{\varphi_1}{\lambda}\left[ch\lambda(x - L) - 1\right] + \frac{\varphi_0}{\lambda}sh\lambda L sh\lambda x - \left[\frac{\varphi_1}{\lambda}\frac{ch\lambda(\ell - L) - 1}{ch\lambda\ell} + \frac{\varphi_1 + \psi_0}{\lambda}\cdot\frac{sh\lambda\ell\cdot sh\lambda L}{ch\lambda\ell}\right]ch\lambda x \qquad (8)$$

where

$$\varphi_0 = \varphi_1 - \psi_0, \quad \varphi_0 = \frac{k_p - w}{kh_c} + \frac{\mu F}{kh_c} \quad , \lambda^2 = \frac{k_p}{km_p h_c},$$

$$\psi_0 = \frac{w}{kh_c} \quad \varphi_1 = \frac{\mu F}{kh_c} + \frac{k_p}{kh_c} \qquad (9)$$

The boundary condition of water flow across the lens, when $F(t) = 0$ is determined with following equation:

$$\ell = \frac{1}{\lambda}\arcsin\left[sh\lambda L - \frac{\lambda akt}{\mu}(sh\lambda L - \alpha)\right], \qquad (10)$$

where

$$\alpha = \frac{k}{\lambda\mu\sqrt{D}}, D = a_0^2 + 4\left[(1 - 2sh\lambda L)a_0 - \frac{2w}{kh_c}\right]c_0 sh\lambda L,$$

$$a_0 = \frac{\mu F + k_p}{kh_c}, \; c_0 = \frac{w}{kh_c} \qquad (11)$$

The highest elevation of groundwater in the center of zone, that is x = 0 to be determined:

$$h_{max} = -\frac{\psi_0}{\lambda}\frac{ch\lambda(\ell + L) - ch\lambda\ell}{ch\lambda\ell} - \frac{\varphi_1}{\lambda}\left[\frac{ch\lambda\ell - 1}{ch\lambda\ell} + \frac{2sh\lambda\ell\cdot sh\lambda L}{ch\lambda\ell}\right] \qquad (12)$$

Average groundwater depth is:

$$h_c = \frac{1}{L}\cdot\frac{\varphi_0}{\lambda^2}sh\lambda L - \frac{\varphi_0 L}{\lambda} + \frac{sh\lambda L}{\lambda L}\left[\frac{ch\lambda(\ell + L) - ch\lambda\ell}{ch\lambda\ell} + \frac{\varphi_1}{\lambda}\left(\frac{ch\lambda\ell - 1}{ch\lambda\ell} + \frac{2sh\lambda\ell\cdot sh\lambda L}{ch\lambda\ell}\right)\right] \qquad (13)$$

The obtained analytic solutions enabling in case of infiltration feed the prediction of techno-genic horizons and groundwater appearance in construction bases on the weak permeable layers (lenses).

3. Results

This article represents high importance for does areas where there is the problem of groundwater table and due to natural and artificial factors that table can fluctuate. Prediction method which has been revealed in this work is the key factor in terms of problem definition and elaboration of engineering protection measures against phenomenon of high level of groundwater. The common method of regulation of groundwater level is the application of drainage systems. Depending on particular situation and characteristic of area, the factors for solution can be different. Prediction results can be basis in development of the engineering protection measures against the negative effect of groundwater for upper and underground parts of structures. Using predefined equations (12) and (13) we can understand the maximum rise of groundwater, so based on this we can select protection measures especially drainage type, installation depth, diameter of drainage pipe and etc.

4. Conclusion

In the scope of following work the theoretical solutions for prediction of fluctuation of groundwater levels in aeration zone has been done, which can serve as well as basis for selection of engineering protection measures against the negative effect of high level of groundwater, as well as the application in practice for design and construction

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HEALTH MONITORING OF SEISMIC ISOLATED BUILDINGS

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Keywords: building, seismic isolator, monitoring.

Abstract

One of the easiest and common solution of seismic isolation of buildings is the using of seismic rubber steel isolators, which work as bearings for building. For construction of building in regions with high seismic hazard usually are used rubber lead bearings (LRB) or high dumping rubber bearings (HDRB). Armenia is country which is placed in region where the risk of earthquakes is too high. After Spitak Earthquake (1988) the HDRB type of seismic isolators started to produced and used for the buildings construction in Armenia. But the using of seismic isolators requires the providing of special maintenance of building, which must include the monitoring of isolators condition and building structure wholly.

The developed complex of measurements and investigations which are providing the health monitoring of isolators and building in all are presented in paper. Also there is shown that the type of measurements and received data analysis can be carried as markers for the evaluation of isolators and building condition. The description of equipment and measurement methods as well as examples of carried out investigations for some seismic isolated buildings in Yerevan are presented.

5. Introduction

The technologies of monitoring of buildings, bridges and other civil structures health in the process of developing worldwide [1,2]. The one of type of buildings which require the special attention are buildings with the seismic isolation.

Usually the seismic isolators are placed on the level of foundation, or on the level of second or third floors of the high-rise building (above the floors occupied by underground garages and other technical premises).

Rubber-steel seismic isolators have a layered structure, where the layers of rubber alternate with steel sheets. There are three main types of rubber-steel isolators: elastomeric whose damping coefficient is very low, isolators with the use of highly elastic rubber, they have damping factor 8-12% and isolators with a lead core, in which the damping coefficient reaches 25-30%.

The Republic of Armenia is located in a zone with high seismic activity. Large territory of the country is located in zones where the acceleration of soil is estimated at 0.4g. After the devastating Spitak earthquake in 1988, which took lives of 25000 people, Armenia began to pay much attention to increasing seismic resistance of buildings [3]. Seismic isolation of buildings through the installation of seismic isolators is one of the methods that have been applied in Armenia for this purpose. Armenia also has production of seismic isolators, which completely covers the needs of the country.

The Table 1 contains data about the buildings that were built in Armenia.

Table1 List of seismic isolated buildings with using of HDRB produced by "Retine Noruyt" LLC

Name of building	Dimensions of building in plan (m)	Stories	Place	Stage of construction
Multi- functional complex "Our Yard"	32x23	16	Yerevan, Armenia	completed
Multistory multifunctional complex "Cascade"	45x17	11	Yerevan, Armenia	completed
Business center "Elite Plaza"	42x36	20	Yerevan, Armenia	completed
Apartment building on Arami str.	33x32	11	Yerevan, Armenia	completed
Apartment building on Arami str.	52x33	13	Yerevan, Armenia	completed
Multistory apartment complex "Dzorap", consist from 2 separate building	99x33	15	Yerevan, Armenia	completed
Multistory apartment complex "Northern Ray", consist from 2 separate building	74×39	15	Yerevan, Armenia	completed
Commercial center-hotel on Hanrapetutyan Str.	45×37	7	Yerevan, Armenia	completed

Multistory apartment building on Baghramian Ave.	41×36	17	Yerevan, Armenia	completed
Hotel complex	56×26	6	Dilijan, Armenia	completed
Multistory multifunctional complex on Demerchyan Str.,	40x23	16	Yerevan, Armenia	under construction

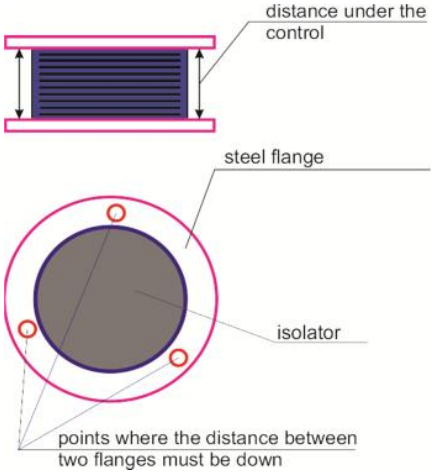

Fig.1 Scheme of geometric measurements during monitoring

6. Methodology

However, the usage of seismic isolators poses new questions to the operating services, especially, the assessment of the technical condition of the seismic isolators and the building itself in operation period. Thus, uneven deformations of the building caused by either deformations of the base grounds or by uneven temporary loading, can cause overstrain in seismic isolators, which in turn can cause loss of the seismic isolator or a group of seismic isolators. This in its own can lead to serious consequences. In addition, in high-rise buildings during the course of time a significant effect on the stress-strain state of seismic isolators have inclines as a result of the impact of wind loads, which creates greater dispersion of the places of accumulation of deformation damages.

In this regard, for high-rise buildings seismically isolated with the use of seismic isolators, there is a need to identify changes in the stress-strain state of structures and the localization of places of such changes.

One of the methods of assessing the current state can be a periodic visual inspection of seismic isolators for the formation of cracks in the rubber, the shifts of the isolator in the horizontal plane, the formation of unacceptable deformations, as well as instrumental measurements.

Periodic visual inspections of seismic isolator surfaces can reveal already developed defects and can not signal about defects that have just begun to develop.

Therefore, for a more substantial assessment of the technical state of the seismic isolator, instrumental measurements must be applied.

By periodic instrumental measurements it means the measurement of the height of a seismic isolator at characteristic points (Fig. 1). A comparative analysis of the measurements made with a certain periodicity can characterize the technical state of the particular seismic isolator. Measurements must be made with an accuracy of 0.01 mm, which is easily provided by modern numerical meters. The difference between the previously measured values and measured at this stage can serve as an indicator of the possible start of developing defects.

However, in an exploited multifunctional high-rise building, access to most of the load-carrying structures can be severely limited, so it may be difficult to monitor the condition of seismic isolators using traditional methods of visual and instrumental examination given above.

As for the evaluation of the stress-strain state of structure, in the last 10-15 years the dynamic methods of sensing of structures were used, based on the measurements of periods and the determination of the logarithmic decrements of the natural oscillations of buildings and structures.

For these purposes, digital portable accelerometers have recently become widely used, which have the ability to measure accelerations in three directions simultaneously.

Accelerometers can be installed both during construction and after putting the building into operation. In this case, accelerometers can be installed permanently (if the installation occurs during construction), and can be removable (to be installed only for measurements).In the second case, the installation of accelerometers should take place in permanent locations, which are determined at the design stage of the building. For these places, the theoretical acceleration spectra are calculated at the design stage, which are then compared with the results obtained during field measurements.

Accelerometers should be installed at the levels of the lower and upper flanges of the isolator, while depending on the type of attachment of the isolator to the concrete beam. Accelerometers can be installed both on the flanges themselves and on the concrete girders. The necessary fixation of the accelerometers is their immobility with respect to the flanges / beams. The scheme for installing the accelerometers is shown in Fig.2.

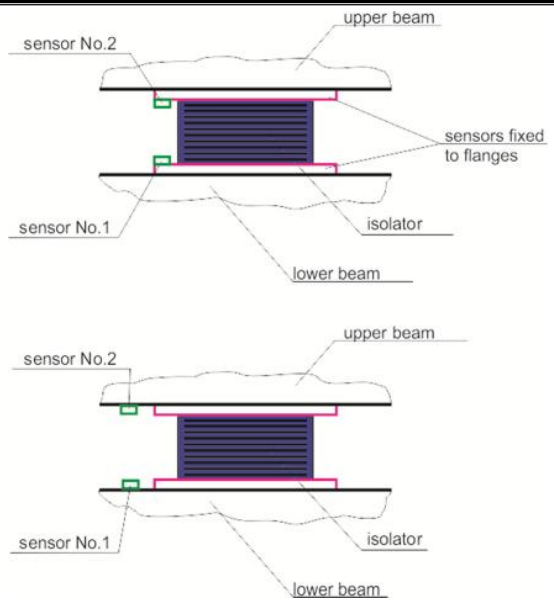

Fig.2 Scheme of installation of accelerometers relative to the isolator

As mentioned above, for the selected points additional accelerometers should be installed. The scheme for installing the accelerometers is shown in Fig. 3, while the number of accelerometers installed in the height of the building depends on the very height of the building and the capacities of the recording and processing equipment. Figure 3 shows a diagram with a minimum number of measurement points.

In parallel with measurements of the acceleration of the oscillations of the building, it is also necessary to periodically perform geodetic measurements, which can reveal undesirable distortions and other deformations. The location of the points of geodetic measurements is also shown in Fig. 3.

If the equipment for the geodesic measurement is known, the sensors for the measurement of accelerations and geometrical data of isolators are necessary to present more detail.

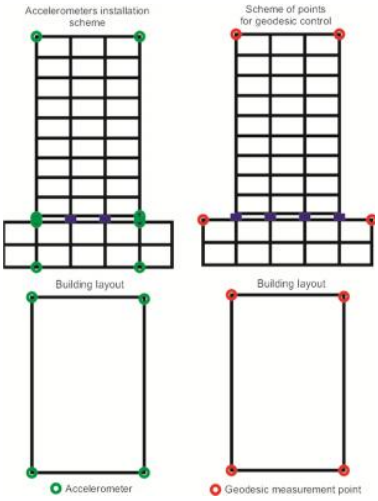

Fig.3 Scheme of location of measurement points during monitoring

The measurement of accelerations was done by the sensors Fig.4, which were elaborated in NUACA on chamber of "Road and Bridges" in cooperation with "Kamurjshin" CJSC and "Retine Noruyt" LLC. As the base was taken accelerometer MPU6050 [4], which use MEMS technologies and has possibilities to measure accelerations in three directions at the same time.

Fig.4 Accelerometer on the base of MPU6050

For the measurement of isolators height the special meter Fig.5 on the base of digital block of calipers. This device allow to measure the height with 0.01mm accuracy and has remote monitor which allow to provide measurements in the places, most difficult to access.

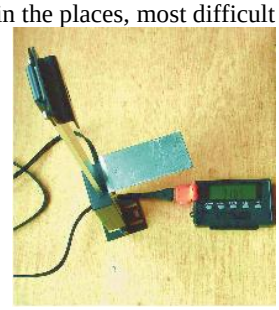

Fig.5 Meter for isolators height measurement

7. Results

The developed equipment was tested on the some seismic isolates building in Yerevan. The data collection process is started and at present time for the Dzorap Building and Business center "Elite Plaza" Buildings the monitoring process is started. The first measurements data of isolators height and oscillation characteristics are input in database. It is foreseen that these two years for all building will developed data base and measurements process will implement every 1-2 years. The review of collected data will allow create final methodology of seismic isolated buildings health monitoring. As can be seen from Fig. 6 (green is movement of bottom flange and cyan – upper flange of isolator), the difference in movement between the seismically insulated part of the building and its lower part is revealed, which is not seismically insulated.

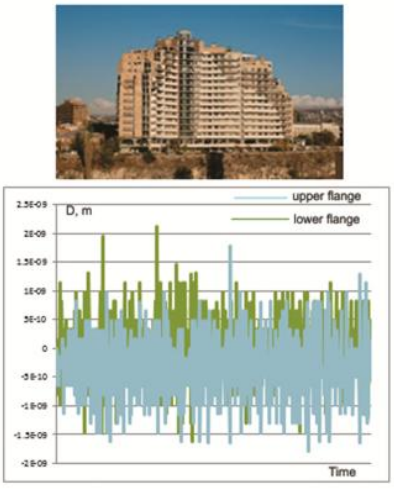

Fig.6 Dzorap Building in Yerevan with seismic isolators. An example of recalculated movements of the

upper and lower flanges of the isolator, respectively (the movements were recalculated from accelerations by the method of double numerical integration)

8. Conclusion

The study of changes during the time some of characteristics such as height of isolators, general deflections of building and dynamical characteristics on the different levels can be used as indicators of building health and can be easier monitored by the methodology proposed in this article.

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REDISTRIBUTION OF FORCES IN RUBBER BASE ISOLATIONS OF MULTISTOREY BUILDINGS DURING EARTHQUAKE ACTION

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Keywords: Rubber isolation, nonlinear analyses, seismic action, multistorey building

Abstract

Various types of rubber base seismic isolation systems for multistorey buildings are commonly used in the world. The isolation systems used in Armenia in contrast to foreign one are not taken tension forces since these systems don't have structural connections to superstructure and substructure of the buildings. This can be cause of additional stresses after redistribution of axial forces in rubber base isolations during seismic action.On the basis of the existing finite element analyses for reinforced concrete frame-wall buildings with application of rubber base seismic isolation systems carried out in the article. The effect of increasing stresses during seismic action is discussed and investigated by both*fast nonlinear time history analyses (FNTHA)*and *nonlinear time history analyses with direct integration (NTHADI))*.The obtained results showed importance of account of redistribution effect in the seismic isolation systems for multistorey buildings for their seismic stability and safety.

1. Introduction

Armenia is located in one the highest areas of seismic risk in this region and experienced a lot of huge earthquakes over the centuries. The latest huge M7.0 was Spitak earthquake in 1988.

After the Spitak earthquake, seismic isolation systems began to be used in Armenia. There are different types of seismic isolation devices used in the world. Seismic isolated layered rubber–steel bearing devices are the main type isolation systems are used in Armenia (fig. 1).

Fig. 1Layeredrubber bearing seismic isolators [1]

Elastomeric isolators are made up of alternating layers of steel laminates and hot-vulcanized rubber, due to the type of the rubber compound.At this moment there are about fifty buildings with seismic isolators in the Republic of Armenia (fig. 2).

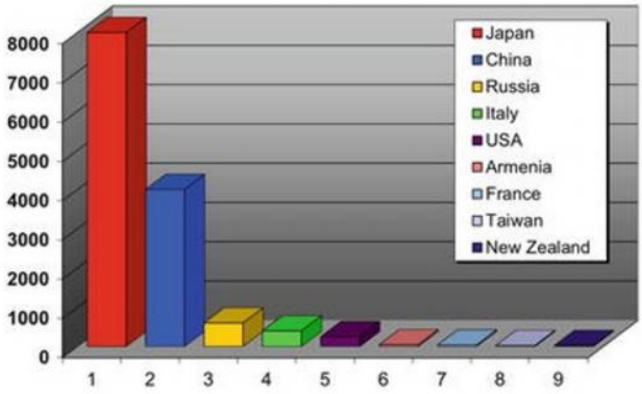


Fig.2 Numbers of seismically isolated buildings in the most active countries, (September 2013) [2]

According Earthquake resistance building codes of Armenia [3] the seismic isolation is used for buildings and structures with natural period of oscillation no more than 3.0 sec, at the same time the natural period of the same system without seismic isolation (with regular foundation) have to be within 0.1 – 1.0 sec. Elastomeric isolators are very stiff in the vertical direction and can support the dead load with negligible creep effects during the whole life of the structure. Seismic isolators are flexible in the horizontal direction, and have high vertical stiffness and at the same time the damping is about 10 ... 15%. The shear modulus G ranges from 0.4 to 1.4 MPa.

By the structural concepts, according to [3], two types of seismic isolation systems are used. First type is the systems located below the level of the pavement around the building and second type is systems located above the level of the pavement, but not higher that second story level of building. The choice of the seismic isolation type is governed by the subsoil conditions and functional purpose of the building.

The analyses of building and structures with seismic isolation systems is carried out by two methods: according to design response spectra by [1], or by earthquake response spectra compiled on the bases of

recorded accelerograms or synthetic ones generated for the given building site. And the design forces should be assumed as per the least favourable of the two mentioned analyses.

At first, for design of buildings and structures with seismic isolation it is necessary to know the force-displacement relationship (fig. 3) and various parameters of seismic isolators (SILRSB) using in Armenia.

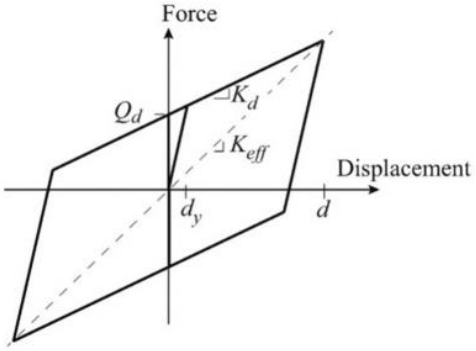


Fig. 3 Force-displacement relationship of seismic isolators

According to [4] the main geometrical and physical-mechanical parameters of SILRSB and their values are the following: number of rubber layers – 14, number of internal steel plates – 13, overall diameter (D = 380 ± 2.0 mm), internal diameter (d_1 = 19 mm), overall height (H= 202.5 ± 2.5 mm), thickness of rubber layers (S = 9.0 ± 0.1 mm), radius of internal steel plates (d_2 = 360 ± 0.5 mm), thickness of steel plates (S_1 = 2.5 mm),radius of steel end-plates (d_3 = 376 ± 0.5 mm), thickness of steel end-plates (S_2 = 20.0 ± 0.2 mm), top and bottom cover layers (S_3 = 2.0 ± 0.1 mm), overall weight (W = 77.5 ± 2.5 kg), shear modulus (G = 0.97 N/mm^2), vertical stiffness ($K_{eff,v}$ = 300 N/mm),effective iffness (K_{eff} = 0.95 kN/mm), post yield stiffness (K_d = 0.81 kN/mm), elastic stiffness (K_e = 2.95 kN/mm), maximal displacement (d = 280 mm), maximal allowable vertical force (Fv = 1500 kN), yield force (Fy = 56 kN), equivalent viscous dumping (β_{eff} = 10 %).

2. Methodology

16-storey reinforced-concrete frame-shear wall building (fig. 4) using SILRSB seismic isolation systems calculated under seismic action is considered in the article.Distribution of columns and shear walls are symmetrically according main axis (longitudinal X, and transverse Y).A quantity of seismic isolators (255 isolators) was determined from the conditions that maximal horizontal displacement at the top of isolator will be less than 280 mm and maximal vertical design load will be less than 1500 kN. Different options of work for seismic isolators are considered in the article. First problem with SILRSB isolators supporting in the vertical direction (Z), both compressive and tensile forces are assumed, and the second problem, carried out by isolators in the vertical direction (Z) take only compressive forces.

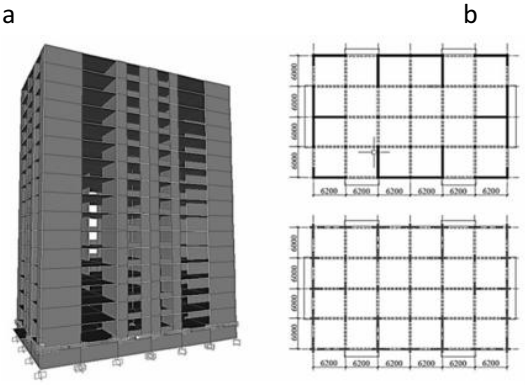


Fig. 43D view of the model of a building (a), a plans of bearing system /columns and shear walls/ and position of rubber bearing system (b)

The storeyheight for building was unchangeable and equal to 3.0 m, respectively the thickness of the slabs and the cross section of square columns are equal to 16 cm and 50×50 cm.The thickness of shear wall of superstructure areequal to 16 cm (minimum value according to the building code [3]). Sections of the girders are different 550x550 and 500x560 mm. All loads are assumed by the building codes. The class of concrete B25 is used for analysis: cube strength of concrete is 25 MPa, modulus of elasticity E_b is equal to 30000 MPa.All calculations have been carried out with implementation both fast nonlinear time history analyses and nonlinear time history analyses with direct integration. At the same time there were used accelerograms of Spitak Earthquake were recorded in Gukasyan. Maximum values of acceleration were increased to 0.4g in horizontal direction (for accelerogram in direction Y, fig. 5) and to 0.28g in vertical direction (for accelerogram in direction Z, fig. 6).A three dimensional model using frame and shell lements represents the building in the analyses.

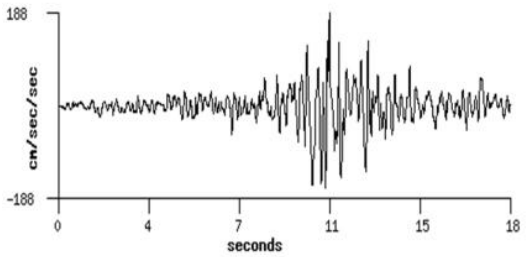


Fig.5 Gukasyan accelerogram of Spitak Earthquake (horizontal component, direction Y)

The natural period of vibration of the building in Y direction is equal 2.09 sec.

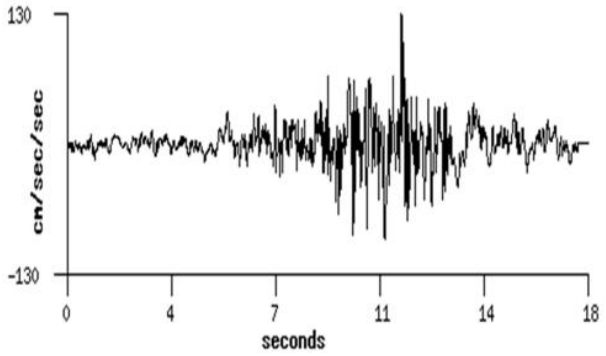


Fig.6Gukasyan accelerogram of Spitak Earthquake (horizontal component, direction Z)

For estimation of the change of the forces in SILRSBduring seismic action, isolators located in the middle parts in the transverse direction (Y)of the building are considered: from the left side, third (fig. 7) and fourth (fig. 8) frames.

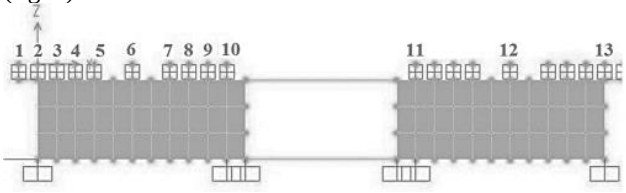


Fig. 7 FE models of substructures of buildings with seismic isolators (third frame from the right)

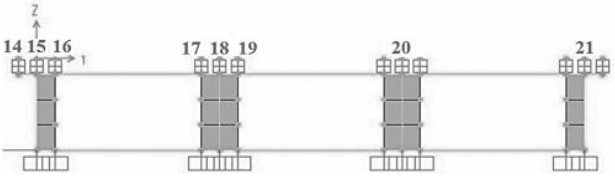


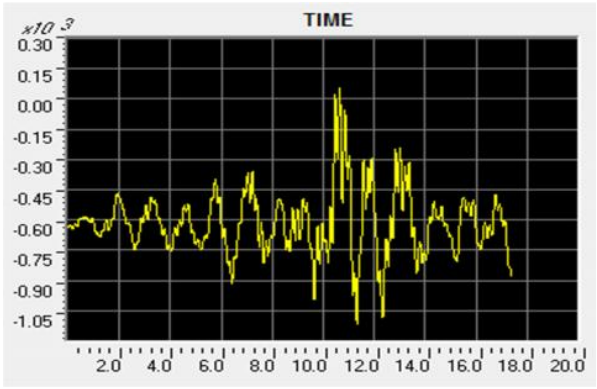
Fig. 8 FE models of substructures of buildings with seismic isolators (forth frame from the right)

The main peculiarity of the workis the investigation of "tensile forces" in seismic isolators of multistorey buildings at seismic action. Since seismic isolators used in Armenia do not work on tension, therefore, a comparative analysis of two different models wascarried out. In the first model, seismic isolators are considered as ordinary finite elements of seismic isolation that are accepted in SAP2000.In the second model, seismic isolation system was described by nonlinear elements working only on compression. For comparison analyses, seismic isolators with numbers 2 to 21 are considered in the article (fig. 7 and 8).Calculations were performed by SAP2000 software, where effect of increasing stresses during seismic action investigated with implementation best known time history nonlinear analysis methods:*FNTHA* and *NTHADI.*At the same time two possible combination were considered in the article. One of them is the static (dead load, short term and long term live loads)and earthquake load in horizontal direction (Y) and second combination is the same as first one in addition with earthquake load in the vertical direction (Z).

3. Results

Studies show that for low-rise buildings with seismic isolation, redistribution of forces is not observed, since there are always only compressive forces in all elements of seismic isolation. Obtained results by FEM are shown, that redistribution of forces in the seismic isolation depend on the different factors, where main of them height of building. At the same time the forces increase depends on also the length (in direction of earthquake forces) of building. The results of calculation using *FNTHA* and *NTHADI*methods are presented in the fig. 9 and table 1. Where the differences of the results by two nonlinear methods are shown in a column (9). As can be seen, the differencesof the values of the compressive forces by the two methods are from 11.6 to 22.6%. In the case of direct integration, the maximum compressive forces in all isolators do not exceed 1500 kN and at the same time during maximum accelerations (fig. 9, b)SILRSB does not work temporary, consequently, in some isolators will be increase of forces. a

a



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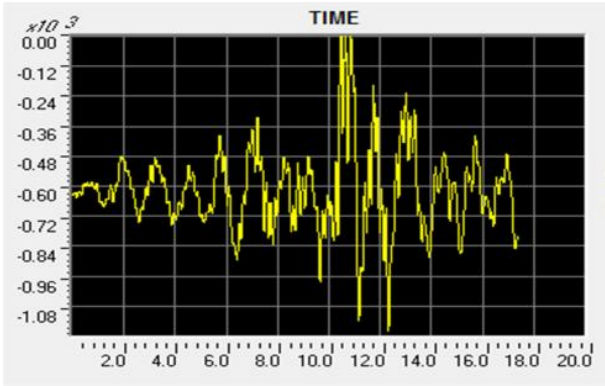

Fig. 9 AxialForcesofseismicisolators(kN) relationshipoftime(sec) by FNTHAanaleses (Gukax Y+ Gukax Z)
seismic isolator N 15: both on tension and compression(a), only on compression(b)

Table 1 Axial Forces values in the seismic rubber isolators by Gukasyan accelerograms depend on types of nonlinear analyses, finite element of seismic isolation and directions of earthquake loads

Number of seismic isolation by fig. 3	FNTHA (tension and compression) Gukax Y	FNTHA (only compression) Gukax Y	Differences (2) and (3)	FNTHA (tension and compression) Gukax Y+ Gukax Z	FNTHA (only compression) Gukax Y+ Gukax Z	Differences (5) and (6)	NTHADI (only compression) Gukax Y+ Gukax Z	Differences (6) and (8)
	[kN]	[kN]	[%]	[kN]	[kN]	[%]	[kN]	[%]
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
2	1084	1135	4.5	1180	1280	7.8	1131	11.6
6	769.9	776.2	0.8	946.6	1009	6.2	866.4	14.1
10	1247	1260	1.0	1486	1697	12.4	1313	22.6
11	1090	1126	3.2	1210	1325	8.7	1169	11.8
12	769.9	784.1	1.8	980.3	998.8	1.9	857.3	14.2
13	1307	1341	2.5	1518	1545	1.8	1362	11.9
15	1038	1092	4.9	1106	1198	7.7	1038	13.4
18	1185	1205	1.7	1717	1894	9.3	1473	22.2
20	1134	1164	2.6	1638	1740	5.9	1390	20.1
21	1167	1224	4.7	1388	1499	7.4	1254	16.3

4. Conclusion

It should be notedthat increase of axial forces after their redistribution during earthquake action in most SILRSB do not exceed8%, but for some isolators this difference varies within the range of 11 … 14.5%. Consequently, an average increase of the compressive axial forces in the seismic isolation bearing systems of this building as a result of redistribution can be taken about 10%.

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THE PECULIARITIES OF BUILDING CULTURE OF ARMENIAN HISTORICAL AND ARCHITECTURAL HERITAGE

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Keywords: Architectural heritage, numerical analyses, FEM modeling, strengthening.

Abstract

This paper gives a general overview of analysis of bearing structures and elements of the main types of Armenian architectural heritage based on examples of the most famous remained or half-ruined monuments constructed from the early to late Middle Ages. The probable reasons of their damages are described too. Furthermore, structural peculiarities of bearing systems in seismic regions are considered and investigated from the structural point of view. Analysis of the particularities of stone bearing walls of monuments constructed in different centuries has been done.

Results have received by research will help to get out rational and useful solutions for architectural and historical monuments during their conservation and strengthening works.

1. Introduction

The Armenian Highland at all times known for its rich architectural and construction heritage. The peculiarities of building culture of Armenian historical architectural heritage since from early fourth century and until twentieth century are connecting to the history of the development of a single type of building: the church.Because of, remarkable period in the history of Armenian art and architecture is undoubtedly bond up with the adoption and spread of Christianity on Armenian soil.

Thousands of Armenian churches were built during the long history of Christianity. Some churches were intended to stand alone, while others were parts of monasteries.

The main stages of development of Armenian historical and architectural heritage are [1]:

- early medieval period (4-7th centuries),
- developed medieval period (9-11th, 12-14th centuries),
- late medieval period (17-19th centuries),
- new period (19-20th centuries).

All these periods are distinguished by their unique composition, new artistic styles and have original building culture. Each subsequent phase is the improved version of its predecessor, so the Armenian historical and architectural heritage have preserved its national identity at all times. Over the centuries the structure and composition of the churches have changed, but its building materials was always the same, with its utmost rational usage. Various kind of tuff and basalt were the basic building material for historical monuments in Armenia.

2. Building typology

The early medieval Armenian architecture was of great importance for its further development, as at that stage the foundations of its composition-artistic and constructive principles were created.

Armenia's early churches, which were built in 4-5th centuries, had a basilican composition were either sighle-naved, or three-aisled basilicas. Some students of the history of Armenian architecture believe that Armenia borrowed the basilican composition for its churches from neighboring Syria. There are, however, characteristic differences. Armenian basilicas are built in stone and almost without exception have stone vaults over aisles and naves, where as in Syria, though walls and apses are of stone, roofs are generally unvaulted and wooden like Byzantine and Roman counterparts. A single roof covers both central and side aisles in most Armenian basilicas, while in Syria and the West the central nave usually has a separate and higher roof. At the same time, Armenian architects of the early Christian period did not stick only to basilica types, also have created new forms of church buildings.Despite the large diversity in the types of early churches, Armenian architecture achieved a distinctive style through the combination of a number of common characteristics and materials. By the late sixth or early seventh century, a unique national style of church architecture came in to being. Since 6-th century in Armenia, begin to build different types of churches with dome structures. At the same time, there is an opinion that the monumental shape of the dome has appeared in Armenian architecture from "hazarashen" roof type, which has been used in national building culture. Some scholars have called this phenomena the first national style in Christian architecture. It had been achieved long before the Byzantine, Romanesque, and Gothic styles were concretely formed.

The second period begins almost simultaneously with establishment of the Bagratid kingdom in the 880s. In the same time during the second period have begun to innovate and experiment with structures for more height and space.The compositions of the buildings, as a stylistic feature are characterized by the verticality.

It is noteworthy that from the 9th century, the churches were being built were reproduced by the plan and composition of the churches of the 6-7th centuries. At the same time, the most common type of church continued to be a "domed hall" type. The Armenian architecture of the 12-14th centuries is marked by the development of engineering ideas that made it possible to improve the structures of roofs in the designs of the churches.

Thus, in the second stage of medieval architecture development, by the skilful implementation of stone becomes possible to improve the bearing systems of buildings, and to erect arches, shapes, and domes with bigger spans.

The late medieval Armenian architecture is distinguished by the fact that the restoration of the ruined churches is being done, some of them was expanded and the most of monastery complexes was fenced. At the same time, numerous new monastery complexes were built. In the late Middle Ages, the three-aisled basilica (vaulted hall) churches have become more and more popular. However, domed churches were built too, especially domed halls and basilicas. Thus, late medieval period of church architecture in Armenia preserves the hereditary connection with the early medieval history, though it has its own style.

The architecture of new period came to a stop in Armenian proper, but Armenian architecture continued in diasporan cities like Constantinople, Tiflis, and more remote areas. In the years 1915 and after Armenian culture stopped totally in the ancient homeland. In that time, large numbers of ancient medieval monuments were destroyed. Though it should be noted that in modern times churches continue to be built and now are being erected in large numbers.

Modern architecture of Armenian churches is extremely dependent on the ancient tradition. Most new buildings consciously imitate the most famous monuments of the fourth to the seventh centuries, using contemporary constructional advances like reinforced and poured concrete for the traditional Armenian methods.

To summarize, it should be added that the main question of all time was how to build churches volumes in stone that would both resist the immense weight of the masonry vaulting and roofing and not crumble under the jarring effects of earthquakes.

3. Building materials and technologies

All churches are built entirely in stone. The main materials for construction was tuff and basalt. The scarcity of wood prevented its architectural use in medieval Armenia. Since wood was not available for making simple flat roofs, stones were employed, but their weight demanded they bear ranged in arcs or vaults.

The walls all of type churches were built from parallel, regular row of stone blocks with space between them filled with calcareous rubble. This masonry system calls "midis type" has stood the test of time and is still in Armenia in our days.

In the earlier churches for walls were characterized by the use of heavy and thick stone, often with mortar placed between joints. The inner core was so narrow that the walls themselves performed the real work of supporting the superstructure (fig. 1).Gradually in the fifth and sixth centuries, as the engineers saw that the dome sand vaults of earlier buildings were steadfast and resistant to shock, the blocks of stone became thinner and the inner core of mortar wider. Subsequently, more rubble was used, so in the 7th century it made up a third of wall thickness, while stone blocks came to play the role of formwork for rubble (fig. 2). Rubble was now the basic load-bearing element of the wall. This wall construction is more practical, and allow small blocks to be used (wall stone blocks in the oldest structures are quite large - up to two or three meters long and up to one meter high). The overwhelming majority of Armenian monumental structures in the 4th to 7th centuries was built from finely cut blocks. Some churches, however, were also built from roughly hewn stone blocks and cobblestone.

Fig. 1Fragment of stonework of monastery complex of st. Astvatsatsin (4-7thcenturies)

Fig. 2 Fragment of stonework of monastery ofAghjots vank (9-13th centuries)

In bearing systems of churches masonry pillars and pylons were used as free-standing supports. Solid columns were rarely used because of their frailty to earthquakes. Arches, semicircular or horseshoe-shaped were typically used in place of architrave roof in medieval Armenian architecture.

4. Methodology

It is possible to present the general development of Armenian historical and architectural heritage building culture based on various sources, but the specificity of this work is in the fact that it is an attempt to make research from the structural point of view, besides; the substantiations are presented through numerical analyses using computer software widely accepted throughout the world.

Additionally, the main purpose of the work is to give guidance to justify the optimal solutions chosen for the reconstruction of monuments. In order to find optimal solutions for each monument, it is important to understand the peculiarities of the building culture of the construction period. In addition, a detailed review of its technical condition should be carried out and it is important to determine the physical and mechanical properties of the bearing soil. After that, based on the experience, it is possible to make a preliminary selection of reconstruction solutions that will later be substantiated by calculations. It should be added that the final solution chosen for each monument is not always the common. They are derived from the initial data of an each case.

Taking into account the fact that in Armenian architecture all of time in the monuments, was using "midis" type of stonework, below have presented the experimental results, which were done earlier. Experiments were made on samples of size 100x50x150 cm. All samples were subjected to compress until cracks occur in them without adding the load (the third phase of the stress-strain-state). After that, the samples were strengthened in different ways and re-tested.

The strengthening of samples were made by injection of polymeric mortar, through the injection of cement mortar and through the double-sided reinforced concrete layers.As a result of the experiments it was found out that the restoration of their initial bearing capacity was restored accordingly by 0.9, 1.13 and 1.6 [2].

The results obtained through experiments can play a crucial role in making fundamental decisions in strengthening projects.

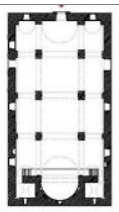
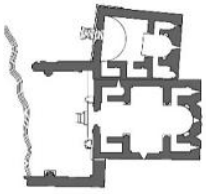
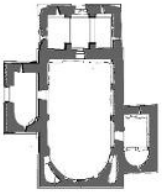
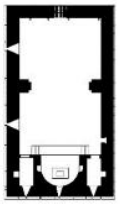
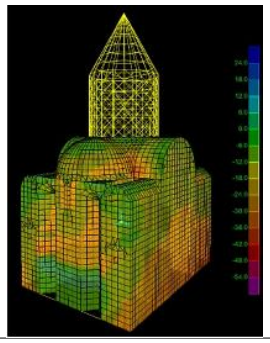
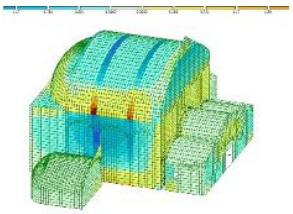
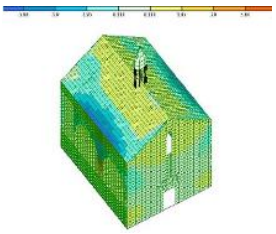
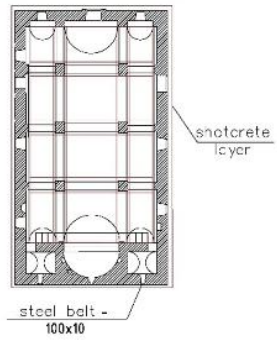
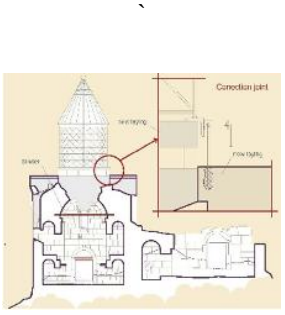
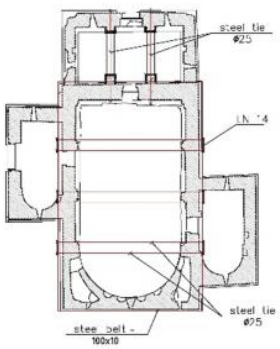
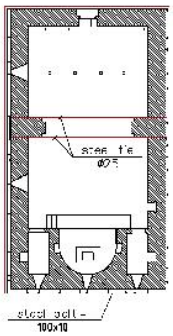
In the article, four monuments from different period of construction were observed and their structural investigations were presented below:

- st. Hripsime church in Horom village (since 4th century),
- Aghjots vank in Khosrov reserve (9-13th centuries) [3-8],
- Tejarujg monastery complex in Meghradzor village (12-13th centuries) [3-8],
- st. Gevorg church in Shagat village (19th century),

In table 1 were presented main characteristics of the churches.

In monuments from finite element method the basalt or tuff masonry was modeled in lime mortar with selected characteristics defined through analysis. The lime mortar was designed with special additives in order to be able to satisfy the bearing and deformability characteristics of structural systems.

Table 1 Structural investigation of the observed monuments.

Name of building	**st. Hripsime church** (since 4th century)	**Aghjots vank** (9-13th centuries),	**Tejarujg monastery** (12-13th centuries)	**st. Gevorg church** (19th century)
General view				
General plan				
Typology of church	early medieval period	developed medieval period	developed medieval period	late medieval period
	three-aisled basilica	domed hall	single nave church	single nave church
	from tuff stone	from bazalt stone	from bazalt stone	from bazalt stone
Structural analyse	In priminary stage has not nesessaty to do the FEM analys. Visual envestigation have showed that the stonework of the monument has not enough stiffness to resist horisontal forces.	**FEM model by SAP** 	**FEM model by LIRA** 	**FEM model by LIRA** 
Principal solutions	- Injection of masonry walls with lime mortar, - Shotcrete reinforesed layer, - Steel belt.	- Injection of masonry walls with lime mortar, - Horizontal steel ties, - New roof.	- Injection of masonry walls with lime mortar, - Horizontal steel ties, - Steel belt.	- Injection of masonry walls with lime mortar, - Horizontal steel ties, - Steel belt.
General scheme after strengthening				

5. Results

The analysis showed that the high-level of medieval architecture is connected with the correct representations about work of structural bearing systems, as well as by a level of development of building culture. Over the centuries, new solutions are becoming the keys for development of further stages of Armenian building culture.

These studies can help to make rational decisions in the field of restoration of architectural monuments.

Based on the performed sample analyses, we can identified two major type of strengthening solutions consisting of different types structures. Therefore, for the same typology of architectural monuments which have the similar construction techniques not always can get out the same principal solutions for retrofitting projects.

6. Conclusion

Nowadays the strengthening, preservation and reuse of cultural heritage around the world is considered to be one of the most important issues which solutions, we suppose, first and foremost, you need to look for architectural and engineering sciences. Because of the preservation of architectural monuments doesn`t only mean the preservation and wise use, but also restoration and strengthening.

The structural analysis in the project of preservation and reuse for architectural monument is solving the major problem for engineers: to get a reliable assessment of the safety margins of existing monuments. For an existing structure, the working life already spent and desirable remaining working life may influence technical decisions. During the work on the basis of calculated studies it is expected to evaluate the decreasing of seismic hazard after the measures proposed for the given structure.

The assessment of the effectiveness of interventions in historical monuments is a domain where decisions should be taken on the basis of risk analyses in order to reach a compromise between the monumental value of the building, the cost of investigations and the probability and foreseeable consequences of a possible structural deterioration that can cause its destruction.

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THE USE OF MODERN ENGINEERING DECISIONS IN BUILDINGS FOR DISASTER RISK REDUCTION

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Keywords: Disaster Risk Reduction, Stability, Pre-stessed Truss, Extreme Bridges, Critical Stress.

Abstract. Modern buildings, and economic and socio-cultural systems are exposed to various disaster risks, such as flooding, heavy snow, earthquakes, landslides, hurricanes and etc. The prevention and reduction of disaster risks, as well as some modern engineering approaches and methods, are considered in the article.

To achieve the goal, the following methods are used, such as the methods of prestressing, the method of the application of working of material in the elasto-plastic area, the method of the transformable application. All these measures will reduce the risk of disasters in buildings and their real seismic resource will be assessed by installing geodesic gyroscopes on the buildings, and the disaster risk can be predicted using the method of studying weak fluctuations.

The structures obtained in this way differ from existing ones and have higher load-carrying ability, large spans, relatively low weight and the ability of geometric transformation. Risks related to disasters can be prevented and reduced through buildings designed using such original approaches.

1.Introduction

Modern people, society, buildings, economic and socio-cultural systems are exposed to various disaster risks. Reducing disaster risk is one of the priorities of the UN activities. Despite UN efforts, over the past decade, as a result of disasters, the number of deaths in the world has exceeded 700,000. About 1,4 million people were injured, about 2,3 million people were left homeless, and the total economic loss amounted to 1,3 trillion dollars. In addition, in 2008-2012, 144 million people were displaced as the result of disasters [1].

Two numbers - 144 million and 2,3 million are noteworthy. Buildings are implied behind these numbers. Some of them were destroyed as a result of earthquakes, storms and other impacts. Some were built in a place where they should not be built, some were built poorly, some were built well, but the building codes were substandard. In all these cases, displaced victims complained to builders. The Sendai Framework for Disaster Risk Reduction 2015–2030 was adopted at the Third United Nations World Conference on Disaster Risk Reduction, held in 2015 in Sendai, Japan. It states that the major leverage for successful implementation of the program is the wide application of scientific and technological achievements. In addition, the priority activities set by this program are necessary to reduce these risks for enhancing disaster preparedness for effective response and to "Build Back Better" in recovery, rehabilitation, and reconstruction, which means the construction with better quality [2]. The aim of the work is to develop engineering approaches in the process of the construction and designing to reduce, prevent or predict the aforementioned disaster risk.

2.Ways to solve problems and results

2.1 Ways to reduce the disaster risks caused by flooding and heavy snowfall

Previously conducted analysis showed that bridge construction technology should be replaced in Georgia. The bridges built on fast-flowing Mountain and plain rivers have middle supports (**Fig.1**). This factor creates especially serious problems in the fast-flowing mountain rivers. This leads to potential

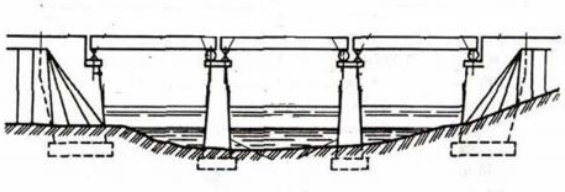


Fig. 1 Bridges with middle supports

landslides, mudflows, and flooding in the valleys of rivers.

The Authors conclusion was that suspension bridges (**Fig.2**) or pre-stressed big span bridges (**Fig.3**) should be built above mountain and plains rivers These bridges do not require middle supports and they will retain an exploitation in a case of washout of riverbeds [3]. The disasters caused by landslides, mudflows, and flooding will be reduced or the risk will be reduced to zero in this way.

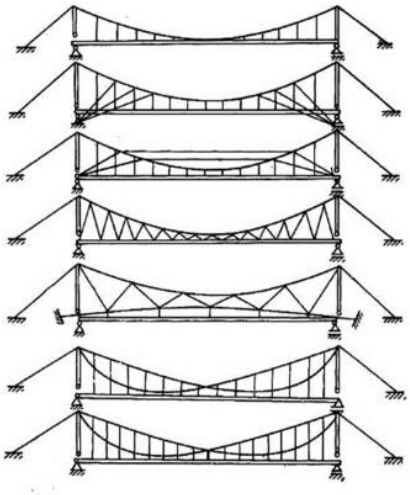


Fig. 2 Suspension bridge systems

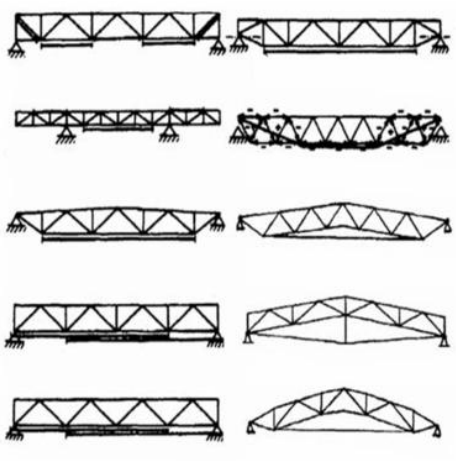


Fig. 3 Pre-stressed systems

load carrying capacity, reduce the prime cost and etc. [4]. The truss is calculated by the stiffness matrix method and is optimized using the metaheuristic - Gradient and Simulated Annealing methods (**Fig. 5, Fig. 6**). Figure 5 shows a diagram of the relationship between the truss span and its mass using the force method of calculation and the optimization method for

simulated annealing. Figure 6 shows a diagram of the relationship between the truss span and its mass using the simulated annealing method in the elasto-plastic area of working of material [5].

These type pre-stressed trusses may be used to avoid possible threats due to heavy snowfall. For example, in big span buildings where conventional roofing systems [3] [6] are used for heavy snowfalls, the risk of disasters caused by their expected destruction can be avoided by using a pre-stressed effect.

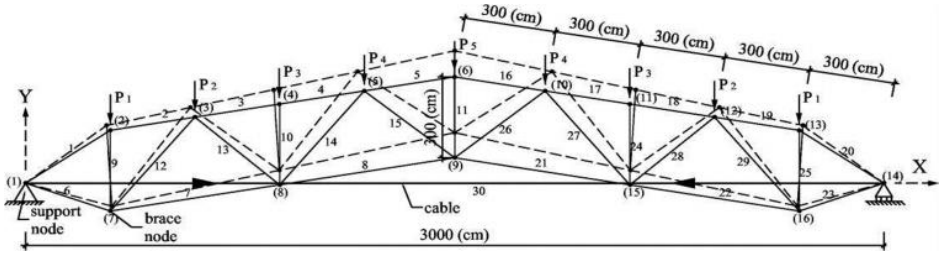
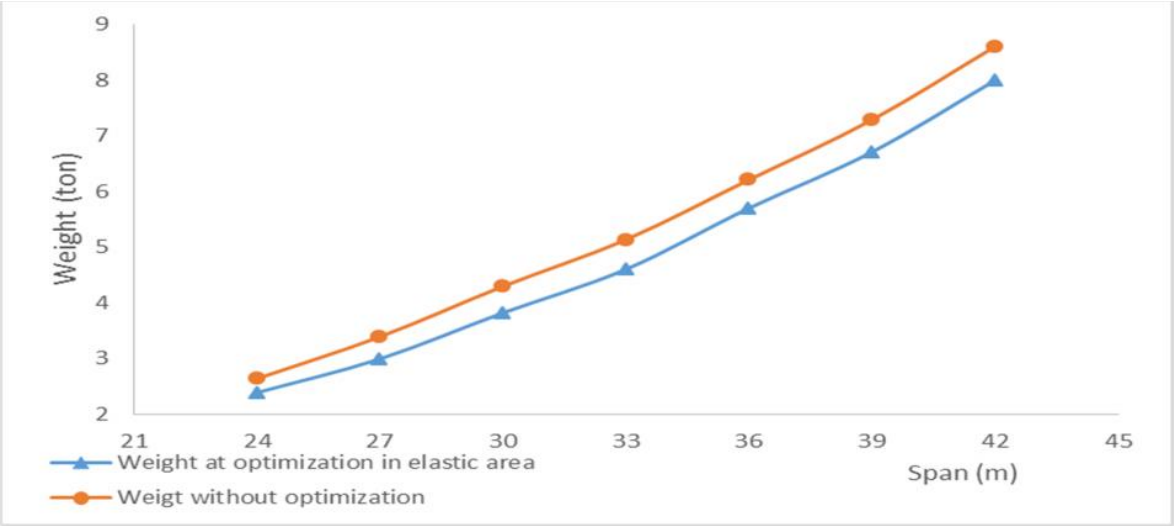


Fig. 4 New type "Arch with a Cable" pre-stressed truss



It is important to build and use extreme bridges under conditions of floods, mudflows, and landslides (which operate in the riverbed) [7]. In this direction, the authors have developed the constructions of multi-span inventory bridges, the experimental version of which was used on the River Mtkvari (**Fig.7**). Special importance is given to the accelerated mechanized bridge construction that will be placed on the truck. Its folded packet is 12 meters and 24 meters in the open state (**Fig. 8**) [8] [9].

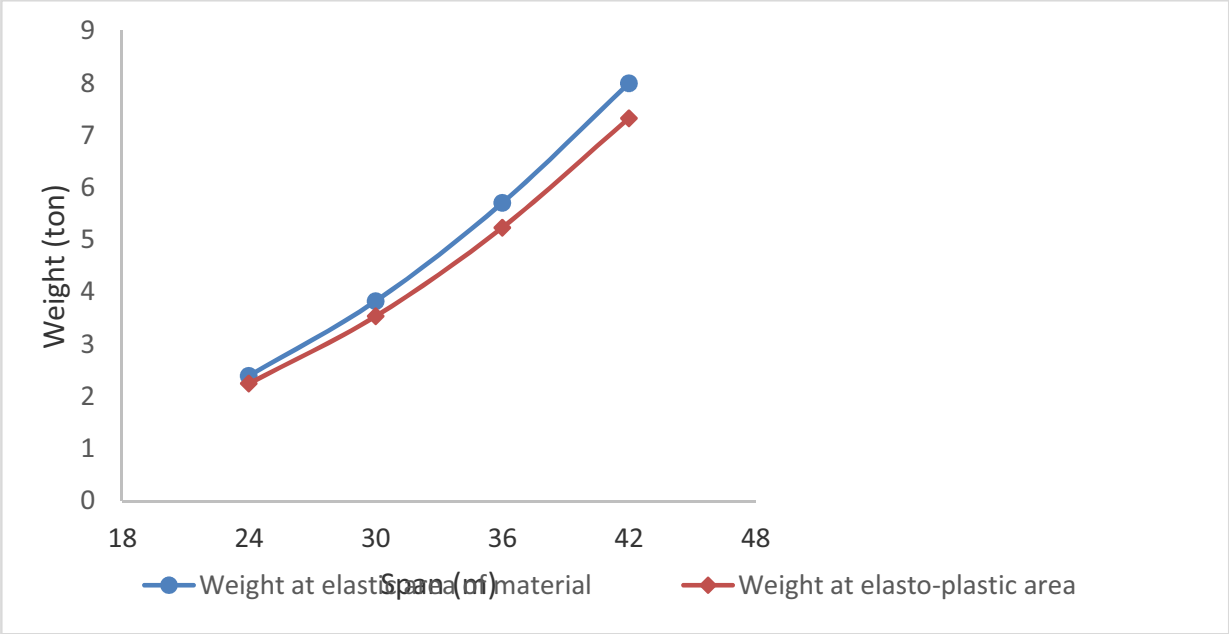


Fig. 6. Optimization of "Arch with a Cable" - new type pre-stressed trusses for different spans in the elasto-plastic area of working of material

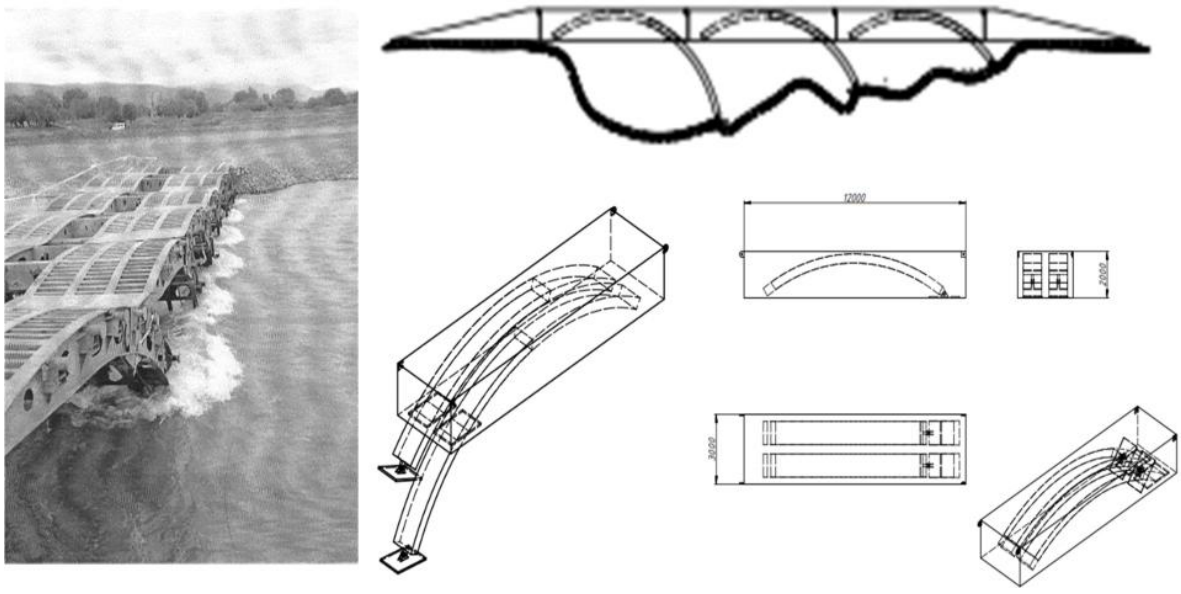
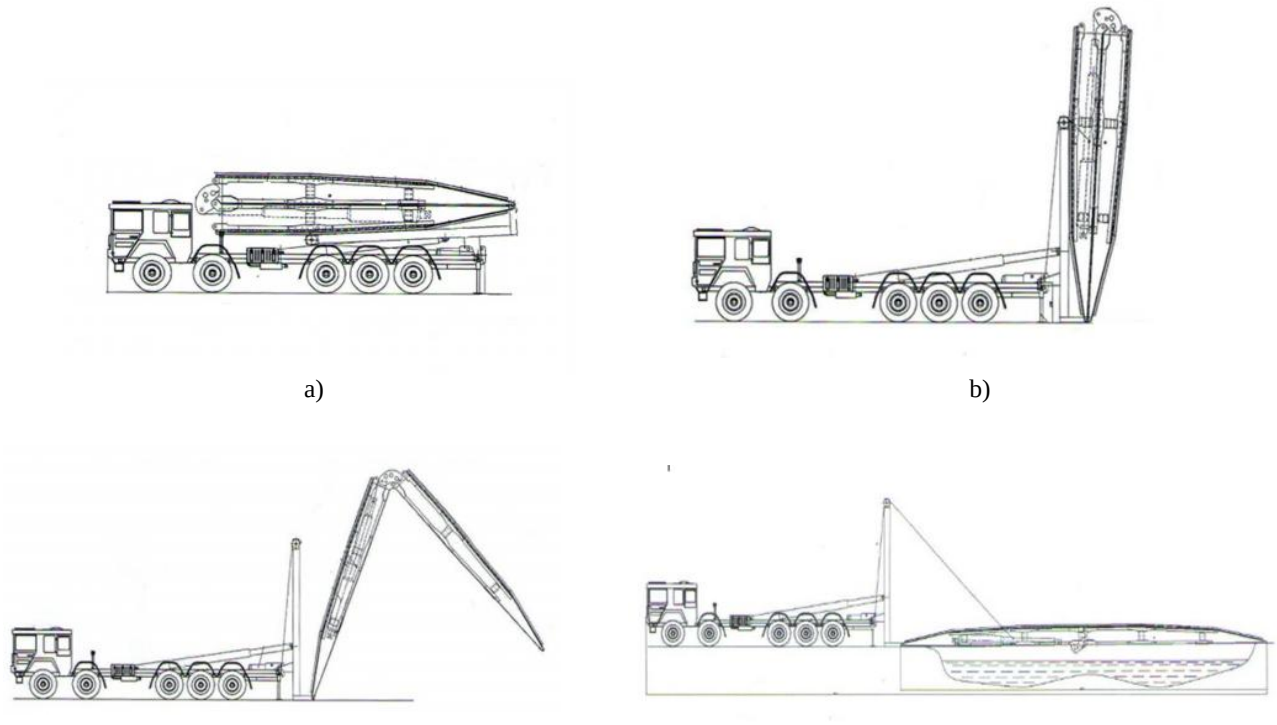


Fig. 7 Multi-span inventory bridges



a) b)

c) d)

Fig. 8. Bridge opening scheme
a) the bridge in folded state; b) the bridge in vertical condition;
c) lowering the bridge on to the supports; d) the view of the opened bridge

2.2 Reducing the risk of disasters caused by earthquakes.

The Caucasus region is located in the zone of crossing the continental plates, which is responsible for the seismic activity of Georgia. The risk of disasters caused by an earthquake is one of the most unexpected and dangerous in comparison with other risks. To avoid this risk, it is important to use lightweight structures for building construction. Figure 9 shows the possible movement of the building during the horizontal acceleration of the ground caused by an earthquake. Figure 10 shows a comparative analysis of forces occurring in light and heavy structures during an earthquake [10]. The forces raised in light steel structures by an earthquake are much less than the forces raised in reinforced concrete construction. Consequently, the construction technology of multi-story buildings should be replaced in consideration of advantages given by light structure.

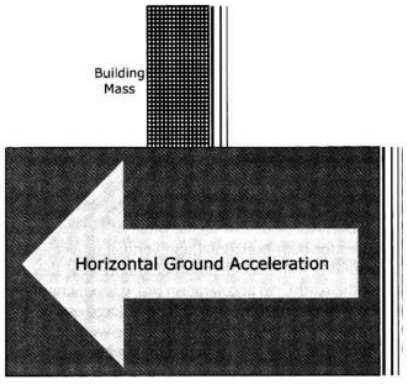


Fig. 9. Acceleration of the ground

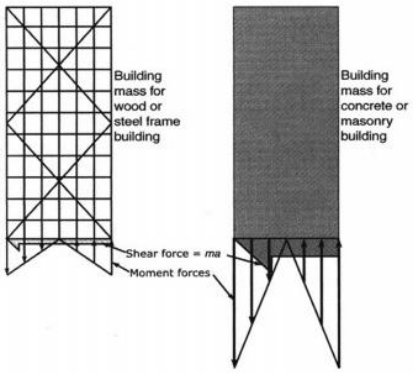


Fig. 10. Inertial forces associated with ground acceleration

Conducted studies have shown that compressed columns and rods are one of the main reason for disasters caused by the loss of load loading capacity in buildings [11]. It is well known by the scientific society that the consideration of work in the elasto-plastic area of material determines the modern level of the theory of seismic stability of buildings, since it is additional supplies for constructions in terms of load carrying capacity. The authors have developed a number of original approaches for determining the values of critical stresses and longitudinal bending coefficients defined by the experimental studies through the approximation of the double elastic modules and tangents (**Fig.11**). Consequently, formulae have been obtained for calculating critical stresses on the stability

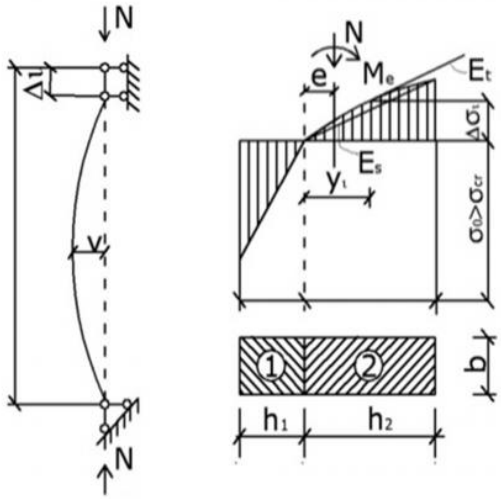


Fig. 11. The elasto-plastic working of compressed element

in the elasto-plastic area of working of material [12]:

$$\sigma_{cr}=\frac{\pi^2 E}{\lambda^2}\left(1-\frac{I_2}{I}\right)+\left(\sigma_{el}-\frac{\lambda^2\left(\sigma_{fl}-\sigma_{el}\right)}{2\pi^2 E}\right)\frac{I_2}{I}$$
$$+\frac{I_2}{I}\sqrt{\left(\sigma_{el}-\frac{\lambda^2\left(\sigma_{fl}-\sigma_{el}\right)}{2\pi^2 E}\right)^2-\sigma_{fl}\left(\sigma_{el}-\sigma_{fl}\right)}$$

(1)

Where, σ_{cr} – is critical stress; E – is modulus of elasticity; $\lambda=l_0/i_{min}$ – is slenderness of rod, which equals the design length of the rod divided by the radius of gyration; $l_0=\mu l$ – is design length of the rod; μ - is the rod effective length factor, which depends on the end conditions (boundary conditions) of the rod; σ_{el} -is proportional limit; σ_{fl} - is yielding limit; I and I_2 – are moments of inertia of the whole and the second part of the cross section in the direction of the neutral axis and their ratio shows the position of the neutral axis in the cross section.

The coefficient of longitudinal bending - φ was obtained from the values of critical forces in accordance with the yielding limit divided by the design resistance (R_y), the numerical values of which are given in Table 1

$$\varphi=\frac{\sigma_{cr}}{R_y}$$

(2)

Table 1.

Numerical values of coefficient of longitudinal bending

λ	φ at I_2/I				
	0.52	0.53	0.54	0.55	*For* R_y=24kn/sm^2
40	0.914	0.916	0.918	0.919	0.894
50	0.909	0.911	0.912	0.914	0.852
60	0.902	0.904	0.905	0.907	0.805
70	0.894	0.895	0.897	0.898	0.754
80	0.883	0.885	0.886	0.887	0.686
90	0.870	0.871	0.872	0.873	0.612
100	0.853	0.853	0.854	0.855	0.542

The values of φ taken from the norms are shown in the last column of this table [3]. The table shows the advantages of authors' approaches compared with existing norms. For example, in the case of λ=100, φ =0,853 obtained by the new approaches are about 15% higher than φ (φ=0,542) that is taken from the norms. This 15% is an additional reserve for compressed elements during disasters. The calculation of "Arch with a Cable" new type pre-stressed trusses with a new approach in the elastic and elasto-plastic areas of the working of material should be noted. Figure 6 shows a comparative analysis of the work of the truss for various spans. The diagram shows the advantages of working the truss in the elasto-plastic area of the working of material, and the weight difference is about 10%.

The equations of critical length of the rod and the development of the depth of the plastic zone (a) in the cross section of eccentrically compressed elements when working in the elasto-plastic area are set out by the authors [13]:

$$l^2=\frac{1}{2}\cdot b\sigma_y\frac{\pi^2}{\varepsilon_y}\cdot\frac{1}{\left(P_y-P\right)}\left(h-a\right)^2$$
$$\cdot\left(\frac{P_y-P}{P}\cdot\frac{h+2a}{6}-6\right)$$

(3)

$$a=2e\frac{P}{P_y-P}\quad\text{or}\quad a_{cr}=2e\frac{\sigma_{cr}}{\sigma_y-\sigma_{cr}}$$

(4)

a_{cr} -the value of critical parameter and size of yielding are obtained from the equation at the given value (3) of P - force and e - eccentricity (Fig.12). These values a_{cr}, P and e are put in the 4th formula and the critical length of the rod- l_{cr} is calculated. The critical condition of the rod is characterized by three associated values: e - load applying eccentricity; P – the value of compressing force of the rod; l_{cr} - The critical length of the rod [13].

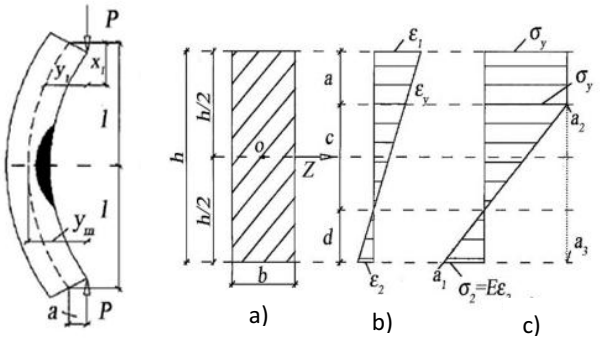


Fig. 12. Distribution of relative deformations and forces in the cross section in case of unilateral yielding

avoid such occurrence and published in various journals. (**Fig. 13**) [14].

2.3 Predicting disasters.

The Sendai Framework also covers the prediction of the risk of disasters. In this regard, the authors have proposed the idea of setting up sensors and gyroscopes on existing and constructive hazardous buildings to obtain data (Fig. 14). Also, it will be important to prepare an appropriate computer program for automatic processing of the results. Based on the results obtained from the sensors, the authors proposed to prevent the risk of disaster using the method of weak fluctuations. This method involves studying the behavior of a

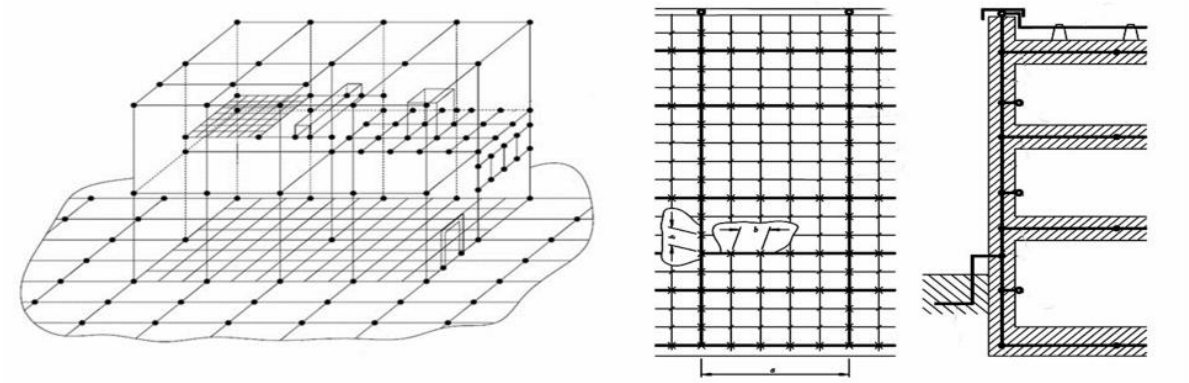


Fig.13. Designing of the lightning protection systems on the building

In addition, the calculation of buildings in the elasto-plastic area is also important from the economic point of view. The ideology of the building design requires the building to be calculated for an earthquake of two intensities. One will be the design, which is the earthquake during weak exploitation. For example, by 7.0- magnitude. At this time, the building can be calculated within the boundaries of the elasto-plastic area of working of material or during the limited development of plastic deformations. This is the case when the construction keeps the usefulness for exploitation. If an earthquake of 8 - magnitude is likely to be expected. The building should be calculated as elasticity for an earthquake with a magnitude of 8.0. It should be calculated at the elasto-plastic stage while plastic deformations are developed with unlimited deformation. i.e. while construction can no longer keep the usefulness for the exploitation.

2.4Reducing the risks of disaster caused by lightning.

The certain extreme conditions represent lightning and protective systems of buildings against it. The cases of destruction of the foundations of multi-story buildings, foundations of which are located on watery grounds, were observed to be caused by the effect of electrohydraulic impact, and conditions were found to building in weak fluctuations to assess its actual technical state or to determine its dynamic characteristics [15]. It is distinguished by its cheapness, mobility; the authors have developed a mathematical model and the algorithm formulated on it, that uniquely determines how the motions recorded under weak influence should be processed in order to reasonably discuss the behavior of a building during the passage of a real seismic wave (seismography) into its foundation.

3. Conclusions

Fig. 14. Gyroscope

A number of approaches, given in this article, allow reducing the risk of disasters. Namely:

2.3 It is advisable to use suspension systems of bridges, pre-stressed systems during floods and mudflows, and transform systems should be used in critical conditions.

2.4 Different approaches should be used to avoid the devastating consequences of the earthquake: high and big span buildings should be built with light structures.

2.5 The elasto-plastic area of working of material should be used in the calculation of buildings in case of earthquakes, which is an additional resource for construction, in terms of load carrying capacity.

2.6 Effective lightning protection systems of buildings should be used to prevent electric hydrodynamic strikes, especially in the case of foundations of multi-story buildings that are located on wet grounds.

2.7 The pre-stressed effect should be used in big span buildings during heavy snowfalls, which makes it possible to increase load carrying capacity and reduce deformations.

2.8 Special sensors-gyroscopes should be installed on buildings having highly responsible requirements and the prospective disaster risks should be predicted during the processing through automated management system of the received signal.

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INTERNATIONAL REGULATION OF CONSTRUCTION DEVELOPMENT

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Key words: construction, Eurocode, European Union, United States, United Kingdom

Abstract

The construction industry is increased and significant in the whole world. Demand of the construction industry product is increasing day by day. Construction developer companies are building and supply to the society. This is very important product for countries and their citizens. This is product, which is staying on the place it is built. It has very high value and because of reason governments of developed countries are trying to work out regulations of construction.

Construction regulations are different in different countries. European Union worked out common regulations for all European Union states. European Union construction regulations called Eurocodes, which are complex and consist 58 parts. Eurocodes fundament is Great Britain's construction regulations. In the article, it is discussed Great Britain's construction regulations.

Construction regulations are paid attention in the United States of America. U.S. construction regulations are 752 pagedocument and include the most important issues for construction.

In the following article it is discussed construction regulations in European Union, Great Britain and U.S.

1. Introduction

The construction is the industry, which is important for both the developed and developing countries. It is an important sector of the economy, it includes planning, designing, construction and etc. processes. The construction if it will be performed by the public sector, or completed by private companies is process, which contributes to the country's economic development process.

The construction industry has a special role for the growth of the country's economy. Besides the fact that it helps to create jobs, this industry is needed for citizens because through the industry is created one of the main values, houses, apartments, office spaces, etc.

Economic growth is an important and strategic goal of each country. Building business is an important factor in the economic growth of the developing countries. The economic situation of the country is well reflected on the construction business. While the country's economic growth slows down, the construction process is going to decline when other industries continue to operate in the short term.

Because of the intensive growth of the building, the government's competence is not only to stimulate and promote construction, the government are commit to control construction processes. There are regulations in the developed countries in which construction developer companies can implement their own construction projects. The growing number of constructions is a positive influence on the economy of the country, but the uncontrolled construction and use of resources will bring the urban area into a deadlock. Governments have control tools, by introducing various laws and regulations. The ecological problems in the modern world are very urgent, so construction regulations also call to the construction development companies to protect ecosystems.

Work presents, the construction development's regulatory international trends. There is discussed the European Union's regulation about construction industry, there is analyzed Great Britain regulation and laws which must be complete construction companies. In the paper is showed what is international construction industry regulations and what kind of regulations has United States of America in construction industry.

2. Eurocodes and Great Britain construction regulations

The modern buildings in European Union are economical, and they are sparing consumers' taxes on electricity; Also they are making economy of natural gas, as well as saving natural resources. According to European Union Commission recommendations, developer companies are required to save 40% of the electricity and 36% of the natural gas in the buildings they will build. Modern buildings built in EU countries consume three times and five times less natural gas per square meter in a year, than old buildings, that require an average 25 liters of fuel per square meter and some buildings need more than 60 liters.

At the present, 35% of EU buildings are over 50 years old. In order to increase energy efficiency, EU countries are in the process of replace old buildings with new ones and save resources. Currently resources are saved from 5% to 6%, while natural consumption is reduced by 5%.

European Commission has published Directives in 2012 in which the priority direction is energy efficiency. [3]

European Commission has published important construction directives related to energy-efficient:

- When a construction developer company buys or purchases construction product, it is mandatory to present certificate about its energy efficiency.
- EU countries should have published checked schemes by experts about heating and air condensation of buildings, which will be appropriate to the standards published by EU.
- New building should be close zero energy buildings. (Till 31 December of 2018 public institutions should meet this requirement). [4]

EU regulations should include standards for energy efficiency:

- EU member state countries should have the funds allocated from national budgets that will be used for energy efficient constructions.
- At least 3% of EU member states' public structures should be placed in energy-efficient buildings.
- Governments of EU member states must purchase only energy efficient buildings for further exploitation.
- EU member states must have national construction strategies that will be directed towards energy efficiency. [3]

European Commission issued a Directive, by this directive for 2018 year, European Union member countries' governmental buildings, where placed public institutions should be close to zero energy expenditure and they have to try to be enormous energy efficient. According to the above Directive, maximum number of public institutions should be placed in such buildings by 2018, according to the Directive, the time specified by countries up to 2020. This is a 23-page document published on May 19, 2010. [12]

The Eurocodes are codes, which design pan – European structural design. There are ten Eurocodes and they include 58 parts. They are: basis of design; actions on structures; design of structural elements in concrete, steel, composite steel and concrete, timber, masonry and aluminum; together with geotechnical and seismic design. They cover the design of bridges, buildings, silos, tanks, pipelines, towers, masts and more.

In 1975, the commission of the European Community has decided to create construction regulations, which are based on Article 94 of the Treaty of Rome. The main purpose to create this program was elimination of technical obstacles and make harmonization of technical specification in all states of European Union. There were different design construction codes in Europe. The Eurocodes harmonize technical specification as public entities as well as private sector of construction industry.

The Eurocodes are limit state codes like the British Standards, although are perhaps a little more explicitly based in reliability theory. Many of the Eurocode rules are based on the same theory as the British Standards, although the Eurocodes embody the most up to date research on many aspects of structural behaviour.

The Eurocode clauses are structured in a slightly different way in that they contain principles that must be satisfied and application rules that offer a way of satisfying the principles. This is intended to stimulate innovation. The Eurocodes are also less prescriptive than the British Standards, with more aspects left open to the designer. [11]

Under the Public Procurement Directive, the Member States must accept designs to the Eurocodes. Eurocodes are the standard technical specification for all public works contracts: where a contract for a public body contains a technical specification this should reference the appropriate European Standards. Accordingly, the Highways Agency has been expressing its requirements in terms of Eurocodes for the design of new highway structures in contracts made after 31 March 2010.

Whilst it will not be mandatory to design to the Eurocodes in the UK, should a designer propose alternative design standards, they will have to demonstrate that the alternative is of 'technical equivalence' to a Eurocode solution. As the British Standards Institution (BSI) will not maintain the withdrawn UK standards, in reality there will be little option but to use the Eurocodes.

The numerous studies carried out so far indicate that the Eurocodes will have little or no effect on member sizes and quantities in common design situations. This has been confirmed by the trial / parallel design studies commissioned by the Agency. The less prescriptive approach of the Eurocodes should also allow greater scope for innovation and encourage designers to use advanced analysis techniques which, combined with the more open marketplace and greater competition brought about by the implementation of Eurocodes, should lead to better value for money.

It is worth noting, though, that at least initially the actual design process is expected to be more costly - due to designers having to become familiar with the Eurocodes.

The Eurocodesare regarded to be safe and reliable and have already been used in Europe on a number of landmark structures. [11]

The Agency is reviewing the Eurocodes with regard to safety. The partial safety factors in the National

Annexes are determined by the UK and will ensure a level of reliability compatible with the existing UK standards. This is backed by a number of trial / parallel design studies commissioned by the Agency, which show that Eurocode design was largely comparable with design to British Standards.

In United kingdom of Great Britain has been paying great attention to the construction regulations. The final revised version of the construction regulations were published in 2010, which include construction regulations for England and Wales. The document is 56 pages and consists of 9 parts.

Introduction and explanations are presented in the first part. The second part discusses the control tools of the construction process. The second part includes following issues:

- Explanation of construction
- Mandatory requirements for begin construction process.
- Construction materials and their composition / substitutes
- Requirements for the construction materials
- Quality of the materials
- Developer companies' rights during construction process
- Conditions for developer companies to avoid construction procedures. [8]

The third part discusses the notes, plans and certificates, which should have the construction development company to start construction. A developer company should present building project plan, also the company should present environmental plan. The developer company should present conclusions of experts, in which should be explained why the project is significant for the suburb development. The developer company should present the plan where will be discussed detail how the company is going to complete the project.

The fourth section describes how the local self-government entities should monitor the construction project. According to British regulations, it is necessary to monitor each construction project at each stage of its implementation by local self-government entities. According to the fifth part of the construction regulations, the developer company should submit documents confirming compliance its construction project to the standards. United Kingdom's construction regulations' fifth chapter has been dedicated to energy efficiency requirements. It contains energy efficiency requirements. The construction developer company, before beginning the construction project, it should be determined the status of energy efficiency of the developer company. It should respond to elements of energy efficiency, which are determined by international standards. The developer company should explain in its documents the methodology how energy efficiency is mounted. The methodology should be clear and understandable. In the regulations also, it pays attention to use water resources.

In the eighth part of the regulations is defined, information about security of persons, who will placed in the buildings. The company should present detail information about fire – fighting, ventilation system, gas and electrical energy supply and about the security system, how it is possible to come out the people during the accident.

The developer company has not just presented these systems of security. The experts and professionals should test the systems. The company should present recommendations and certificates.

In the last tenth chapter of UK and Welsh construction regulations, it is discussed how the buildings should be tested after their completion. After the implementation of the project, its will be inspected construction materials or different security systems. In the 10^{th} chapter also discusses which sanctions are valid if the developer company fails to hold a pre-defined deadline and cannot complete the construction project. There are sanctions if the building built by company cannot tested by the government entities.

The British pay great attention to the construction regulations. It ensures on the one hand, to create barrier to residents and non-residents to invest in construction industry and start building process in the United Kingdom and on the other hand, guarantees that buildings will be high quality and will satisfy customer demand as at the present, as well as in the future.

3. The United State construction regulations

Regulation process of the construction industry in the United States began in 1800. The construction code was created in 1800, which was mainly regulated construction processes of the densely populated urban areas. Initially, the building code included primitive construction directives. With the development processes of technologies, the demands of the society was changing and accordingly was changing the construction code. Gradually, the health, safety and well-being of citizens, as well as natural risks, such as earthquakes, have become important for the society.

The construction code has been developed from professional staff of various fields. It is a combination of high-level regulations that are developed by professionals. Nowadays, in the United States of America, the developer companies need confirm the

requirements of the construction code to start building process.

In the 1990s, three organizations (BOCAI, IBCO, SBCCI) were incorporated in the framework of the International Construction Code Council (ICC International Code Council) and in 2000 published International Standards of Construction Code. International standards have published three types of construction:

- International construction code - includes all types of construction, whether it is offices, residential, governmental or industrial structures.
- Construction code of international residents - includes the international standards of private housing.
- International Construction Code for Existing buildings- includes standards for restoration and modification of existing buildings.

The International Construction Code was published by ICC 2000, 2003, 2006, 2009 and 2012. In the all states of United States, the law has established the legitimacy of the regulations set by the construction code and the developer companies commitment to comply with the construction code regulations.

The construction code published in 2012 is a 752-page document containing important points :

- The standards of buildings and other structures.
- Construction code requirements for reinforced concrete
- National Design Specifications
- Requirements for stainless steel used in construction.
- Construction codes requirements and specifications for concrete and stone used in construction.

The building code is approved by the United States Government to ensure the health, safety and well-being of their citizens, which should be met by the customer's product requirements. The construction code includes all-important aspects of construction, structural stability, fire safety, ventilation, air conditioning system, electronic system as well as energy efficiency.[13]

4. Conclustion

The construction product has longevity use of all other types of products. Thus, when the developer companies create their own products, they would take into account the needs of future. The developer companies should think what kinds of requirements would be in future after several years and if the product created at present time will be relevant for the future customer's requirements. Governments of developed countries are trying to develop different types of regulations that will enable development companies to build future buildings and make a better world.

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THE DEVELOPMENT OF PRACTICAL METHODS FOR THE CALCULATION OF THIN-WALLED ROD SYSTEMS

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Keywords: Thin-walled, Rod, Stability,Wind loading.

Abstract

In the modern construction, in the manufacture of load-bearing structures, the rod systems are increasingly used. They could be used to ensure the required mechanical properties and small deadweight of the structure. The thin-walled metal profiles are often used in the construction of lightweight building frames and structures, although when creating such spatial systems, the mechanical properties of the thin-walled rod elements and the load structure are not taken into account.

The paper dwells on the calculation of the elements of the thin-walled rod systems on the stability with account for the static and dynamic forces. The developed methodology allows for achieving the minimum deadweight and reducing its manufacturing costs by optimizing the geometric parameters of each element.

1 Introduction

The thin-walled rods have been extensively used in construction industry as structural members of frames, farms and as autonomous members. Such an important property, as lightweight, can noticeably affect structural-load capacity. As practice shows, due to errors in building design, accidents are not infrequent. Therefore, the Lightweight steel thin-walled structures, before their introduction in construction industry, should be subjected to a detailed analysis on load- bearing capacity.

The main form of loss of load- bearing capacityof thin-walled steel rods working in compression is loss of stability, rather than strength. Loss of stability occurs at stresses smaller, than at loss of strength. As a result, actual member load- bearing capacity appears significantly below the limit value on strength of cross-section. Consequently, the design procedure of such member is based on the calculation of its load-bearing capacity by loss of general stability.

Slenderness of a rod is essential in the resistance of rods to a longitudinal bending. Therefore, the question on the shape of the cross-section is no less essential, than the question on the value of the cross-section area. As practice shows, an expert, as the most favorable there should be recognized as *closed sections* - circular, as well as box-shaped thin-walled cross-section. The continuous rectangular and *open sections* are considered unsustainable. The nature of the stress-strain state of thethin-walled rods, under the influence of external loading differs significantly from such in the thin-walled closed section rods and in the solid cross-section beams. If to compare the torsional angles at free torsion, then at the ratio of a thickness to diameter as 1/10, the open-section pipe is twisted approximately 300 times more [1].

As to the choice of material, for the rods with great flexibility, the use of high-strength steel is inexpedient. That is because the modulus of elasticity, in this case, is the only mechanical characteristic defining resistibility of a rod at loss of stability, and its size practically is identical for various grades of steel. For the low-flexibility rods, the use of high-strength steels is found to be beneficial, since with an increase of a yield strength, the critical stresses, as well as marginal stability are also increased.

2. Methodology

When considering a problem of stability of the central compression of rod, it has been established, that when compressive force reaches some value, an originally rectilinear form of equilibrium becomes unstable. Alongside with the linear form of equilibrium, there are also become possible the uncountable flexural modes of equilibrium: flexion occurs in a plane of the smallest stiffness.

Depending on the purpose of a problem and its initial data, it is possible to highlight the following three types of stability calculations [1].

I. Checking calculation. The objective of this calculation consists in an assessment of stability of the compressed member under the action of specified external loading. The problem is solved in following sequence:

1. Slenderness ration of a rod is calculated.
2. Depending on elasticity, there is chosen the formula, according to which the stability calculation is carried out. Conditionally, the rods are divided into three groups:

a) The high-slenderness rods, for which the calculation is carried out by Euler's formula,

b) The medium- slenderness rods, the stability calculation is carried out by Yasinsky's formula;

c) The low-slenderness rods, for which the stability calculation is not carried out, and only their strength is tested.

3. Critical force is calculated.

4. The stability factor is calculated

5. There is checked whether the condition of stability is observed by comparing design stability factor with the standardized value.

II. Calculation of a permissible load. In this calculation, there is determined the maximum permissible value of compressive force, at which the required marginal stability will be ensured. By the derived value of critical force and a specified stability factor, the value of compressive force is calculated.

III. Design calculation. The purpose of this calculation consists in determining the required dimensions of a rod, at which it will have a required marginal stability. Stability calculation is the most common method, which is included in the design norms.

With the aim of ensuring safe operation of structural members, which are being under the action of compressive load, it is necessary, that a allowable loading is less than critical value. From the practical point of view, of interest is only the lowest value of critical force at which there occurs loss of stability of a rod. While pressure is within the proportional limit, the lowest value of critical force is described by Euler's formula:

$$\mathbf{F_{kr} = \pi^2 E J_{min} / (\nu \ell)^2,}$$

where,$\nu\ell$ – is a reduced length of a rod;

ν – length reduction coefficient.

As can be seen, the lesser is ν, the more is a critical loading, and consequently, allowable loading of a rod. For example, loading of a rod, built-in by two ends, maybe 16 times more than loading of a rod, which is stiffend by one end. Therefore, where probably, it is necessary to stiffend both ends of a rod. However, this not always can be carried out in practice. Under the action of critical loading$\mathbf{F_{kr}}$, in the cross-sections of a rod, there arise the normal stresses named also the critical ones, we have:

$$\mathbf{\sigma_{kr} = F_{kr} / A = \pi^2 E J_{min} / A(\nu \ell)^2,}$$

where $\mathbf{\sigma_{kr}}$ – a critical stress;

A – a rod cross-section area.

or

$$\mathbf{\sigma_{kr} = \pi^2 E / \lambda^2;}$$

$\mathbf{\lambda = \nu\ell / i_{min}}$ - the dimension-less value, which is called slenderness ratio of a rod, and characterizes its capacity to resist to a curvature depending on the dimensions and a method of end fixity. As can be seen, the critical stress depends only on elastic properties of material and slenderness of a rod. The larger isλ, the smaller is $\sigma_{kr}$and the smaller compressive force is required to cause a rod buckling. To determine its precise value, the successive approximations and linear interpolation methods are used.

The thin-walled rod structures are most commonly used in roofs when constructing and reconstructing buildings and premises. Therefore, in the calculations, in addition to static loads, it is necessary to consider dynamic - wind loads. Structural calculation on wind loading has become inseparable from technological issues, when overall reliability of system is considered or its optimum decision is found out. Perfection of a design procedure of constructions, and introduction of high-strength materials have led to the increase of slenderness of structures and their lighting. This made us to leave behind looking at wind loading only as at static loading [3].

Earlier, wind gusts were considered in the form of the elastic impact on a construction, and were taken as dynamic factor in the calculation. Under such an approach, the question remained open on the influence of the wind gusts that has led to introduction of repeatability factor.

Wind loading on a construction depends on wind velocity and gustiness, dynamic characteristics of a construction, aerodynamic factors of the form, the dimensions and position of a construction relatively to stream. Aerodynamic factors are determined on experiences. The lightweight steel thin-walled structures are used in the framings of buildings which are exposed to wind loadings. According to the authors, in the calculations, it is important to consider wind loadings, which in case of the lightweight steel structures of hothouses, storage sheds and so on, represent the basic type of loads [3].

Wind loading on a construction can be determined by the formula

$$\mathbf{Q = n S c_x b q S}$$

where **n**— overload factor of introduced in the calculation of constructions on a limit state;

$\mathbf{c_x}$— aerodynamic coefficient — head drug coefficient;

q — a normative high-speed pressure of wind at a level of the middle of the considered area of a construction;

β — the factor named dynamic one, which takes into account the reaction of a construction on the action of wind gusts;

S — the projection of the area of a construction to a plane, which is normal to the direction of wind.

The rated wind pressure:

$$\mathbf{q_r = n\, q_0\, (1 + x\, m)}$$

where $\mathbf{q_0}$ — a normative wind pressure;

m — pulsation factor of wind velocity pressure, or the dynamic additive determined statistically;

ξ –the dynamic response factor, which takes into account the reaction of a construction to turbulence of wind.

3 Conclusion

More stringent requirements to strength and reliability at reduction of material consumption, create complex problems of the analysis of the stress-strain state and stability of thin-walled structures. In this connection, one of the main tasks is correct selection of calculation and design methods. The account of wind loading, together with static loads, will allow for designing more reliable constructions at the lowest material costs..

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THE FIRST LIMIT STATE AT THERMOMECHANICAL LOADING

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Keywords: Bar, overload coefficient, limit state, force.

Abstract
In operation formulas for analyse of bars on stability by the first limit state in case of thermo mechanical loading are offered. The axial load of the straight member is considered. We make assumptions, that all cross sections of the bar remain plane during the deformation and temperature field is stationary. Stability condition on the first limit state at thermomechanical loading is presented.

1 Introduction
In the design of any structure it is necessary to use method of limit states. The method, developed by N. Streletsky and A.Gvozdev, expends of stock coefficients and demands deep studying of the actual operating conditions of the building constructions operating on them loadings and properties of construction materials [1]. Specifics of this method consist in special approach to definition of actual loadings and actual strength. In order to simplify of analyse the forces and displacements those arising in the members is determined by an elastic stage.

In actual practice temperature in the members is distributed un uniformly that leads to the addition internal stress. Stability condition on the first limit state registers for mechanical loading and influence of temperature is poorly studied. Therefore, it is important to study influence of un uniform distribution of temperature on the magnitude of actual loading and the choice of the corresponding coefficient of an overload.

2. Methodology
Consider thermomechanical loading of straight bar by axial force and temperature field, that is symmetrical according to xz plane (Fig.1). We use assumptions that temperature is constant during x,z axis and changes during axis y, stress state is one axially $\sigma_x = \sigma \neq 0; \sigma_z \approx 0, \sigma_y \approx 0$. Full strain is a summer of elastic strain and thermal strain:

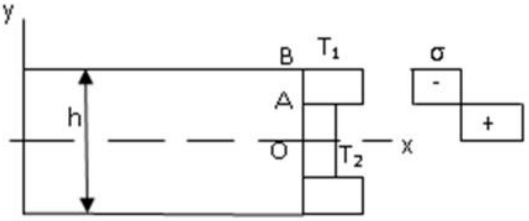

Fig.1Distribution thermal stress in the cross section

$$\varepsilon = \varepsilon^e + \varepsilon^T = \sigma / E + \alpha \Delta T$$

where: $\alpha =$ the linear coefficient of thermal expansion,

$\Delta T =$ the change in temperature,

$E =$ modulus of elasticity.

From last equation we have:

$$\sigma = E\varepsilon - E\alpha\Delta T \quad (1)$$

In the case of thermal loading force equilibrium requires the magnitude of the resultant force developed by the stress distribution to be equal to zero. Using equation (1), we have:

$$\int_A \sigma dA = 0 \quad ,$$

$$\int_A (E\varepsilon - E\alpha\Delta T) dA = 0 \quad (2)$$

In the case both thermal and axial loading, force equilibrium requires the magnitude of the resultant force developed by stress distribution to be equal to P

$$\int_A (E\varepsilon - E\alpha\Delta T) dA = P \quad (3)$$

We will make the following assumption: all cross sections of the bar remain plane and perpendicular to the longitudinal axis during the deformation. Based on this assumption, from equation (2) we define:

$$\varepsilon = (\int_A E\alpha\Delta T dA) / \int_A E dA \quad (4)$$

Substituting equation (4) in the equation (1), we have:

$$\sigma = E[(\int_A E\alpha\Delta T dA) / \int_A E dA - \alpha\Delta T] \quad (5)$$

In the case, when $\alpha = const$ and $\Delta T = const$ in cross section equation (5) issues $\sigma = 0$.

When $E = const$, equation (5) gives:

$$\sigma = E[(\int_A \alpha \Delta T dA) / A - \alpha \Delta T] \quad (6)$$

where A - area of cross section.

Consider, for example, the undeformed bar which has a rectangular cross section.

$$\sigma = E(\frac{2}{h} \int_0^{h/2} \alpha \Delta T dy - \alpha \Delta T) \quad (7)$$

In the regions OA and AB (Fig.1) $\Delta T_1 = T_1 - T_0, \Delta T_2 = T_2 - T_0$, where T_0 is a initial temperature. Using equation (6) we define stress. In the region OA

$$\sigma = E\alpha(T_1 - T_2)(1 - 2h_1 / h)$$

and in the region AB

$$\sigma = -E\alpha 2h_1(T_1 - T_2) / h$$

where $T_1 > T_2$, $0 < h_1 < h/2$, $h_1 = |OA|$ - length of region OA

Consequently, if an internal axial force at the section is tensile, the region OA is reloaded by thermal stress, and if an internal axial force at the section is compressive, the region AB is reloaded by thermal stress, that must be considered in design.

It is important note, that non homogeneous distribution of temperature leads both normal and shear stresses. The shear stress may be determined using equilibrium equation of theory of elasticity.

Unsymmetrical distribution of temperature in the cross section is considered in the article [2].

3. Results

Based on the abovementioned and using equation (5), for a case of an axial loading and the symmetric distribution of temperature in a cross section of the bar, stability condition on the first limit state has an appearance:

$$N_{con.} n_1 + N_u n_2 + n_3 AE(\int_A E\alpha \Delta T dA / \int_A E dA - \alpha \Delta T) \leq AR^H \cdot k \cdot m$$

where: N_{con}, N_u - are normal forces respectively from constant and temporal loadings, n_1 and n_2 - are overload coefficients on loadings, n_3 - is an overload coefficient on temperature, A -is the bar cross-section area, R^H - is the normative resistance of material, k - is the coefficient of nonhomogeneous of material, m - is the coefficient of working conditions.

Coefficient n_3 defends both relations h_1 / h, T_1 / T_2 and deviation of temperature from normative quantity.

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GEOMETRICAL ANALYSIS OF CAVITY DEFECTS BEHIND LINING IN HIGH-SPEED RAILWAY TUNNEL

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Keywords: tunnel lining, lining defect, lining thickness, cavity geometry cavity, probability of cavity.

Abstract

Analysing the quality defects on-site data of tunnel through the probability and mathematical statistics analysis, the distribution quality defects and diseases of lining are found, the general law of lining behind contact state is statistically analyzed, and the geometric distribution characteristics of cavity behind lining and the safety of the structure are obtained. The conclusion is that the strength of lining concrete is not the main defect of lining, and lack of thickness and presence of cavity behind it are important factors affecting the safety of the tunnel structure. The conclusion of this paper is meaningful to study the safety effect of tunnel structure.

1 Introduction

With the advance of the urbanization process, the demand for monitoring and monitoring of the tunnel traffic network is increasing. In the face of the huge test data, it is an important research topic to analyze the scattered data. Therefore, it is necessary to systematically and comprehensively analyze the lining defects of tunnel lining structure in order to formulate targeted and reasonable measures for tunnel lining disposal, so as to effectively control the occurrence and development of tunnel defects and prolong the service life of tunnels, Master the actual operating state of the tunnel.

2 The main types of tunnel lining defects in high - speed railway

When the lining is in close contact with the concrete, the lining structure can withstand the pressure of the surrounding rock, and it can produce sufficient formation reaction force, so that the lining structure is in a good state of force, and when the lining is in poor contact, the stress of the lining structure is deteriorated Conditions, making the lining of the local stress concentration, serious lead to the lining of the structure of cracking, loss of pieces and other diseases [1]. In order to fundamentally correct the tunnel, it is important to make clear the main types of tunnel lining contact. The internal types of the lining of the tunnel include: insufficient lining strength, insufficient thickness and voids.

3 Statistical analysis of lining concrete strength

The strength of the concrete lining is measured and the statistics are shown in Table 1:

Table 1.Statistical table of lining concrete strength

Item / Name	Number	Lining concrete strength		
		Qualified	Unqualified	Pass rate
Tunnel 1	9	8	1	88.89%
Tunnel 2	23	21	2	91.30%
Tunnel 3	17	16	1	94.12%
Tunnel 4	11	10	1	90.90%
Tunnel 5	16	14	2	87.50%
Tunnel 6	19	17	2	89.47%
Tunnel 7	14	13	1	92.86%
Tunnel 8	9	9	0	100.00%
Tunnel 9	6	5	1	83.33%
Tunnel10	7	6	1	85.71%
Tunnel11	12	11	1	91.67%
Total	143	130	13	90.91%

It can be concluded that the lining of the tunnel is better, and the strength of the lining concrete is basically 90.91%. Obviously, the lining concrete strength is not the main defect factor to detect the lining state of the tunnel.

4 Statistical analysis of the thickness of lining and the geometrical characteristics of voids behind lining

Tunnel design specifications require tunnel lining and the surrounding rock close fit, in fact, there is often between the lining and the surrounding gap or between the two backfill is not dense, this phenomenon is called the lining behind the cavity. Professor ZHANG in the Ministry of Railways key scientific research project "railway operation tunnel disease comprehensive treatment technology research and safety evaluation" support, access to the Beijing-Kowloon line, Tan Long line, Beijing-Guangzhou line, the sea line, Chengdu line, South Xinjiang and other railway lines on more than 100 tunnel geological radar non-destructive testing data, statistical analysis of defects in the tunnel with the defect in the detection of the proportion of the tunnel from 11.56% [2]. It should be pointed out that in the tunnel construction process, due to the lack of lining thickness, the design thickness and the actual thickness of the difference, the construction of the collapse of the disposal is not complete, and left behind the lining of the phenomenon is quite common.

4.1 Statistical analysis of the thickness of lining and the geometrical characteristics of voids behind lining **The mean average u_k, standard deviation σ_k and coefficient of variation δ_k of the k (a) ~ (c) of the tunnel lining are analyzed:** $u_k = \frac{1}{n}\sum_{i=1}^{n} u_i$ **(a)** $\sigma_k = \frac{1}{n-1}\sqrt{\sum_{i=1}^{n}(u_i - u_k)^2}$ **(b)** $\delta_k = \frac{\sigma_k}{u_k}$ **(c)**

According to the statistical results, the calculation is shown in Table 8:

Table 2. Statistical table of lining length characteristic result in each defect level for Zhang Wu lining

Characteristics / Defect levels	u_k			σ_k			δ_k		
	vault	hance	sidewall	vault	hance	sidewall	vault	hance	sidewall
1	21	22	17	5.657	7.071	1.414	0.269	0.321	0.083
2	13	7	1	8.485	1.784	1.350	0.653	0.202	1.510
3	2.5	0.5	0	2.121	0.707	0.707	0.849	1.204	1.312
4	0	0	0	0.000	0.000	0.000	0.000	0.000	0.000

Table 3. Statistical table of lining length characteristic result in each defect level for Gao Shan lining

Characteristics / Defect levels	u_k			σ_k			δ_k		
	vault	hance	sidewall	vault	hance	sidewall	vault	hance	sidewall
1	20.9	13	7.8	5.798	9.899	3.253	0.277	0.761	0.417
2	13.8	12.5	2.5	1.697	3.536	3.536	0.123	0.283	1.414
3	4	1.5	1.85	1.414	2.121	2.616	0.354	1.414	1.414
4	0	0	0	0.000	0.000	0.000	0.000	0.000	0.000

Analysis of the tunnel lining defects can be made as follows:

(1) Located in the vault and arch at the lining of the lining of a defect level 1, defect 2 and defect 3 level more, should focus on monitoring and remediation.

(2) The general rule of the mean of the length of the defective lining is located at the top of the vault > arch side > wall, and the position of the lining is the most serious.

(3) The general rule of variation coefficient of length of defective lining basically satisfies the variability of vault > arch side > wall, which indicates that the variation range of defective lining is larger in the dome.

4.2 Statistical analysis of lining thickness

Lining thickness is not associated with the back of the hole is usually linked, according to ensure the normal use of lining and driving safety requirements, the lining thickness is divided into minor, more serious, serious and very serious four categories.

Table 4 Table of lining imperfect defect quantitative indicators Defect levels / Severity	1 Slight	2 More severe	3 Serious	4 Very serious
$1 > h_l / h \geq 0.9$	L		-	-

$0.9 > h_l / h \geq 0.75$	$L < 5$	$L \geq 5$	-	-
$0.75 > h_l / h \geq 0.60$	-	$L < 5$	$L \geq 5$	-
$h_l / h < 0.60$	-	-	$L < 5$	$L \geq 5$

Among them: h_l for the measured the thickness of the lining; h for design thickness; L for detect the in sufficient thickness of lining section of the line length in a row.

The first detection of the first Lao Ling tunnel were detected left and right dome, left and right arch waist and left and right, the edge of the six lines. Due to the need to avoid the tunnel within the catenary and the next anchor and other ancillary facilities, the length of each side line is not the same. On the lining thickness of the defect level analysis, the survey results of the test line in Table 5 to Table 7:

Table 5. Table of left vault lining thickness

Length of the line / Defect levels	Cumulative length/m	Percentage
1	17	21.79 %
2	19	24.36 %
3	1	1.28 %
4	0	0.00 %

Table 6. Table of left arch waist lining thickness

Length of the line / Defect levels	Cumulative length	Percentage
1	27	17.88 %
2	6	3.97 %
3	1	0.66 %
4	0	0.00 %

Table 7. Table of left wall lining thickness

Length of the line / Defect levels	Cumulative length	Percentage
1	18	11.76 %
2	2	1.31 %
3	1	0.65 %
4	0	0.00 %

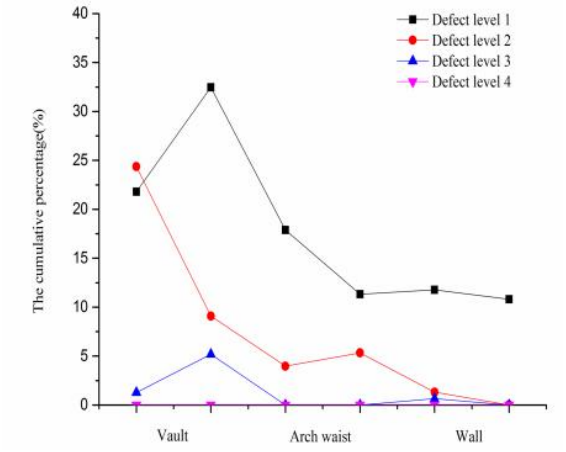

Fig.1 Diagram of defect statistical for ZhangWu tunnel lining

From Table 2 to Table 7, we can draw the following understanding:

(1) The first and second Lao Ling tunnel vault at the location of the lining behind the situation is relatively serious, the edge of the wall at the location of the lining behind the situation, the arch at the location of the lining behind the empty situation is less Detection of 4 defects.

(2) The lining defects of the third ridge tunnel are mainly concentrated in the vault and arch position, and the measured thickness of the dome is less than 60% and the design thickness is less than 60%.

4.3 High probability analysis of voids behind lining under different positions

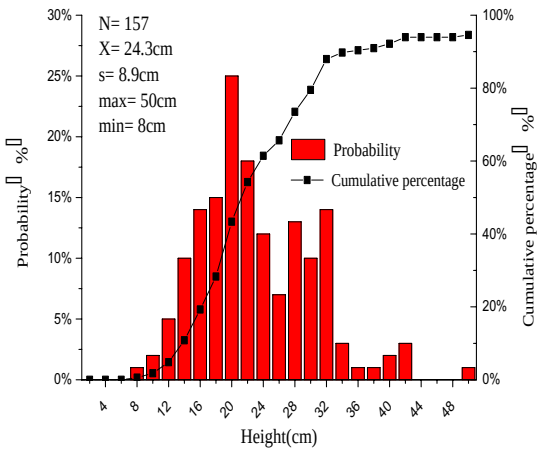

Fig.3 Height characteristic statistical graph of cavity at the arch waist position

From Figure 2 to Figure 3, the following conclusions can be drawn:

(1) The distribution of lining thickness at each location showed a more obvious normal distribution in the range of 8cm ~ 24cm.

(2) 69% of the arch measuring points, 18% of the wall measurements are found empty. This shows that the side wall is fit, not compact degree of small.

5 Analysis of Cavity Probability in Lining Holes under Different Positions

According to the current test data, tunnel lining defects are prevalent in the position of vault, arch and side wall, and the length of defect of tunnel lining is 12.16% of the total length of tunnel. Therefore, there is a hole in the tunnel lining is one of the common quality defects.

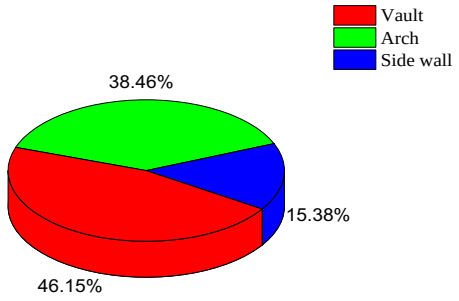

Fig.4

Fig.2 Height characteristic statistical graph of cavity at the vault position

The quantity distribution ratio graph of cavity in different position

From Figure 4 can be drawn, the vault position behind the lining of the situation is relatively serious, arch waist position lining the back of the hole after the

situation, while the edge of the wall lining the situation behind the lightest situation, therefore, should focus on the arch lining to do the focus of remediation.

6 Conclusion and suggestion

The study of more than 10 railway operating tunnel lining contact state, from the mathematical statistics analysis obtained the following understanding:

(1) The existence of voids in the tunnel lining is a more common quality defect phenomenon. The lack of lining thickness and the existence of holes are important factors that affect the safety of tunnel structure.

(2) The probability and geometric mean, standard deviation and discrepancy of the contact defects in the back of the lining are gradually reduced by the order of the vault, arch and side wall.

(3) The distribution characteristics of the lining thickness are related to the distribution characteristics of the lining behind the lining.

(4) Obviously, the ratio of lining thickness or concrete strength to its design value, if smaller, the greater the degree of impact on the lining capacity of the tunnel lining. Similarly, the longer the length of the lining line is longer (or the longer the continuous length of the lining of the concrete is less), the greater the degree of impact on the lining capacity of the tunnel lining.

Acknowledgements

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EXPERIMENTAL STUDY ON MECHANICAL MODEL OF CLOSED CAVITY DEFECTS IN VAULT LINING OF HIGH SPEED RAILWAY TUNNEL

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Keywords: tunnel lining the cavity defects local model train load test design

Abstract

Inside the tunnel lining arch with closed cavity leads to lining structure is complicated and adverse. Especially when high-speed train through the tunnel and on the dynamic load and static load coupling leads to lining edge prone to cracking. In this paper, the behind lining arch tunnel with closed cavity defects a local model is designed. According to the loading characteristics of arch with cavity defects using C type block to undo the composite arch stress environment. Depending on the speed of the train vibration load is simplified to a certain frequency sine curve of load and has a certain amplitude loading force. According to the field monitoring and the way of the combination of finite element simulation to determine the size of the train vibration load transfer to vault. According to the speed at 250 km/h, 300 km/h and 350 km/h three speed under vibration load and the hole Angle range of 0 °, 15 °, 30 ° made C type block. According to the static load and dynamic load under coupling action of C type block experiment and compared with finite element software simulation combined with the evolution regularity of closed cavity, degrade of lining structure.

1 Introduction

Railway tunnel in China every year, about 300 km and even faster construction speed growth. Although our country's high-speed rail technology is the world's advanced level, but according to the 5000 existing railway tunnel in China's statistical data analysis showed that are behind the different levels of tunnel lining cavity diseases, diseases accounted for about 50% of tunnel, as shown in Figure 1-1, including high-speed rail tunnel cavity disease accounted for about 38%. There are a lot of tunnel lining behind contains hollow, Domestic authority for north China do a lot of tunnel lining cavity behind features of statistical analysis.The typical cases such as the ZhangWu tunnel hole distribution as Figure 1-2.

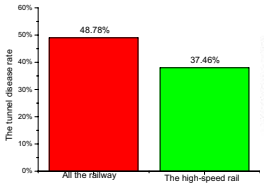


Figure 1-1

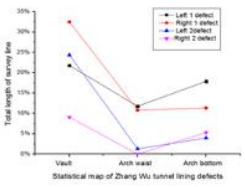


Figure1-2

In the tunnel lining arch, arch shoulder, hance, arch foot and other parts have different degrees of cavity disease.The cavity existence is caused tunnel collapse even one of the most potential instability of whole, falling on the road and cause great potential safety hazard.[1][2]

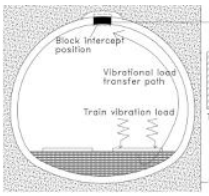
HeChuan[3][4] through the indoor model test of the tunnel, The deformation law of tunnel lining have hollow structure, load-bearing characteristics and disease patterns were studied. Based on more than 100 railway tunnel lining nondestructive testing and statistical analysis. Based on more than 100 railway tunnel lining nondestructive testing and statistical analysis by ZhangDingli[5].In poor contact behind the lining segment is more serious in the section tunnel surrounding rock stability is poorer, and biggest vaults, distribution regularity of arch foot are minimal. Currently on the cavity defects of tunnel under train vibration load of research are relatively few. Because ballastless track is used for high-speed railway and stiffness is larger than ordinary railway track. As a result, the vibration of the train load will have very little attenuation and lead to the vibration of the structure on the energy is very large. That means most of vibration energy will be passed to the lining structure. Therefore, this article in view of the tunnel lining cavity study, by combining the numerical simulation to design the local model test of tunnel lining behind the hole under the environment of train dynamic load on the influence of lining structure.

2 Test ideas

2.1 the selection of test block and similarity theory

Because for lining with cavity very few test under dynamic loading conditions, so we designed a set of simple and feasible C block dynamic load test to reducing the stress of the tunnel lining the environment. Designed to C glyph is the purpose of the test block C glyph block is used to simulate the stress of the tunnel lining. Figure2-1 in the rectangular area is the area test .Two flank edge effect is convenient to test load, is a

kind of structural measures and here is not tested data collection.



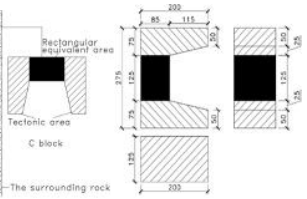


Figure2-1 Figure 2-2

Train vibration load through the track plate, basal cushion layer to the inverted arch, then through the inverted arch is passed to the waist lining arch and vault, the transfer process is bound to cause vibration amplitude attenuation.Select local area containing cavity behind the lining as the key object of study. Figure 2-1 in the black part to choose the lining of the local. Reduced the local model of lining is made after 4 times Figure 2-2 C glyph block style. Black rectangular part of the C block for experimental simulation, two times cross-court delimit flange for test structure, just to give rectangular area provide a certain amount of bending moment and the stress of the tensile stress in order to restore the lining of the local environment, the construction area is not test record data area. Selection is real tunnel lining, the lining thickness is 340 mm, 4 times by similarity theory equation analysis method [7][8]in the scale thickness is 85 mm after the C block, the width of the black area.Local similarity conditions of the model by mechanical relations.

The bending moment of cross section A under load P: $M = Pe_0$ (1)

The positive stresses of A under the load P: $\sigma = \frac{P}{A} + \frac{My}{I}$ (2)

Geometric similarity condition: $\frac{L_p}{L_m} = C_L$; $\frac{A_p}{A_m} = C_L^2$; $\frac{I_p}{I_m} = C_L^4$; $\frac{y_p}{y_m} = C_L$; $\frac{e_{0p}}{e_{0m}} = C_L$

The concrete material used in this experiment is the same as the concrete elastic modulus E in the actual project, which requires the effect of the load P on both sides of the test block, $C_P = \frac{P_p}{P_m}$. At the same time, the bending moment, stress and actual similarity of A cross section of the test model are required, and the bending moment and stress similarity are respectively: $C_M = \frac{M_p}{M_m}$; $C_\sigma = \frac{\sigma_p}{\sigma_m}$;

The similar constants of the physical quantities are arranged: $M_n C_m = P_m e_{0m} \cdot C_P C_L = M_p$ To deduce: $C_m = C_P C_L$ (3)

$$\sigma_m C_\sigma = \frac{P_m}{A_m}\frac{C_P}{C_L^2} + \frac{M_m y_m}{I_m}\frac{C_M C_L}{C_L^4} = (\frac{P_m}{A_m} + \frac{M_m y_m}{I_m})\frac{C_P}{C_L^2} = \sigma_p \quad (4)$$

We can derive it by the same way : $C_\sigma = \frac{C_P}{C_L^2}$, The data obtained by the model is multiplied by the corresponding similar constant. In this experiment $C_L = 4, C_\sigma = 1, C_P = 4$.

2.2 Local test preparation

The die for the c-shaped block in the figure.Because the vertical placement of the C font is not easy to achieve, the horizontal vibration table is attached to the external C and fixed.The pressure of the surrounding rock is simulated by using a spring fitted pad to simulate the surrounding rock.And by adding and cutting springs to simulate the existence and Angle of the hole.A hard rubber is used on the left side of the block to absorb the excess vibration energy and simulate the elastic boundary without the bounce of the vibration wave boundary.The stiffness of spring in the test model K. By applying certain displacement, the test block is subjected to certain surrounding rock pressure.The vibrating table and the test block are connected by the circular curved hinge support to transfer the load, which avoids the force of the block due to the unbalanced force.

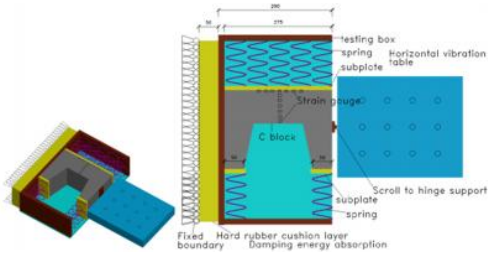


Figure2-3

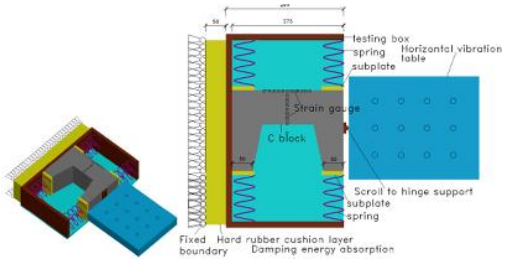


Figure2-4

Concrete of different parameters are determined and experimental study, while pouring concrete block 2 kinds of size, one is C block, the other is 150 ×150 ×150 mm^3 reference block can obtain mechanical parameters of concrete block, plastic molding and 24h after ripping, and 28d standard curing in the concrete curing room.The cement materials are made from hunan cement factory

grade 42.5 ordinary polish cement, the powder coal ash is made from Xiangtan, Hunan Province.The fineness modulus of fine aggregate in the test was 2.67kg/m^3, and the coarse aggregate was made of limestone with grain size of 5~10mm.The strength of the concrete specimen design is C35, concrete and concrete with table 1.According to the speed at 200 km/h, 250 km/h and 300 km/h three speed under vibration load and the hole Angle range of 0 °, 15 °, 30 ° made 27 C type block.

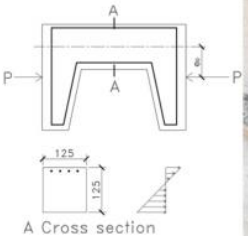


Figure2-5 Figure 2-6

Table 1

Concrete mix ratio	Water/cement ratio	0.4
	Cement / （kg·m^{-3}）	277
	Fine aggregate / （kg·m^{-3}）	747
	Coarse aggregate / （kg·m^{-3}）	1075
	Coal ash/ （kg·m^{-3}）	108
	Water reducing agent/ （kg·m^{-3}）	2.85
	Water / （kg·m^{-3}）	153

The test block can be constructed with reinforcement by calculating the reinforcement. According to the similar theory, the reinforcement can be used in ordinary steel wire.

2.3 The determination and imposition of load

Train vibration load transfer law at home and abroad is still no definite conclusion, China academy of railway sciences the latest monitoring data in the process of gain because of the limitation of equipment and on-site environmental monitoring in the process of the vibration acceleration can detect from the highest orbital location of lining plate of 5 m high.

Figure 2-7

The monitoring results show that the lateral acceleration of the vibration load of the train is greater than the longitudinal acceleration.The dynamic stress data of the measured points is calculated by using the acceleration value of the lining structure, and then applied to the local model test.Due to vibration in the process of design and dynamic load test equipment, simplify the train dynamic load for a certain initial static load and dynamic load amplitude of the sinusoidal loading, according to the different speed were 8.3 Hz frequency of 10.4 Hz 12.5 Hz to simulate the speed of 200 km/h and 250 km/h and 300 km/h when the vibration of the load.The initial static load was 0.9 KN in this test and 1.8 KN.

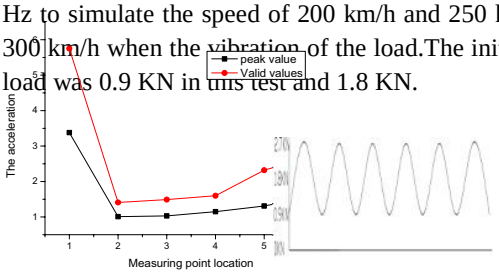


Figure2-8 Figure 2-9

3 Accumulation theory of damage

Based on the continuous media damage mechanics, the method of cumulative damage prediction was established and the study of the law of the evolution of concrete damage was carried out.This paper establishes the relationship between the damage variable and the number of cycles, and describes the development process of the concrete accumulation injury[13].

1)The determination of the damage variables of the concrete under the initial conditions;

2)The value of the damage of the concrete in each cycle of the cyclic load;

3) the damage threshold for concrete damage condition.

In this paper, the nonlinear characteristics of the change rate of the damage are expressed through the damage evolution equation, and the evolution equation of the specific D is as follows:

$$D = \lambda^{pD}\frac{\partial F^{pD}}{\partial Y} = \lambda^{pD}\left(\frac{Y}{S}\right)^{s}\frac{1}{(1-D)^{\varphi}} \quad (5)$$

λ^{pD} is Damage multiplier; F^{pD} Damage potential function:

$$F^{pD} = f + \frac{1}{2}\frac{b}{Q}R^2 + \left[\frac{1}{(1-D)^{\varphi}}\right]^{\frac{s}{a+1}}\left[\frac{Y}{S}\right]^{s+1} \quad (6)$$

a,b,s,φ is Model parameters.

$$D \quad \frac{Z\sigma_{eq}^{r-1}\sigma_{eq}}{\beta^{m}(1 \quad D)^{\Phi}} \quad (7)$$

$Z = \left(\frac{S_0}{2ES_1}\right)^{S_1}\frac{m}{k}$, $r = 2S_0 + m, \Phi,\ \beta, m, S_0, S_1$ is Material parameters.

Expression of a single injury variable:

$$\frac{\partial D}{\partial N} = \frac{2X\ (\sigma_{max}^{r} - \sigma_{min}^{r})}{\gamma\beta^{m}\ (1 \quad D)^{\Phi}} \quad (8)$$

σ_{max}^{r},σ_{min}^{r} is Material stress;The parameter r is the model material constant.

In order to describe the process of concrete damage evolution, the equation of the equation of concrete injury under cyclic loading under the formula is as follows:

$$D \quad 1-[1-\frac{2ZN(\sigma^r_{max}-\sigma^r_{min})(\varphi+1)}{\gamma\beta^m}]^{\varphi+1} \qquad (9)$$

Combined with the formula, the definition of damage variables under initial conditions and damage conditions of concrete damage was given.

$$D|_{N=N_0}=D_0\ ;\ D|_{N=N_f}=1$$

N_0 is the number of cycles in the initial injury condition; N_f is the number of cycles per cycle of fatigue damage; Damage damage judgement is $D_{cR}=1$.Under cyclic loading, the N_f can be expressed as:

$$N_f=\frac{\gamma\beta^m}{2Z(\sigma^r_{max}-\sigma^r_{min})(\varphi+1)} \qquad (10)$$

In order to establish the calculation model for the cumulative damage of the cycle, the following damage evolution equation is given:

$$D=1-[1-\frac{N}{N_f}]^{\frac{1}{\varphi+1}} \qquad (11)$$

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ANALYSIS OF DYNAMIC RESPONCE OF IN-SERVICE HIGH SPEED RAILWAY TUNNEL WITH CAVITY DEFECTS IN LINING

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Key words: high-speed railway; tunnel lining; cavity; dynamic response; numerical analysis

Abstract

With the rapid construction of high speed railway tunnels in China, tunnel lining back cavities become a key problem that has important effect on the safe operation of high-speed railway tunnels. In this paper, based on high speed railway tunnel lining back existing cavities under IV surrounding rock, the finite element model of the tunnel lining with different range cavities is established to simulate the dynamic response of the tunnel lining structure under 250km/h train load by using finite element model of stratum-structure and ABAQUS finite element software. The results show that with the increase of the cavity range, the influence of the vault, side wall, junction of the side wall and the inverted arch is much larger than that of the invert arch under track and inverted arch center. Among them, the tensile stress at the top of the lining and the acceleration at the boundary between the side wall and inverted arch are the most affected. Since the concrete material with the tensile strength far below the compressive strength, the existence of the back hole seriously affects the service performance of the high-speed railway tunnel.

1 Introduction

Aiming to the vibration response of the railway tunnel and the back cavities of tunnel lining, a large number of domestic and foreign experts and scholars have done a lot of simulations and experimental researches[1]-[5].

However, most of them are not involved in the study of high speed railway tunnel lining with back cavities.

As the high speed railway tunnel lining back empties change the state of close contact between surrounding rock and lining causing the mechanical behavior of lining is different from that of close contact, so it is very necessary to study the influence of back cavities on structure of high speed tunnel lining under vibration loads of train.

2 Calculation principle of dynamic response of tunnel train

2.1 Simulation of train vibration load

When the train is running on the uneven track, considering the influence factors of the train vibration load, the vertical vibration load of the train can be simulated by a exciting force function[6]:

$$F(t)=k_1k_2(P_0+P_1\sin\omega_1 t+P_2\sin\omega_2 t+P_3\sin\omega_3 t) \quad (1)$$

The calculation of specific parameters in the formula refer to the reference [6].

In this paper, the train axle load is 20t, and unsprung weight is 750kg.According to the code for design of high speed railway in China, Table 1 shows the irregularity of the vibration wave length and the height of versine when the train vibration load is simulated. Fig. 1 shows the time history of the vertical vibration of the train when the vehicle speed is , which is simulated by the excitation force function of the first two seconds.

Table 1 The value of irregularity vibration wavelength and versine height

Control condition	wavelength/ m	Versine height/mm
Ride comfort①	10.0	3.5
Dynamic additional load on line②	2.0	0.4
Waveform abrasion③	0.5	0.008

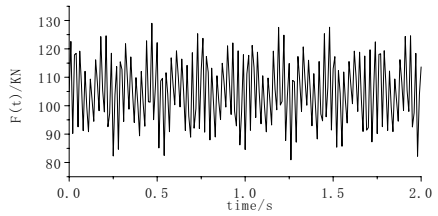

Fig. 1 Time history curve of train vibration load

2.2 Dynamic calculation principle of tunnel structure

According to the principle of structural dynamics[7], the differential equation of vehicle-tunnel system under the train vibration load can be expressed as:

$$M\ddot{X}+C\dot{X}+KX=F(t) \qquad (2)$$

In the formula, M、C、K are respectively structural system mass, damping and stiffness matrix, and $\ddot{X}$、$\dot{X}$、X are respectively acceleration, velocity and displacement vector at each node in the system. What's more, $F(t)$ is respectively train vibration load vector.

The damping mechanism of material is described by Rayleigh damping[7],in which the damping matrix is assumed to be proportional to the mass matrix and stiffness matrix. And the calculation of specific parameters in the formula refer to the reference[7].

3 Calculation model

3.1 Engineering background and calculation conditions

To avoid serious disease of ordinary railway tunnel, Chinese high-speed railway tunnel strengthen the design of bottom structure for III, and above the rock areas, using the two form of inverted arch lining.Fig.2 shows standard section of high speed railway tunnel of IV grade surrounding rock in condition of v=250km/h.

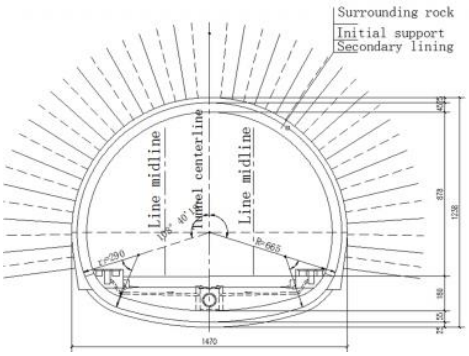

Fig. 2 Standard section of high speed railway tunnel (unit: cm)

In this paper, multi dynamic models of a void in the lining back in different range (shown in Fig. 3) are established to analyze the effect of back cavity on the vibration of tunnel structure under different conditions.

And specific conditions are shown in Table 2.Attentively, the basic calculation model is the tunnel structure under IV levels of surrounding rock, the angle of 30 degree and the depth of 80cm while other computational models only change the operating conditions corresponding to Table2.

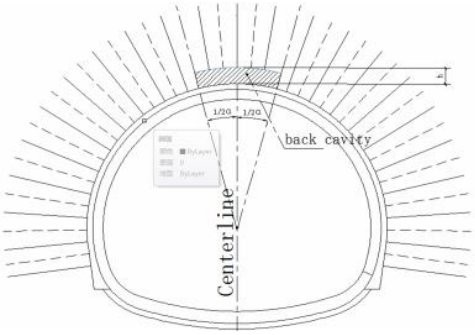

Fig. 3 Schematic diagram of cavity distribution in lining

3.2 Calculation parameters

Table 2 Calculation condition

Influence factor	Working conditions
Back cavity range α	0° 15° 30° 45° 60°

The grade IV surrounding rock materials regarded as a continuous, homogeneous and isotropic medium uses Mohr-Coulomb elastic-plastic constitutive model, and the physical and mechanical parameters are selected according to the current code for design of railway tunnel[8] and code for the design of high speed railway tunnel[9].What's more, the supporting structure is simulated by linear elastic solid element, and the material parameters of concrete are selected according to the current code for design of concrete structures[10].And the basic mechanical parameters of the surrounding rock, the initial support, the two lining, the filling layer, the track plate and so on are shown in Table 3 and Table 4.

Table 3 Physical and mechanical parameters of surrounding rock

rock grade	γ (kN/m^3)	E/ GPa	μ	C /MPa	φ / (°)
IV	21.5	6	0.35	0.45	32

Table 4 physical and mechanical parameters of tunnel structure

Structure name	γ / (kN ● m^{-3})	E/ GPa	μ
Secondary lining(C35)	26.3	31.5	0.2
preliminary bracing(C25)	23	28	0.2
Invert filling and foundation (C30)	25	30	0.2
Track plate(C50)	27	34.5	0.2

3.3 computational grids partitioning

Since the length of the cavity along the tunnel axis is much larger than its diameter, the finite element ABAQUS software plane strain model is used to simulate the influence of tunnel lining back cavity on lining structure under different conditions. And train load is considered by two lines.

In order to ensure the accuracy of the calculation, the calculation range is: the horizontal direction of the tunnel is taken as 45m on both sides of the axial direction, the total length of the vertical direction is 66m, and the tunnel is considered by the shallow buried tunnel, the depth of which is 15m, what's more, the fixed boundary condition is adopted at the bottom and the left and right sides of the model. The computational model grid is shown in Figure 4. And the positions of A, B, C, D and E are respectively represented by the vault, side wall ,junction of the side wall and the invert arch, invert arch under track and invert arch center.

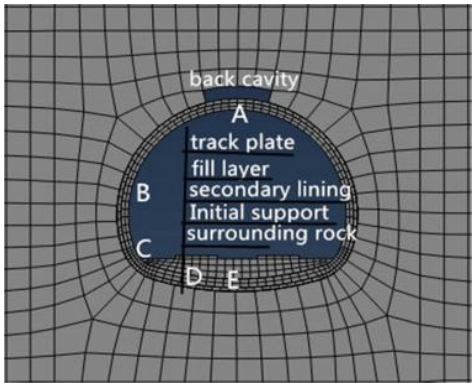

Fig. 4 Schematic diagram of mesh division and analysis

4 Calculation results and analysis

The dynamic response of high-speed railway tunnel is analyzed and compared in different conditions of range, measurement points in the premise of ensuring the train speed, the supporting structure parameters and rock parameters, cavity depth of 80cm and other factors were the same, while only change the scope of the back empty tunnel lining, select the empty range of 0 degrees and 15 degrees, 30 degrees, 45 degrees and 60 degrees. The influence of the cavity on the vibration acceleration, vertical displacement, and the maximum and minimum principal stress of the tunnel lining is shown in Fig. 5, Fig. 6, Fig. 7 and Fig. 8.

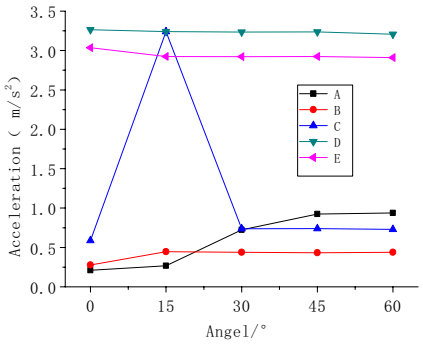

Fig. 5 Relationship between acceleration and the void range

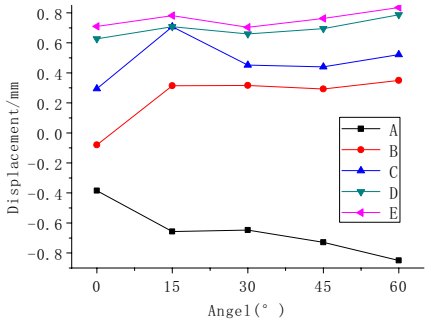

Fig. 6 Relationship between the vertical displacement and the void range

As can be seen from Fig. 5, with the increase of the cavity size, the acceleration of the vault and side walls are increasing, and the growth rate is smaller. However, the change of the acceleration of the invert arch under track and the center of the inverted arch is not large, and basically remain unchanged. With the increase of the void range from 0 to 15 degrees, the acceleration of the boundary between the side wall and inverted arch is changed from 0.59m/s^{-2} to 3.24m/$^{s-2}$, which is 5.5 times than the former, then with the increase of the cavity

range, acceleration is reduced to 0.73m/s$^{-2,}$ and basically unchanged. In general, the change of the back cavity range of the lining has little influence on the acceleration of the side wall, the invert arch under track and the center of the invert, and has a great influence on the acceleration of the vault and the junction of side wall and inverted arch.

As for the change of the vertical displacement, Figure 6 shows that the absolute value of the vertical displacement of each measuring point increases with the increase of the cavity. By the way, with the increase of the void range from 0 to 15 degrees, The maximum vertical displacement of the vault and the side wall, the junction of side wall and inverted arch were increased from -0.38mm to -0.66mm, increased from -0.08mm to 0.31mm and increased from 0.29mm to 0.71mm, the increase rate were 0.28mm, 0.39mm and 0.42mm.It can be seen that with the increase of the cavity size, the maximum value of the vertical displacement of the tunnel structure is increasing, which makes the structure stress is unfavorable.

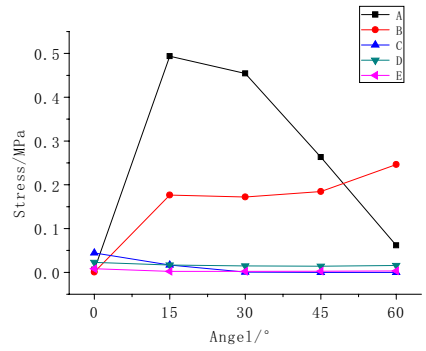

Fig. 7 Relationship between the maximum stress and the void range

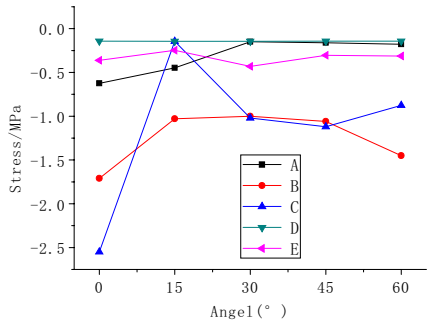

Fig.8 Relationship between the minimum principal stress and the void range

Fig. 7 and Fig. 8 show that with the increase of the cavity size, the maximum principal stress value of the side wall and the side wall, the junction of side wall and inverted arch and the center of inverted arch basically remain unchanged; For the crown, with the empty range increased from 0 degrees to 15 degrees, the maximum principal stress increased from 0.006MPa to 0.49MPa, which is 81 times more than the former. And for the side walls, along with the increase of the cavity range, side walls at the maximum principal stress value is increased, as the empty range increased from 0 degrees at 15 degrees, the maximum principal stress increased from 0.0007MPa to 0.176MPa, the latter is more than 250 times. However, for the minimum principal stress value, each measuring point varies with the change of the void range, but it is less than the minimum principal stress value under the condition of no cavity. It can be seen that the change of the void area has a significant influence on the tensile stress of the lining structure, and the increase of the hole size makes the tensile stress of the lining structure increase obviously.

5 Conclusions

In this paper, the dynamic response model of high-speed railway tunnel under train vibration load is established to study the influence of the different cavities on the back of lining:. The main conclusions are as follows:

(1) With the increase of the cavity size, the influence of the arch, side wall and the junction of side wall and inverted arch is much larger than that of the inverted arch under track and inverted arch center

(2) Compared with the dynamic response of each part of the lining under different working conditions, it can be seen that the change of the back hole area has the greatest influence on the vault. Among them, with the empty range increased from 0 degrees to 15 degrees, the maximum principal stress increased from 0.006MPa to 0.49MPa, which is 81 times more than the former. For the concrete material with the tensile strength far below the compressive strength, the existence of the back cavity seriously deteriorates the stress condition of the lining structure.

(3)This paper only focuses on the back cavity range effect on the dynamic response of tunnel lining structure, the need for the influence of other factors on the tunnel lining structure is further discussed, and the conclusion remains to be the test reliability of follow-up tests and engineering practice.

Acknowledgements

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STUDY ON TEMPERATURE CONTROL MECHANISM OF CONCRETE CRACK UNDER COMPLEX STRESS FIELD IN THE CONSTRUCTION OF LARGE CABLE STAYED BRIDGE TOWER

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Key words : Hydration heat; Mass concrete; Crack control; Temperature control.

Abstract:

At present, the mass concrete structure cracks has become an important task for scholars, but many studies focus on the research of the theory of temperature field and stress field, neglected to study the construction details of the. At present, the mass concrete structure cracks has become an important task for scholars, but many studies focus on the research of the theory of temperature field and stress field, neglected to study the construction details of the. This paper takes Anhui province at the WangDong Bridge, compared with software calculation and monitoring data, according to the structure design are discussed and analyzed the temperature monitoring data. The results are compared with some rules for the calculation software, combined with the distribution and change of temperature field and stress field were analyzed. The influence of large volume concrete hydration heat temperature field and stress field distribution of the main design parameters: cement hydration coefficient, pouring into the mould temperature, pouring section height, transverse segmental pouring, internal structure and layout size of tensile reinforcement, respectively different parameter value of parameter analysis, the temperature field of different design parameters based on the stress distribution and its change trend are analyzed, the lower the water pouring mass concrete thermal impact parameter optimization design suggestions and measures.

1. Introduction

Concrete is the most common construction materials, widely used in industrial and civil buildings. With the development of China's economy and technology, all kinds of engineering construction scale is more and more large, large bridges such as bamboo shoots after a spring rain like emerge, the mass concrete crack problem has become one of the problems faced by the construction of large bridges now, with the production of cement has higher heat release rate than in the past, which exacerbated the temperature crack of concrete in large volume, now has become a problem to be solved.

2. Significance of mass concrete construction optimization

In recent years, with the increasing scale of engineering construction (such as: the largest cap size of Long-tan River Bridge is 22.4X19.4X4m), the bridge engineering has already begun to attach great importance to the hydration heat control of mass concrete. But there is no consensus on the definition of mass concrete. The definition of "standard" in the construction of large volume concrete, the mass concrete is large volume concrete minimum geometry concrete structure entity is not less than lm3, or is expected to be harmful cracks in concrete due to concrete hydration of cementitious material caused by the temperature change and contraction of [1]. The American Concrete Institute (ACI) states: "any mass concrete that is poured in place, the size of which is large, must be required to determine the hydration heat and the consequent volume deformation problem to minimize cracking" [2]. The "technical specification for construction of highway bridges and culverts" (JTGT F50-2011) states: "on-site pouring of the small side size is 1 to 3m and measures must be taken to avoid the temperature of hydration heat caused by more than 25 DEG C concrete mass concrete is called" [3]. Professor Wang Tie-meng in the "crack control of engineering structure" in the definition: "in industrial and civil buildings, underground continuous wall structure, general cast structures and equipment foundation is easy by temperature shrinkage stress caused by fracture

structure, known as the large volume concrete structure" [4], and the American Concrete Institute (ACI similar provisions).

For large volume concrete structure size exceeds a certain range, temperature deformation and temperature must be considered in cement hydration heat accumulation cause stress, should take appropriate measures to reduce the temperature cracks caused by the deformation, to ensure the engineering quality of the concrete structure.

In the early stage of hardening of concrete, due to the hydration of cement, a large amount of heat is released, and the temperature of concrete increases gradually. Due to the good heat dissipation condition, the concrete members of ordinary size have little difference in temperature between the inside and outside of the concrete, and the deformation of the whole member is basically the same. However, because of the large size and low heat dissipation, the internal temperature of the concrete structure is obviously increased. Mass concrete plays a very important role in modern engineering construction, which is widely used in water conservancy and Hydropower Engineering, high-rise building, bridge engineering and other fields. Mass concrete has the following characteristics:

The concrete is a brittle material, the tensile strength is about compressive strength of about 1/10; the tensile deformation is minimal, the ultimate tensile load deformation is only short-term ((0.6 1) x 10-4, equivalent to 6 to 10 DEG C when the temperature decreases the deformation limit; long-term loading of the tensile deformation is only (1.2 x 10-4 2).

The mass concrete structure with large cross-section size, concrete pouring, due to the hydration reaction of the cement produces a lot of heat of hydration, the internal temperature of the concrete structure rises sharply, then the elastic modulus of concrete creep is small, large, caused by hydration heat temperature stress is smaller; but when the temperature decreases the elastic modulus, creep, small, under certain conditions will produce considerable tensile stress.

The reinforced concrete in the mass concrete structure is less, only a small amount of steel bar is arranged near the surface or the hole. Due to the configuration of the steel less, once the tensile stress, it can only rely on the tensile strength of the concrete to bear.

There is a high demand for crack control. Mass concrete is widely used in foundation, dam, etc., in addition to meet the requirements of strength, stiffness, stability and so on, but also has a certain integrity, waterproof and impermeability.

Based on the above characteristics, there is no need for tensile stress or only small tensile stress in mass concrete structures. In the construction process, due to the large temperature difference between the inside and outside of the mass concrete structure, often have a lot of tensile stress, it is difficult to put this kind of internal and external temperature difference caused by the tensile stress in the allowable range. It is for this reason that in large volume concrete structure often appear temperature cracks. Therefore, in the mass concrete construction quality control, the control of concrete cracks has become a key issue.

3. The specific content of the construction optimization

3.1 Project overview

Anhui Province east of Yangtze River Highway Bridge Main Tower for the diamond type tower above the deck is an inverted Y type tower for vertical single tower; the middle tower is split into two out of the tower in the tower beam at the towers at together, under the tower for rigid whole wall section the structure, size, but also transverse prestressed reinforcement design, beam and tower binding site using arc transition, both ends of the beam under the outward cantilever 7.15m in large span cable-stayed bridge first belong to this type of structure design of main tower.

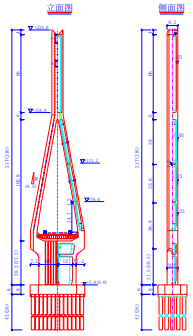

Fig.3.1-1 Tower shape

3.2 Method of the study

The East Yangtze River Highway Bridge Based on the site for the construction of large volume concrete water tower heat of long-term real-time monitoring, and through the numerical finite element model of concrete segment hydration heat state prediction, and the numerical model of model updating in order to improve the accuracy of numerical calculation. Based on the updated numerical model, this paper analyzes the key parameters of the mass concrete pouring of the bridge tower, and puts forward the optimal design and construction method to reduce the hydration heat.

3.3 Pouring mold temperature

According to the engineering background of the project, the average temperature in a year is 16 C, respectively, take concrete pouring into the mold temperature is 18 C, 20 C, 22 C, 24 C, 26 C, 28 C, similarly to the second section as an example, research on different concrete pouring into the mold temperature under the structure of the difference in temperature and temperature stress field changes.

Results of temperature field analysis:

The effect of different pouring temperature on the maximum temperature difference between the inside and outside of the structure is shown in figure 3.3-1:

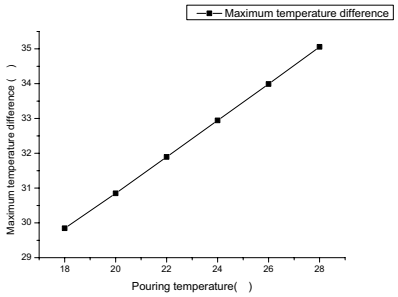

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Fig.3.3-1 The variation of the maximum temperature difference between the inside and outside

The temperature difference between the internal and external temperature of the 2 section of the first section of the 3.3-2 under different pouring temperature is shown in figure:

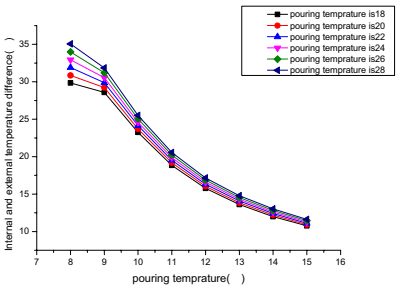

Fig.3.3-2 internal and external temperature difference of the structure with the same pouring temperature

From figure 3.3-1 - 3.3-2 can be seen with increasing pouring into the mould temperature, the temperature difference between internal and external structure also increased, and poured into the effect of temperature on the structure of the mold temperature difference between inside and outside the approximately linear. The second map, different pouring into the structure inside and outside temperature die temperature change with time trend is almost parallel to that the effect of temperature on mold pouring into the inside and outside temperature approximately linear. Pouring into the mould temperature rise of 2 degrees Celsius, the maximum temperature difference between inner and outer structure increased to about 1 DEG C to fourth to 5. After pouring into the day, effect of temperature on the structure temperature difference has become smaller, the molding temperature increases every 2 DEG C, inside and outside when the temperature is only 0.2 degrees Celsius.

The effect of different pouring temperature on the maximum tensile stress is shown in figure 3.3-3:

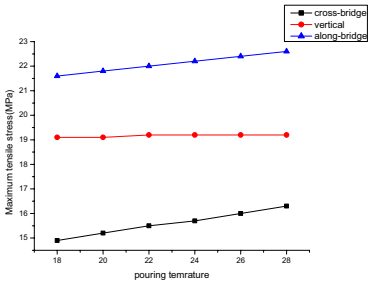

Fig.3.3-3 Variation of maximum tensile stress with temperature

Can be seen from the figure, with increasing pouring into the mould temperature, transverse to the bridge and the bridge to the maximum tensile stress increased, but little growth. Almost no impact pouring into the mould temperature on the structure of the vertical stress, probably because of the heat convection structure with the outside world mainly occurs in the inside and outside side, in the vertical the vertical plane only the top surface in contact with air heat convection heat exchange, so the vertical deformation with the change of mould temperature and pouring into little change. For the transverse direction of the bridge and the tensile stress, because the heat convection and air exchange is relatively large, the temperature gradient is mainly manifested in the these two directions, so the non-uniform deformation is relatively large, caused by the change of stress, therefore increased with pouring into the mould temperature, transverse to the bridge and bridge to the maximum tensile stress than the largest increase in vertical tensile stress increases obviously. However, the total amount of growth is not large, the temperature of the casting mold every 2 degrees, the maximum tensile stress in these two directions increased by 0.2~0.3MPa.

Under different pouring temperature, the maximum tensile stress of the transverse, vertical and longitudinal direction of the bridge (the location of which is described in the fourth chapter does not change obviously) with time, as shown in figure 3.3-4~3.3-6:

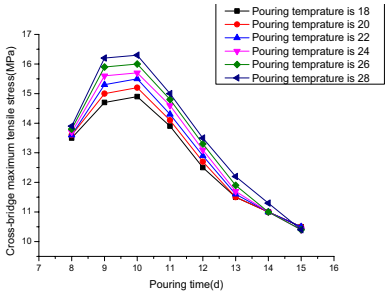

Fig.3.3-4 The variation of the maximum tensile stress of the bridge at different pouring temperatures

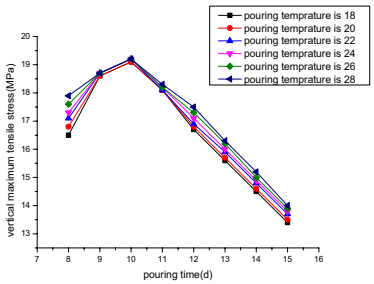

Fig.3.3-5 The variation of vertical maximum tensile stress with time under different pouring temperature

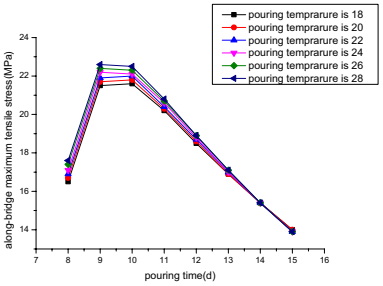

Fig.3.3-6 The variation of the maximum tensile stress along the bridge with time under different pouring temperature

From figure 3.3-4 - 3.3-6 can be seen in different maximum tension pouring into the mould temperature under the direction of three stress versus time curve was approximately parallel to the same with the results of the previous section. Specifically, the cross bridge to the bridge and to the maximum tensile stress, the change of temperature of pouring into the mould a few days before the construction of the maximum tensile stress value is obvious influence to the last two days of different pouring into mold temperature to cross bridge and bridge to the maximum tensile stress is equal to the vertical; the maximum tensile stress, pouring into the mould temperature change occurred on the third day of the maximum tensile stress peak value almost no on this day, before the maximum tensile stress and growth after this day, the maximum tensile stress decreases with the casting process, the temperature changes, the maximum vertical tensile stress changes with time to line nearly parallel to one another. According to the construction experience, the concrete moulding temperature The temperature is 20 DEG C, which is higher than the local average temperature of 16 DEG C at a temperature of 4 DEG C

3.4 Section pouring height

Due to the huge hydration heat generated inside the construction of mass concrete structures can not be lost, the whole structure of partial segmental casting is one of the main measures reduce the hydration heat of concrete by the great temperature gradient and thermal stress.

The following sections were taken pouring height of 3.0m, 3.5m, 4.0m, 4.5m analysis, discuss the influence of different segment pouring height on the structure of difference in temperature and temperature stress of concrete segment. The different height of the structure of the maximum temperature difference effect as shown in figure 3.4-1:

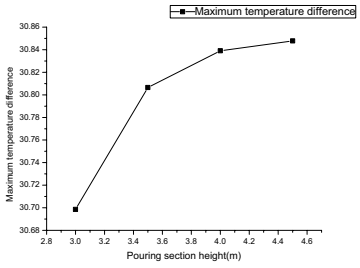

Fig.3.4-1 The variation of the maximum temperature difference with the height of the pouring section

The difference of the temperature difference between the inner and outer sections of the 2 section of the first section of the 3.4-2 under different pouring height is shown in figure:

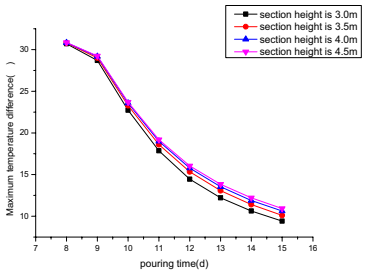

Fig.3.4-2 Variation of temperature difference between internal and external structures with time under different pouring height

See from figure 3.4-1 ~ 3.4-2, pouring each segment height, inside and outside the concrete structure temperature increasing, growth trend from steep to slow. The reason may be that: when the partial segment pouring height increases, the volume of concrete and air convection area increased at the same time, just in height increase in the section, due to the cooling effect of heat convection area and the increase in the volume of concrete hydration heat effect is greater than the

increase brought by the difference in temperature increased rapidly; with the increasing of the height of the concrete volume increase and prolific students hydration heat effect on the cooling effect of heat convection slowly approaching the area increased from therefore, the increase of the temperature difference between inside and outside gradually slow. But overall, the pouring height change on the inside and outside temperature change is not obvious, no increase of pouring height of 0.5m, and the maximum temperature difference increased by 0.1 DEG C About

The effect of different pouring height on the maximum tensile stress is shown in figure 3.4-3:

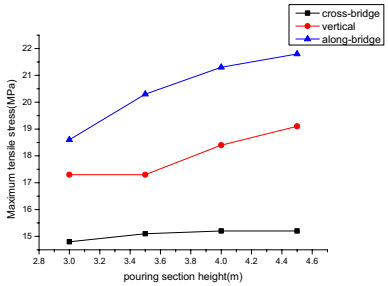

Fig.3.4-3 The maximum tensile stress varies with the height of the pouring section

Under different pouring section height, the horizontal, vertical, the maximum tensile stress to the bridge (the location of the chapter, as shown in the fourth chapter, did not change significantly) with time, as shown in figure 3.4-4:

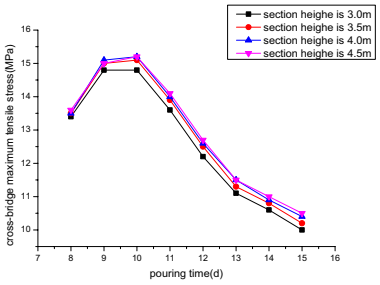

Fig.3.4-4 The variation of the maximum tensile stress of the bridge under different pouring height

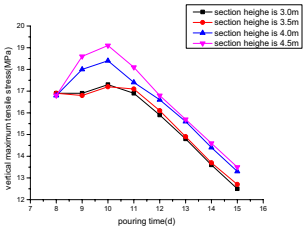

Fig.3.4-5 The variation of vertical maximum tensile stress with time under different section height

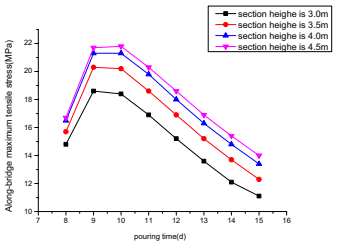

Fig.3.4-6 The variation of the maximum tensile stress along the bridge at different section height

From figure 3.4-4 ~ 3.4-6 can be seen as the segmental pouring height increases, the vertical stress is the main stress and bridge to have large, showing high increase with increasing segment, segment height increased 0.5m, the two directions of the maximum tensile stress can be increased by 1 ~ 2MPa. The transverse stress changes little. The reason may be that with the increase of the height of concrete segment, the structure size of the horizontal direction has not changed, resulting in the differences mainly in the vertical deformation and stress, so the vertical stress is significantly affected. While increasing the vertical deformation and stress by structure between the internal constraints, need to spread to the horizontal direction in the horizontal dimension on the tower structure along the bridge to the size is far less than the size of the transverse direction of the bridge, along the bridge to receive the constraints of large stress increase correspondingly larger, and the transverse deformation That is less affected by stress. Through calculation, according to each part of the concrete structure size and convenience of the actual construction, decided to cast each segment at the height of the tower: the initial height of 50cm, and platform synchronous pouring, first segment tower height 4.0m, second segment and 6 segment height for 4.5m, seventh segments, eighth segments height is 5.6m, the ninth segment height is 4.5m, the tenth segment height is 4.3m.

3.5 Transverse section pouring

The tower as the mass concrete structure, cross bridge substructure to size up to 32m, as a result of reduced concrete hydration heat temperature gradient and temperature stress, the model of transverse block, two block into equal, 8 days before the first cast the first block, 8 days after the pouring of second pieces of in this case the structure of difference in temperature and temperature stress analysis, discuss the influence of the transverse segmental pouring.

Compared with the case of no transverse section pouring, the temperature difference between the inside and outside of the structure with time in the two cases is shown in figure 3.5-1:

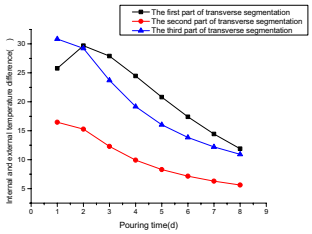

Fig.3.5-1 Comparison of internal and external temperature difference with time

You can see from figure 3.5-1, and transverse segmental pouring compared to the first block inside and outside concrete structure temperature transverse segmental pouring in the first day, were slightly increased, increased value at 2~4 degrees Celsius. But the temperature difference between inside and outside the second pieces of concrete and transverse segment of transverse segment has been reduced greatly compared. In a horizontal section after pouring, pouring the concrete volume to 1/2 of the original, so the final hydration heat generated is greatly reduced. For the first half of the concrete pouring, while heat is reduced, but the heat convection with the outside surface area is reduced, the heat may not be distributed, in addition to the first pouring natural heat is too slow (and hydration coefficient), temperature is not horizontal pouring low outside temperature difference between inside and outside the remaining days will be slightly higher than the overall. For pouring. For second pieces of concrete after pouring, although the heat convection area it is reduced to half of the original, but in the first 8 days after the pouring of concrete pouring, the equivalent of long construction period, so this piece of concrete in exchange with the outside world for heat convection and advance cooling the first piece of concrete and lower concrete and heat conduction through the heat absorption, hydration heat, thus it produces rapidly reduce the temperature difference between inside and outside is greatly reduced, only for integral pouring case about 1/2. The temperature difference compared with no transverse segmental placement, two cases of transverse to the bridge. The vertical direction of the bridge and the maximum tensile stress (such as location described in Chapter fourth, were not significantly changed) changes with time as shown in figure 3.5-2 is shown to 3.5-4:

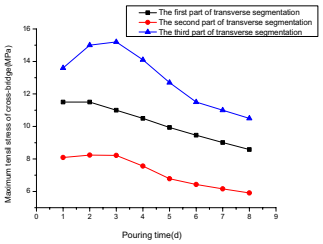

Fig.3.5-2 The comparison of the maximum tensile stress of transverse bridge with time under the condition of transverse section pouring

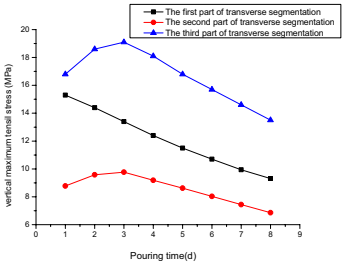

Fig.3.5-3 Comparison of vertical maximum tensile stress with time under the condition of transverse section pouring

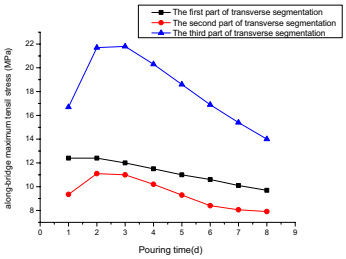

Fig.3.5-4 The comparison of the maximum tensile stress along the longitudinal direction of the bridge under the condition of transverse section casting

See from figure 3.5-2 ~ 3.5-4, horizontal segment pouring, the maximum temperature can be greatly reduced by two block three directions of the stress value of concrete pouring of the first half of the maximum tensile stress can be reduced to about half of the original concrete after pouring, half reduced more. Transverse segmental pouring the temperature gradient is greatly reduced, the structure of inhomogeneous deformation is greatly reduced, and the volume of pouring into the original concrete, between the internal constraints is also greatly reduced, all these reasons make the temperature stress value is greatly reduced. But after transverse section, the construction period the same segment is changed to the vertical 2 times the original, so if the time is not allowed, the stress decreased within the prescribed scope of temperature made from other cooling or reinforcement measures.

3.6 Internal dimensions of the structure

By the above analysis results show that the temperature of the concrete structure node cross bridge direction at both ends and the stress is maximum, as

shown in figure 5-31. The change at both ends of the thickness of H, respectively H 2m, 3m,4m, maximum temperature difference effect, influence of thickness H the internal dimensions of the structure change of structure caused by difference in temperature and temperature stress of different structure as shown in figure 3.6-1~3.6-2:

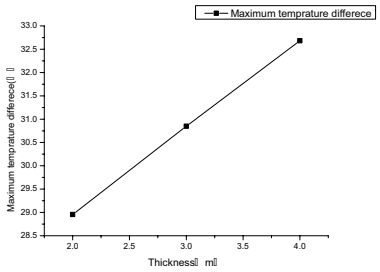

Fig.3.6-1 The variation of the maximum temperature difference with thickness

The difference of the temperature difference between the inner and outer sections of the second section of with different thickness H is shown in figure3.6-2:

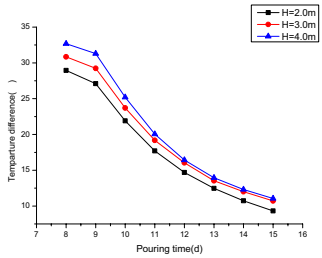

Fig.3.6-2 Variation of temperature difference between inside and outside structures with different thickness

As can be seen from figure 3.6-1~3.6-2, the end of the thickness of H is larger, the greater the temperature difference between inside and outside the H, each 1m increase in temperature can be increased by 2 degrees Celsius. Because of the increase of the thickness of concrete mainly increased heat production, heat dissipation system at the same time and the same time slows down, so the more the thickness difference is greater.

The effect of Thickness with different thickness on the maximum tensile stress is shown in figure 3.6-3:

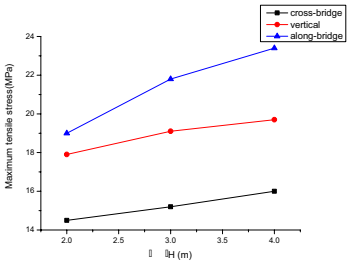

Fig.3.6-3 Variation of maximum tensile stress with thickness

Under the different thickness of the end of the H, the transverse bridge, vertical, the maximum tensile stress along the bridge (the location as described in the fourth chapter, did not change significantly) with time change as shown in figure 3.6-4 ~ 3.6-6:

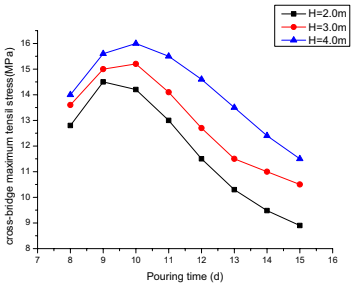

Fig.3.6-4 The variation of the maximum tensile stress of the bridge under different thickness

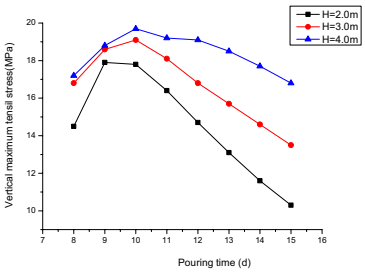

Fig.3.6-5 The variation of vertical maximum tensile stress with time under different thickness

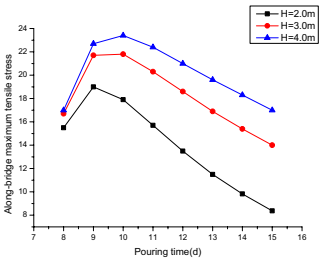

Fig.3.6-6 The variation of the maximum tensile stress along the bridge with time under different thickness

From figure 3.6-4 ~ 3.6-6 can be seen at the end of the thickness of H increases, the greatest impact on the three directions of tensile stress and the temperature difference between inside and outside the influence of the same H, each 1m increase to three, the maximum tensile stress can be increased by 2 4MPa. thickness H increases, temperature difference increases, coupled with increasing size makes concrete internal constraints increasing, so the temperature stress is also increased. Especially for the longitudinal tensile stress due to deformation increases with the thickness of H makes the structure transverse to the bridge, along the bridge to a stronger constraint, also the highest increase of tensile stress to the bridge, when the H from 2m increased to 3m, along the bridge to the maximum tensile stress increased by about 4MPa.

Especially for the internal structure of thickness H of great changes in the 8 section, the internal dimensions of the thickness gradually from 3m transition to 8.8m, the highest temperature and the maximum stress location and location were changed. At this time, the stress changes as described in Chapter fourth, the transverse direction of the bridge and the bridge to the stress increased significantly, increased by about 4MPa, and the maximum stress distribution is more and more centralized, the transverse stress to the value even more than the vertical stress value. And the vertical stress changed little, and the stress distribution tends to be uniform, the deformation of vertical deformation relative to the horizontal direction also became larger. In this structure size segment mutation, should be combined with the actual construction of the actual temperature and stress data, the size of the proper way to ease the transition, reduce the pouring height, the longitudinal and transverse stress to St. Decrease.

Conclusion

This paper first introduces the significance and current situation of the study of mass concrete hydration heat analysis, and then studied the properties of concrete, mass concrete temperature stress calculation and crack control, then to Anhui at the influence factors of hydration heat during the construction of the East Bridge in May to control, and analyzed the measured temperature the data calculated. Finally, the suggestions for construction optimization value by using Ansys software

Because of the large volume concrete structure will be affected by some certain and uncertain factors in the process of construction, and bridge construction in construction stage when more, and some bridge closure when the stress problem, which needs to consider factors is very complex. So, in order to achieve the purpose to reduce the heat of hydration considering the content is various, this article on some factors were studied.

Acknowledgements

This work is supported by the Beijing Key Technology Co- innovation Center of Energy Conservation and Emission Reduction, the Projcet from the National Natural Science Foundation of High-speed Rail Joint fund Projects (U1434211), the National Natural Science Fund Project (51378045), the Projcet from the Beijing Natural Science Foundation (8142012), the Projcet from the Science and Technology Foundation of Transportation Department of Henan Provincial, China.

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STRESS ANALYSIS OF THE KEY PARTS IN THE CANTILEVER CONSTRUCTION PROCESS OF LARGE SCALE CABLE STAYED BRIDGE TOWER

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Keywords: diamond-type bridge tower, mass concrete, tower construction, stress analysis

Abstract

In order to study the stress characteristics and force transfer mechanism of the large cantilever structure caused by shrink inward of the diamond tower lower column, a large cable-stayed bridge diagonal bridge construction control is cited as an example for the engineering background; The large diamond-type cable-stayed bridge tower lower column shrank inward, resulting in a large cantilever structure at both ends of the lower beam, while the tower is bulky, bearing a huge load, stress distribution being uneven, the mechanism of force being complicated, construction difficult; Based on the above basic situation, ansys finite element model was established, concrete being made of solid45 unit, 1/2 horizontal model, the bottom of the tower being consolidated, The column beam being subjected to lateral prestress;,all of the units being combined with mapping and free division; In this paper, the local stress analysis of the lower beam is carried out to calculate the stress condition under the most unfavorable load conditions. Calculations proves that: the transverse stress of the lower beam crosses to meet the construction control requirements; the Maximum longitudinal bridge stress of the tower wall chamber interior is larger, and it is necessary to strengthen the reinforcement on the surface of the middle column and the lower beam to meet the requirements of the bridge shear capacity.

1 Introduction

Since the reform and opening up, China's economy has made brilliant achievements, the development of the economy demand for bridge construction. China has built a large number of large-span cable-stayed bridge, and the cable tower as the cable bridge design of the key, determines the success or failure of the entire bridge. meeting the architectural conditions and functions,at the same time, more able to reflect the bridge beautiful architectural modeling, diamond-type pagoda has been more and more used in high tower, large span, cable system cable bridge construction [1].

Diamond-type tower from the H-type bridge through the fold line evolved, generally by the foundation, bearing platform, tower seat, lower tower column, lower beam, middle tower, upper beam, upper tower and tower top decoration [1]. Stay cable forms a space force, tower, cable between the space enhanceing the three-dimensional sense, elegant and beautiful bridge tower shape, giving a slim beauty[1].The design of the large diamond-shape cable- stayed bridge main tower combined traditional double limbs of the lower tower together to increase the rigidity of the tower itself, resulting in the construction process at Lower column 7.15 m large cantilever concrete structure；the midpoint of the column is 8.8 meters on the lower beam it is also a vertical large cantilever concrete structure，and the anti-bending point is formed in central column. The space cantilever structure formed by two large cantilever constituted the diamond-type bridge tower of the most critical force part, This type of structure is the first case in a large span cable-stayed bridge tower.

2 Project Overview

The cable-stayed bridge is 3.305km long, useing 78 + 228 + 638 + 228 + 78 = 1250m of the PK box composite beam, double tower double-cable semi-floating system cable-stayed bridge [2]. bridge tower uses the Diamond type which upper part is closed, middle part is open, bottom is closed[3]. using C50 concrete, tower top elevation + 224.0m, The east side of the platform elevation is +7.0 m, tower height 217.0m and west side of the platform elevation is +8.0 m, tower height 216.0m; Bridge tower includs upper tower column, middle tower column, lower tower column and lower beam, as shown in Figure 1;The tower is a vertical single column with a height of 76 meters, the middle column is divided into two columns of outwardly inclined columns; middle tower column joins together into a whole at the lower beam of the pagoda, which height is 102.8 meters. The

lower column is the overall wall section, the east side is 38.2 meters high, the west side is 37.2 meters high.

Fig.1 Bridge tower layout

The lower column of the bridge is a large section of the whole wall with large stiffness, the structure is bulky, and the transverse prestressing tendons are also designed. Binding site of the lower beam and the lower column takes arc transition, the lower beam at both ends of the cantilever cantilever 7.15m. From which we can can see , the lower beam (including the lower column) of the bridge tower is a typical large volume prestressed concrete structure, In which there is a significant structural mutation area. The structure, whether in the construction phase or after the completion of its space force are extremely complex, must be a special scientific research to ensure good construction quality and structural safety. So it needs to perform ansys simulation, In which the adverse stress in the construction to be controlled.

3 Stress Analysis of Lower Column

The lower tower than the ordinary diamond-type bridge tower made an innovation, which put ordinary lower limbs together ,as a whole wall structure. In the construction process, the lower beam also cantilevered; Continue to construct the tower column until the first crossbar is set, the middle tower is also a vertical cantilever, which weight will be on the root that is column junction of the lower beam and the middle column with additional bending moment, So the lower beam chamfering and the middle column junction must focus on analysis, to control the tensile stress in the safe range.

3.1 Ansys Model and Computational Load

Calculation uses a common finite element program ansys and concrete uses solid45 units. Horizontal is 1/2 model. The bottom of the tower is consolidated and the column beam is subjected to lateral prestressing. All of the units are combined with mapping and free division. Finite element model of the main tower shown in Fig. 2, lower beam prestress of the main tower shown in Fig. 3.

Fig.2 Bridge tower finite element model

Fig.3 Finite element model of bridge lower beamprestressed beam

3.2 Calculation Parameter and Calculation Load

The concrete in the tower is made of C50 concrete, elastic modulus 3.45e4 Mpa, Poisson's ratio $\nu_c = 0.2$, axial compressive strength standard value $f_{ck} = 32.4$ Mpa ,tensile strength standard value $f_{tk} = 2.65$ Mpa, axial compressive strength design value $f_{cd} = 22.4$ Mpa, tensile strength design value $f_{td} = 1.83$ Mpa, the tensile stress of the prestressed steel beam anchor is 1395MPa. For the maximum axial force and maximum bending moment analysis, the experiment takes the following calculation parameters.

(1) pagoda weight: calculation at 2.6 kN / m3;
(2) support reaction: 8320kN /a;
(3) Prestressed load: consider 80% prestress loss;
(4) The most unfavorable load of the upper tower column bottom passed by the cable: axial force 393342 kN, along the bridge to the shear force 1939 kN, along the bridge to the bending moment 385415 kN • m.

3.3 Calculation and Analysis of Lower Beam

Because of the large axial force transmitted from the upper part and the large cantilever of the bridge itself, the additional bending moment is generated. The model analysis shows that the maximum transverse tensile stress of the lower beam of the bridge tower is about 1.2MPa, In which the middle of the column junction.,the horizontal normal stress distribution is shown in Fig. 4. The normal stress of the 25 units at the straight line length from the midpoint of the lower beam to the lower beam chamfer and the middle column are taken from the model. As shown in Fig. 5.It can be seen from the figure that the stress value of the unit fluctuates more slowly, and the stress change at the junction of the chamfer and the middle column is relatively large, there is a stress mutation, the closer to the lower beam chamfering and the middle of the column junction at the junction of the greater the stress value.

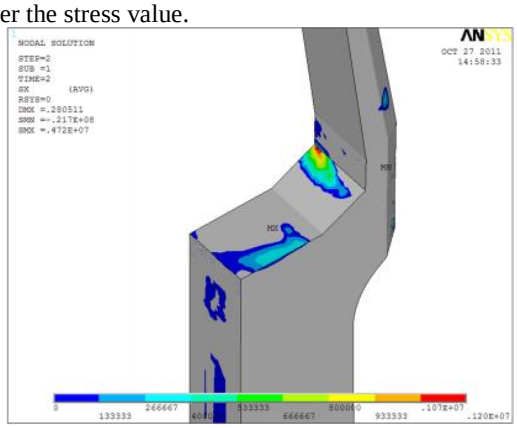

Fig.4 Distribution of transverse normal stress of bridge under bridge tower (Pa)

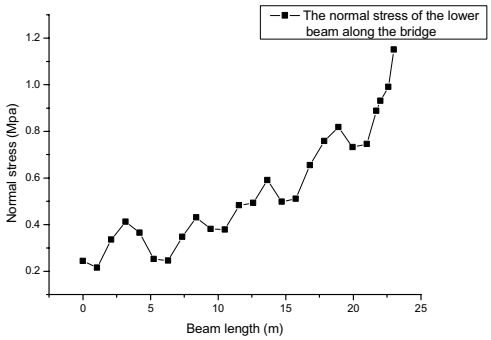

Fig.5 under the beam element is the tension distribution of the situation

Due to the additional bending moment in the upper part of the tower, the local tensile stress of the lower edge of the middle column is about 2.2MPa, which is located at the junction of the chamfer and the middle column. Transverse bridge and vertical stress diffusion are faster; as shown in Fig. 6 below. From the model, the vertical normal stress of the 25 units is obtained from the midpoint of the lower beam to the lower beam chamfering and the middle column. The stress value is shown in Fig. 7; It can be seen from the figure that the stress value at the midpoint of the lower beam is relatively gentle and the value is smaller. However, the closer the stress change of the lower beam chamfer and the middle column is greater;;In a certain range, the stress value of the unit fluctuates greatly, and there are several large mutations, which also shows that the intersection of the chamfer and the middle column is a place where the stress change is large and should be important as the construction monitoring focus point.

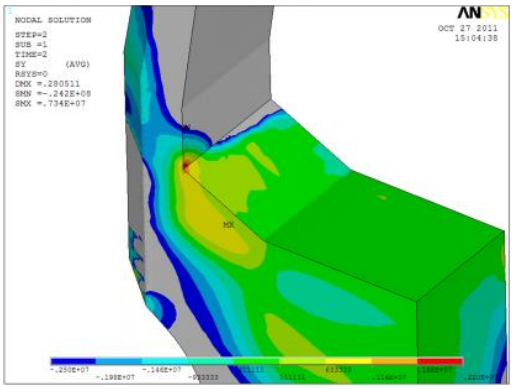

Fig.6 Vertical normal stress distribution of the lower beam of bridge tower (Pa)

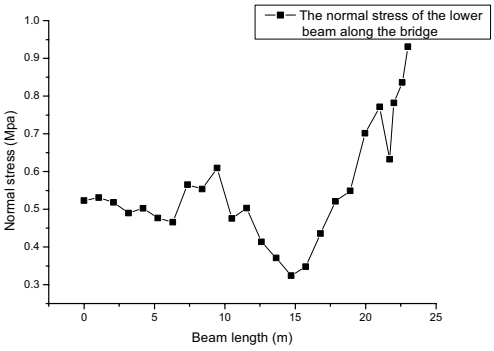

Fig. 7 Vertical stress distribution of the lower beam element

The lateral stress of the bridge tower is relatively small, however, the inner column of the lower tower, due to the upper transmission of the larger axial force, produce a certain tensile stress to side of the bridge; The maximum longitudinal stiffness of the tower wall chamber is about 1.8MPa. The distribution of the normal bridge in the inner column of the lower column is shown in Fig.8. The stress from the midpoint of the inner column of the lower column to the lower edge of the cantilever end is taken from the model. As shown in Fig. 9, the stress at the inner sidewall of the lower column changes greatly, most of the stress value is small and little fluctuation, the stress value of the local existence unit fluctuates greatly, and there is a large stress

mutation, the maximum value is at the cantilever end of the lower beam.

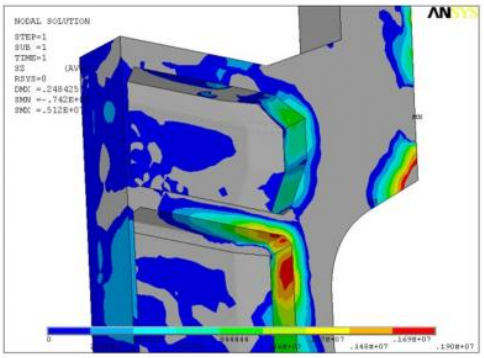

Fig. 8 Bridge lower tower inside the bridge to the normal stress distribution (Pa)

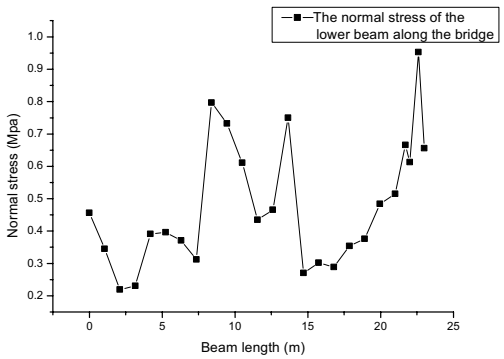

Fig.9 The normal stress distribution of lower tower column inner wall unit along the bridge

4 Summary of research

The maximum transverse bridge of the lower beam is about 1.2MPa, which is located at the junction of the chamfer and the middle column. The transverse normal stress at the outer surface of the lower beam is 0.4MPa and the transverse stress is small. The maximum vertical stress of the lower beam is about 2.2MPa, which is located at the intersection of the chamfer and the middle column. Stress concentration in the corner is more serious, faster stress diffusion; The maximum longitudinal stiffness of the tower wall chamber is about 1.8MPa. The installation of the bridge bar at the junction of the middle column and the lower beam of the tower is to be strengthened, and the steel bar arrangement of the lower beam of the bridge tower needs to meet the shear capacity requirement of the cross section.

Acknowledgements

This work is supported by the Beijing Key Technology Co- innovation Center of Energy Conservation and Emission Reduction, the Projcet from the National Natural Science Foundation of High-speed Rail Joint fund Projects (U1434211), the National Natural Science Fund Project (51378045), the Projcet from the Beijing Natural Science Foundation (8142012), the Projcet from the Science and Technology Foundation of Transportation Department of Henan Provincial, China.

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STUDY ON CONSTRUCTION AND CONTROL OF LARGE VOLUME CONCRETE IN SUPER-LARGE CANTILEVER CONCRETE PYLON

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Keywords: large volume casting, temperature monitoring, crack control.

Abstract

The mass concrete structure usually requires no tensile stress or a small tensile stress, but in the construction process often due to changes in temperature have great tensile stress in mass concrete, when the tensile stress is greater than the tensile strength of concrete will appear crack. A large cable-stayed bridge the main tower of large volume concrete construction as an example, the key to the tower beam under the cantilever section as the research object, analyze the numerical simulation of heat to the construction scheme and the corresponding parameters of the construction phase of the large volume concrete structure temperature changes, and compared with the actual pouring section of the monitoring data, summed up the law of temperature changes, at the same time of hydration the thermal parameters are optimized.

1 Introduction

The mass concrete structure design, usually require no tensile stress or a small tensile stress, the weight and water pressure of the external load, this is not difficult. But in the construction process often due to changes in temperature have great tensile stress in mass concrete, the the temperature change caused by the tensile stress limit is rather difficult in the allowed range. Especially when the large size concrete structure (mass concrete structure), the hydration heat can be quickly dissipated, the internal temperature, then at ambient temperature under the influence of decreased body temperature varies with the block time will have a greater even unacceptable temperature stress in block constraint conditions, once the tensile stress is greater than the tensile strength of the concrete will crack.

2 Engineering background

The main tower cable-stayed bridge bridge is a large diamond type tower, tower 217m, above the deck is an inverted Y type tower for vertical single tower; the middle tower is split into two out of the tower in the tower beam at the towers at together, under the tower for large stiffness of the whole body the wall section structure, large volume, but also transverse prestressed reinforcement design, beam and tower binding sites take the arc transition in the construction process, appear at both ends of the beam and the tower in 7.15m outward cantilever bending point on the lower beam of 8.8 meters, is also a vertical cantilever in large span cable-stayed the first bridge belong to this type of structure design of main tower.

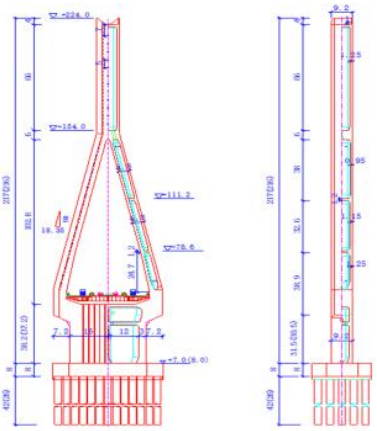


Fig.1 Schematic diagram of main tower

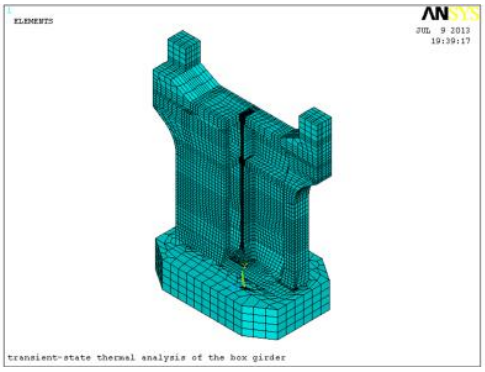
3 Establish finite element model

3.1 Numerical analysis parameters

3.1.1 Model parameters

Using the general finite element program ANSYS to analyze the hydration heat, the first thermal analysis of main pier structure, then the thermal analysis of the temperature field is obtained as a body load applied to the structural elements on thermal structure coupled field analysis, finally structure. The stress distribution analysis using solid70 three-dimensional thermal analysis unit,

selection the solid45 unit of the structure, the model is divided into 27105 elements and 33846 nodes.



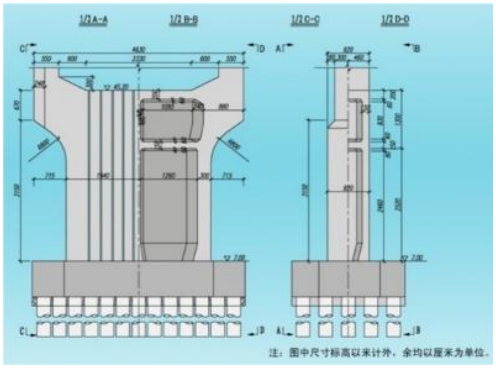


Fig.2 Analysis object model

3.1.2 Hydration heat analysis parameters

1) hydration heat generation rate

The hydration heat of cement is not released completely at one time, but gradually released with time.

$$Q_{(t)} = Q_0(1-e^{-mt}) \qquad (1)$$

In the formula, $Q_{(t)}$ is the cumulative hydration heat of t, kJ/kg;Q_0 is the final hydration heat, kJ/kg;t was age; m was hydration coefficient.

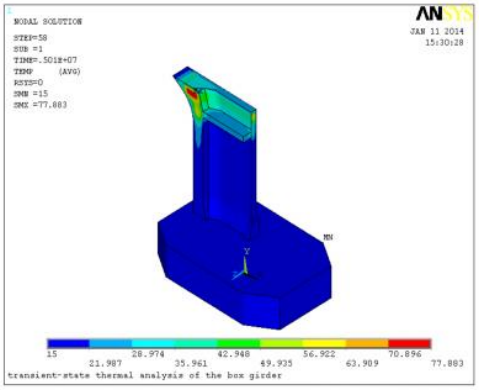
2) the elastic modulus of concrete

The elastic modulus of concrete is changed with the change of pouring time, and the exponential formula $E_t=E_0e^{-0.6/t}$ is obtained, and C50 concrete thermal conductivity 2.94 (W/m. C), the specific heat capacity is 960 (J/Kg. C), thermal expansion coefficient of 0.00001.

ANSYS analysis of the temperature field the local average temperature of 16 degrees Celsius, the temperature of the concrete into the mold temperature of 20 degrees Celsius

3.1.3 Hydration heat analysis parameters

According to the structural characteristics of the lower tower column is divided into 11 sections. The tower construction initial height of 50cm, and the cap. The first



segment synchronous pouring tower height 4.0m, second segment and 6 segment height were 4.5m, seventh section, eighth section height is 5.6m, the ninth quarter is 4.5m, the tenth section height 4.3m.

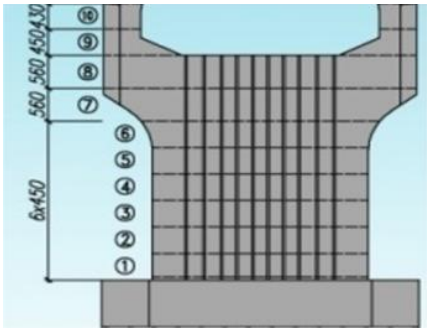


Fig.2 Sketch of construction section

3.2 Numerical simulation results

In the construction, the lower beam (including tower), beam and tower column junction mutations in distinct regional structure, and the cantilever beam under outward, so the calculation results mainly from numerical calculation results when the cantilever pouring part is analyze the construction of seventh and 8 segments of the results, some simulation results as below show:

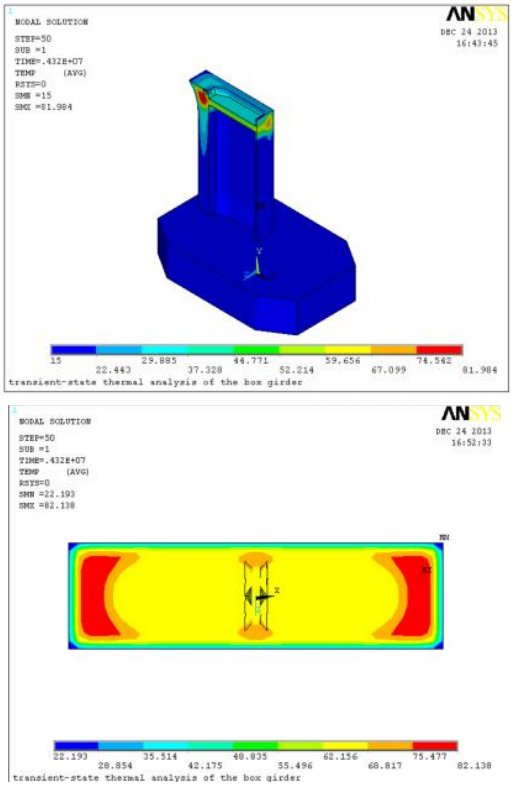


Fig.3 7 section pouring temperature distribution of third days

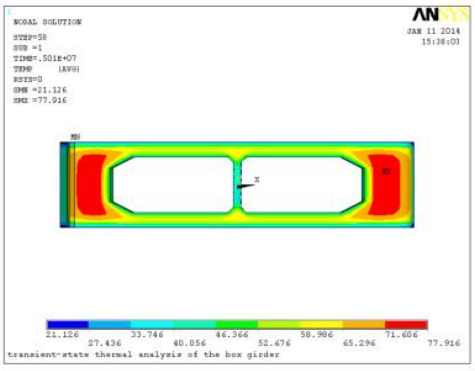


Fig.4 8 section pouring temperature distribution of third days

The calculation shows that the temperature field of casting seventh segments and eighth segments, the maximum temperature occurs in the area of cantilever structure, and concentrated in the middle part of the cantilever section.7 section and 8 section of the concrete amount is relatively large, and the internal heat transfer coefficient is low, the cooling cycle will be longer stress. The stress distribution and temperature field distribution is similar.

4 Field temperature monitoring

Select the pouring segment in 7 and 8 temperature monitoring data were analyzed, the temperature sensor in each layer of concrete is close to the center line of key arrangement, the highest temperature distribution in this area can represent the entire concrete section; at the same time to consider the segment under the surface of the layout of the temperature - stress sensor, detection of new and old concrete interface temperature should be. The stress distribution of the middle layer. The focus plane layout, due to the temperature gradient region therefore larger surface, the arrangement of measuring points is dense, and the center of regional temperature gradient is small, so the arrangement of measuring points is reduced.

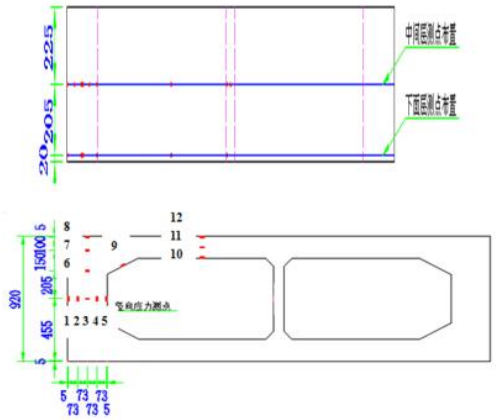


Fig.5 Layout of measuring points

According to the actual construction, the seventh pouring time for November 2, 2013 9:15~ 24:00 in November 2nd, that is, the time to enter the model. Take the middle surface layer temperature gradient of M3 and M5 two points of the measured temperature variation diagram, as shown in Figure.

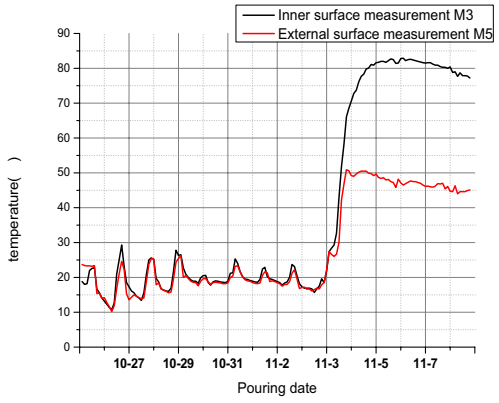


Fig.6 Internal and external surface temperature variation

The eighth section pouring time is from November 19, 2013 to 16:30 4:20, that is, the time to enter the model. Take the temperature gradient in the middle layer of the M1 and M4 two points of the measured temperature change law, as shown in Figure

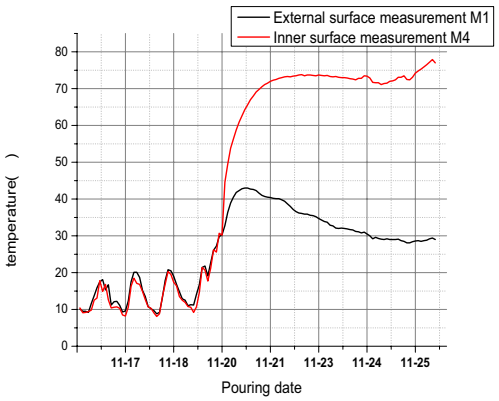


Fig.7 Internal and external surface temperature variation

Compared with the measured results, the following conclusions are drawn:

1) the concrete into the mold temperature: according to the results of the analysis, the section of the concrete into the mold temperature is about 20 degrees, section of the concrete into the mold temperature is about 19.7 degrees in.

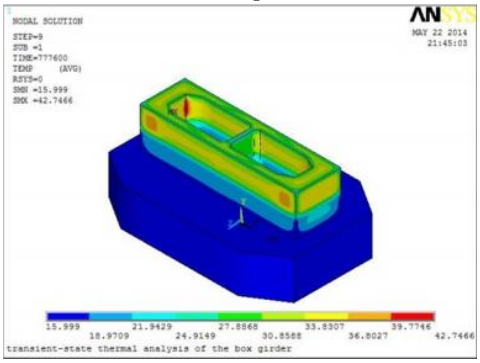
2) the hydration heat growth law: from the 7 section temperature variation, in November 2nd after the completion of concrete pouring, the heat of hydration of cement, concrete and internal surface temperature increased rapidly, rising nearly 3~4 days, middle level measuring points to reach the temperature value in November 6th, then decreased slowly by day. Section 8 temperature variation, in November 20th after the completion of concrete pouring, the heat of hydration of cement, concrete and internal surface temperature increased rapidly, the internal point continues to heat up nearly 2~3 days, middle level measuring points to reach the temperature in November 22nd, and then to maintain the temperature until the last moment. The external measurement points quickly after heating, in the temperature reached the peak, then due to the environment temperature is cold, the external temperature is decreased rapidly, then stabilized at about 30 degrees, is not consistent with the internal rules of internal measurement points, the measuring points in pouring In sixth days the temperature suddenly rises to a higher temperature.

3) the highest value of temperature: the middle level of M3, M4 hydration heat temperature is more intense, the highest temperature of M3 is up to 82.9 degrees, the highest temperature of M4 is up to 77.9 degrees, which is basically the same with the finite element calculation

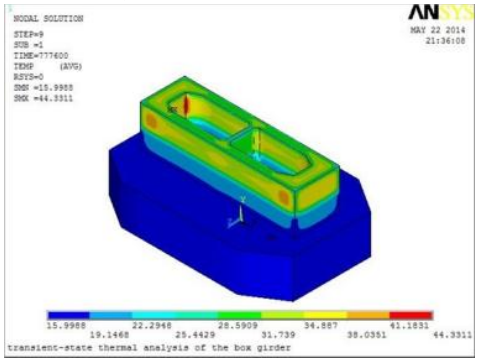
4) section 7: temperature difference between inside and outside the middle level inside and outside surface of maximum temperature is 43.4 degrees (M3-M5), and the finite element calculation of consistent (calculated 49.4 degrees); section 8 intermediate level inside and outside surface of maximum temperature is 43.5 degrees (M4-M1), and the finite element calculation accords (theoretical calculation 47.9 degrees).

5 Optimizing hydration heat coefficient

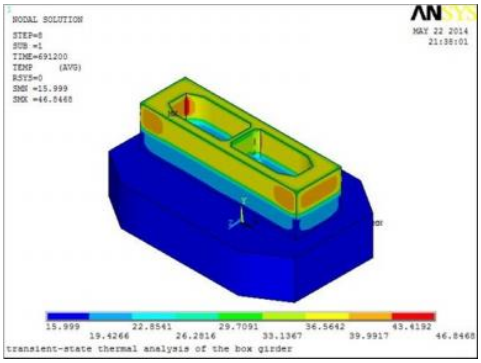
The hydration heat of cement and is not a full release, but with the gradual release of the time, the cement hydration coefficient directly affects the process of concrete hydration heat release. The hydration coefficient with the cement varieties, structure change and temperature condition. In order to pouring second section as an example, respectively, of water value for 0.8,1.0,1.2,1.4 calculation, analysis results are shown in Figure 8 the calculation of temperature field distribution



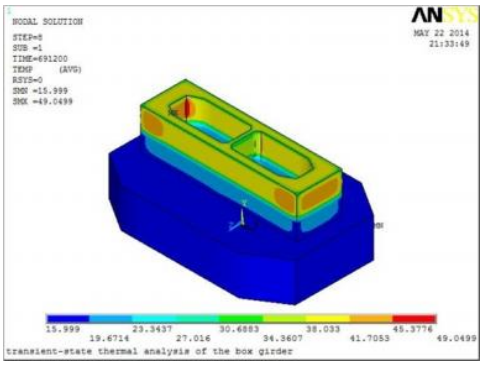
hydration coefficient m=0.8



hydration coefficient m=1.0



hydration coefficient m=1.2



hydration coefficient m=1.4

Fig.8 Temperature field distribution of different hydration coefficient

According to the calculation results the relationship of hydration coefficient and water concrete structure in the process of heat between the maximum temperature difference, as shown in Figure 9, and hydration coefficient under the different changes of temperature difference between inside and outside the second segments with time as shown in Figure 10.

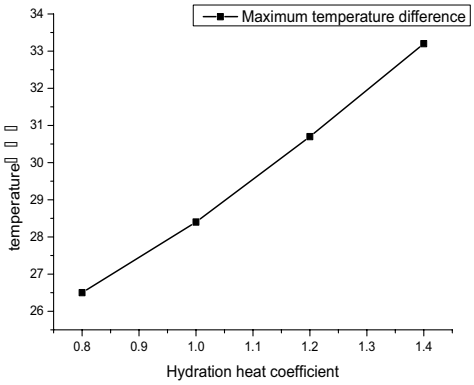


Fig.9 Variation of maximum temperature difference with hydration coefficient

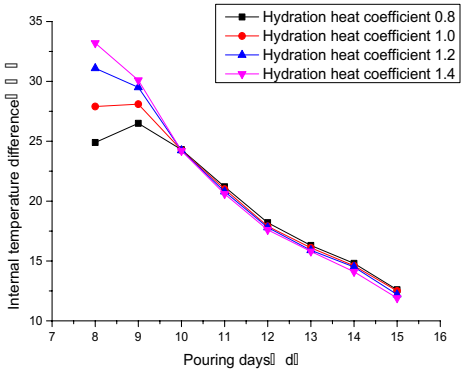
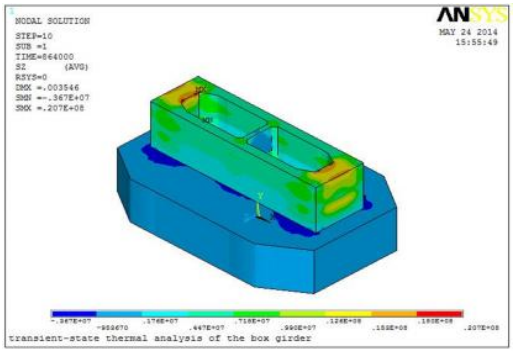


Fig.10 Variation of internal and external temperature with time under different hydration coefficient

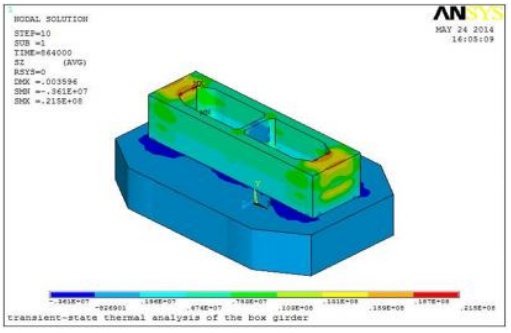
As can be seen from the above two diagrams, the greater the hydration coefficient, the greater the temperature difference between the inside and outside of the section, the greater the hydration coefficient, the faster the temperature difference between the inner and outer sections decreases with time

Analysis results of coupling stress field:

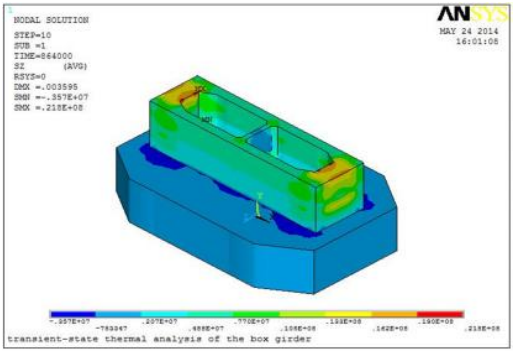
In the corresponding temperature field along the bridge, the hydration coefficient of 2 different segments to the maximum tensile stress (produced in the upper surface of the inner edge of the structure) as shown in Figure 11. The stress field under the different values of parameters are given only along the bridge to the maximum tensile stress.



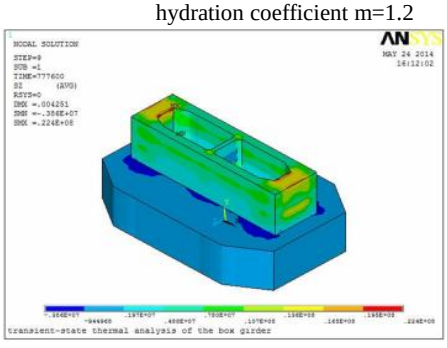
hydration coefficient m=0.8



hydration coefficient m=1.0



hydration coefficient m=1.2



hydration coefficient m=1.4

Fig.11 Maximum tensile stress under different hydration coefficient

According to the calculation results, the variation law of the maximum tensile stress peak with the hydration coefficient is obtained in the second section of the transverse, vertical and longitudinal direction, as shown in figure 12:

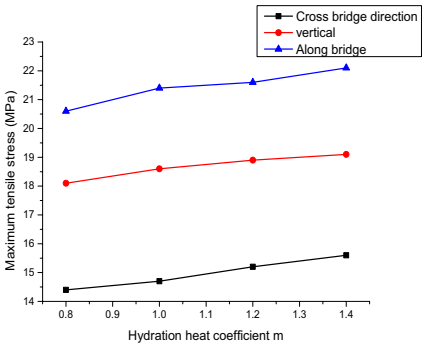
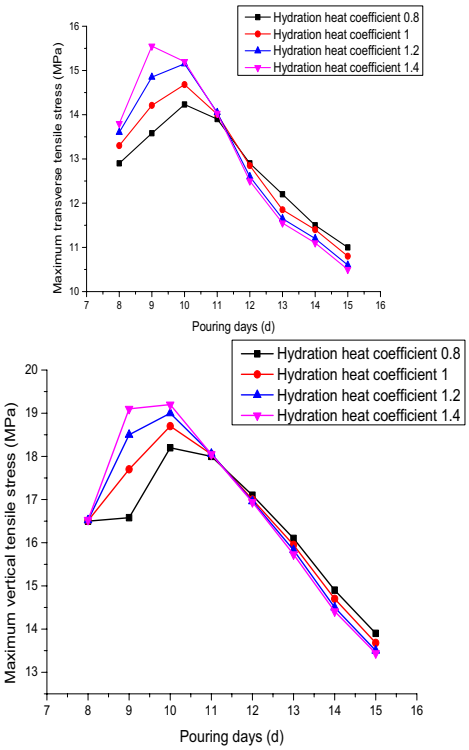


Fig.12 Variation of maximum tensile stress with hydration coefficient

Under different hydration coefficient, the maximum tensile stress of transverse, vertical and longitudinal directions with time is shown in Figure 13.



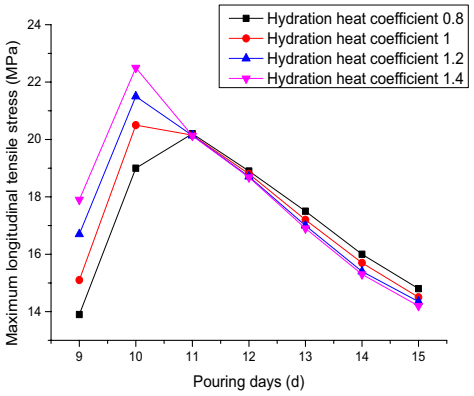


Fig.13 Variation of maximum tensile stress with time under different hydration coefficient

From 13 it can be seen that the effect of hydration coefficient on the temperature stress and the temperature difference between inside and outside the influence of similar hydration coefficient increases, resulting in segment the maximum tensile stress is larger; the hydration coefficient is larger, the three directions of the pulling stress rate decreases with time more quickly, on the other hand, pull should the rate of force decreases more slowly, especially when m=1.4, a day early temperature stress reaches a maximum value, then began to decrease.

6 Conclusion

The construction background of a large cable-stayed bridge based on the seventh and eighth tower under construction segment temperature control of mass concrete construction process, combined with the finite element software ANSYS numerical simulation and analysis of influence of hydration heat numerical obvious hydration heat coefficient in parameter optimization, draw the following conclusions:

1) by finite element model analysis found that seventh and eighth section concrete pouring after third days, the maximum temperature appeared in the middle part of the cantilever beam under the position, due to the large volume of concrete, part of the internal heat dissipation coefficient is low, the heat cycle is longer, and the high temperature reached 82.9 and 77.9 degrees, the maximum temperature difference respectively 49.4 and 47.9 degrees, so the lower beam cantilever area easy to crack. The results can provide the basis for the actual temperature monitoring, optimizing the layout of measuring points.

2) the data measured seventh and eighth concrete pouring segment growth of water heat, concrete pouring has been completed, due to the heat of hydration of cement, concrete and internal surface temperature increased rapidly, rising nearly 3~4 days, the temperature reached the value of measuring points of middle level, then decreased slowly. The maximum temperature measured and the maximum temperature difference and finite element analysis are basically the same, the accuracy of the model is proved, and the finite element

analysis can be more accurate parameters according to the measured data, can be used to optimize the model, making the analysis more accurate results.

3) To optimize the discovery through the hydration heat parameter, hydration coefficient should not be too large or too small, take the middle value of about 0.9~1.2, which can control the maximum temperature stress, and can make the temperature and stress with time decreases not too slow.

Acknowledgements

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RESEARCH ON KEY TECHNOLOGY OF SUPER-LARGE CANTILEVER CONCRETE TOWER CONSTRUCTION OF LARGE CABLE STAYED BRIDGE

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Keywords: large cantilever construction, hydraulic climbing formwork, accurate measurement

Abstract

The large cable-stayed bridge main tower for the diamond type tower above the deck is an inverted Y type tower for vertical single tower; the middle tower is split into two out of the tower in the tower beam at the towers at together, under the tower for rigid whole wall section, under the beam and the lower pylon binding site adopt arc transition, under the tower tower at 7.15 meters and the cantilever bending point on the lower beam 8.8 meters of vertical cantilever two big cantilever force of the key position. Under the tower pylon by formwork construction, on the pylon by hydraulic climbing formwork construction method, with accurate measurement and control technology, the construction of large cantilever concrete pylon.

1 Introduction

The large cable-stayed bridge using PK composite box girder 1250m. High beam 3.5m. main tower with diamond type tower, using C50 concrete, the main tower 217m. tower for vertical single tower, the middle tower is split into two out of the tower in the tower beam at the towers at together, under the the whole wall section. The main pier fortification ship collision force is large, the main tower of up to 4800t, to meet the design force of ship collision, the traditional tower tower under the double limb together increase the stiffness of the tower, resulting in the construction process under the tower at 7.15 meters in the tower bending cantilever; in the lower beam of 8.8 meters, is also a vertical cantilever pylon, as in the inflection point, the main tower structure as shown in figure 1.

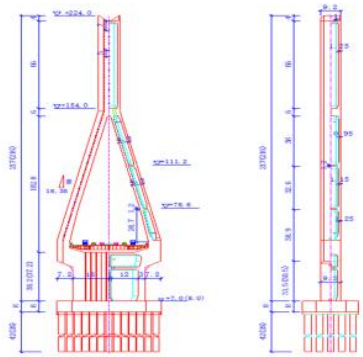


Fig.1 Schematic diagram of main tower

2 Bridge tower construction scheme

The pylon by step pouring construction method. Under the pylon by bracket turnover form construction, beam and tower crown with support formwork construction, the remaining segments are used hydraulic climbing formwork construction technology. The tower is divided into 51 sections (not including the 80 cm initial pouring together with the pile cap construction, under the tower section) the 12 section, in the tower sections of 22 segments, 17 segments on the tower section.

3 Construction Emphasis

1) The appearance quality of concrete tower directly reflects the overall level of construction of the bridge, is also an important factor affecting the durability of the structure. Therefore, the appearance quality of concrete tower, tower construction is the key.

2) The tower beam under the force of special structure, for the construction of large volume concrete, and prestressed pipe is numerous, is the key of construction control.

3) The high tower construction crane, elevator layout and running, climbing safety hydraulic climbing formwork construction system, is a key and difficult point of construction.

4) The tower construction precision control requirements are very high, the construction control to put forward the high requirements.

4 Key construction technology

4.1 Preview Under the tower construction technology

4.1.1 Setting the stiffness matrix

In order to ensure accurate positioning the location of steel tower construction, but also to prevent at high altitude, tilted steel pylon deformation, internal need to set the skeleton. According to the structural characteristics of tower and beam, reinforced the construction of stiff skeleton design for plane space truss column and the horizontal direction of the temporary connection part two truss. When installing, using two line hammer two vertical section of frame column the precise positioning, and then draw the main reinforcement skeleton in the above position, assist in positioning binding main bars.

4.1.2 Temperature control technology of mass concrete pouring of lower beam

In mass concrete construction, often by laying cooling water to cool down, according to the beam structure of this project under the bearing characteristic and prestressed pipe dense, such as the layout of cooling pipe, will further reduce the cross-sectional area, the structure stress and prestressing effect. Through research on the beam structure, prestressed pipe dense can be used as a cooling pipe.

In the process of construction, the main use of water cooling pipe. The prestressed pipe pressure for plastic corrugated pipe, corrugated pipe pressure water to the air compressor, accelerate the bellows internal air circulation, bring out the concrete internal temperature, reaches a temperature effect. The beam of prestressed pipe layout is as follows.

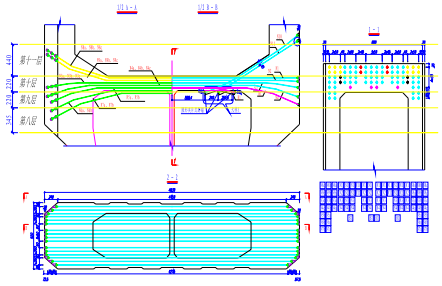


Fig.2 Prestressed beam layout of the lower pylon

The air compressor pipe access and ventilation pipe, adjust the water content, the water enters the air compressor, high pressure air and water in the mix, the water in the high wind pressure, oscillating granular form a fog effect, then the air containing mist is pressed into the corrugated pipe, the pipe heat by water absorption at the same time through the fog, wind speed high pressure air circulation pipeline, temperature control is achieved.

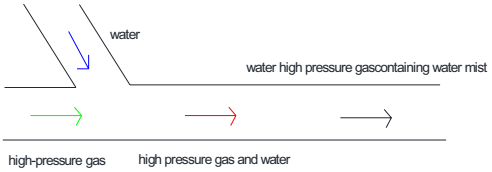


Fig.3 Schematic diagram of working principle

MIDAS FEA is used to analyze the whole model of the solid element, and the influence of the discontinuous and continuous construction of each layer is fully considered.

Fig.4 Simulation model

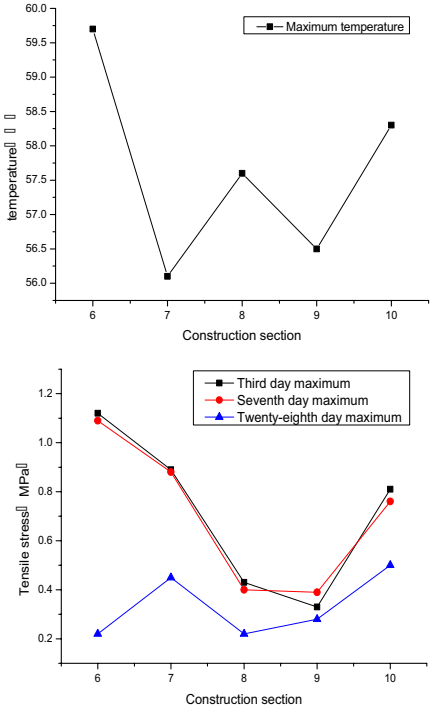


Fig.5 Model results

The model calculates the maximum temperature generally occurs third or fourth days after concrete pouring, under the tower structure mutation at low temperature, the main tower pier under the concrete internal maximum temperature calculation value of 59.7 degrees.

Through the water after the corrugated pipe wall temperature can be reduced, and close to the outside air temperature, temperature control of mass concrete can meet the requirements, but in the water process, to fully consider the size of the pressure of the air compressor, ensure the ventilation effect of high pressure steam. The scene of implementation, field measurement, temperature peak time is about third days after pouring, the highest temperature is 52.3 degrees centigrade, the comparison of model, found the temperature control effect is obvious.

4.1.3 Prestressed construction of lower beam

The lower beam is arranged in the construction of prestressed beam, pre buried corrugated pipe, concrete pouring has been completed through the beam, tension, grouting, anchor seal construction. The tower beam of prestressed steel beam is longer, and the root number, take the current installed in the prestressed pipe to wear steel strands in concrete strength and elastic. To meet the requirements of die after prestressing. The prestressed steel strand tensioning order should be carried out in accordance with the tension after the first intermediate upper and lower left are symmetrical, 90%. tower prestressed prestressed concrete strength is not less than the design strength of the grouted by vacuum process, improve the plumpness, reduce the influence of bubbles.

4.1.4 Prestressed construction of lower beam

The tower and beam construction under the tower formwork construction method. In the template before installation, accurate measurement lofting plane position of pylon by total station polar coordinate method (including axis and edges) and tower bottom elevation, measured skeleton plane position. To be reinforced, template installation is completed, check again check with level template, the top surface elevation, using adjustable rod precise adjustment template to the correct position fixed. After each template, according to the design position accurately calculate the top opening of the 3D coordinates, using the same method of repeated check adjustment, the plane position, top elevation and verticality have reached the design requirements. After the baseline level setting out vertical distance measurement check the measured elevation, do double control checks.

4.2 The construction technology of middle pylon

4.2.1 Structure of hydraulic climbing formwork system

The main tower pier, pylon design of formwork system for hydraulic self climbing form, using the self climbing formwork system of LG-100 hydraulic. In the upper standard pouring height of 4.5 meters, the tower, 168 meters high, a total of 38 times the work cycle, the shrinkage section of tower body by cutting template to realize the combination of steel and wood template, template, climbing the lower frame body share 28 pieces of hydraulic, 30 specimens of upper body. In order to reduce the cost of the template, hydraulic climbing mold body adopts separate frame, hydraulic climbing formwork body is assembled into the advantages of standard construction can rely on the power system's climbing tower crane to hoist do not take, can greatly accelerate the construction progress, and a frame body with a perfected security maintenance system, ensure the safety of the construction.

Hydraulic self climbing formwork is the power source of the hydraulic lifting system itself, the hydraulic lifting system comprises a hydraulic cylinder and the upper and lower reversing box, commutating box can control the lifting guide rail and the lifting frame body, the hydraulic system can make up the formation of mutual support body and the guide template, so that the hydraulic self climbing form steadily upward climb, hydraulic self climbing formwork without other lifting equipment in the construction process, convenient operation, climbing speed, high safety coefficient. The hydraulic self climbing form is mainly divided into the template system, support system, embedded system, the four part of the hydraulic system.

4.2.2 The middle column of the active cross brace construction

In the tower to tilt to the inside, with the increase of construction period, need to set the active level of support, the unbalanced force generated by the balance tilted tower. The hydraulic climbing formwork in the construction of the tower, the first horizontal braces applied thrust force 334.6t, second horizontal braces applied thrust force 297.2t. third horizontal braces applying thrust force 269.4t. fourth horizontal braces applied thrust force 305.3t.

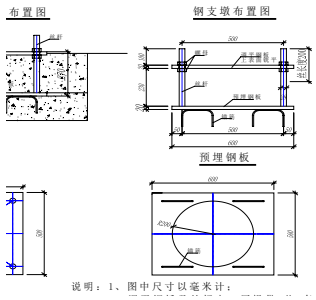


Fig.6 In the schematic diagram of horizontal braces

According to the tower structure and construction characteristics of the active cross brace of abrupt structure stress monitoring, stress measurement points of the section are: under the tower section (C-C), the upper side section and the main tower closure cap the maximum cantilever position (A-A, B-B). Off the mask position according to the monitoring calculation the results and practical engineering to determine the comprehensive consideration.

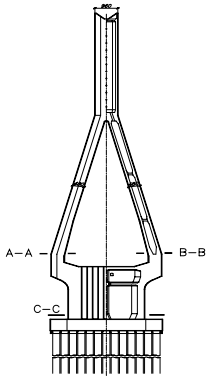
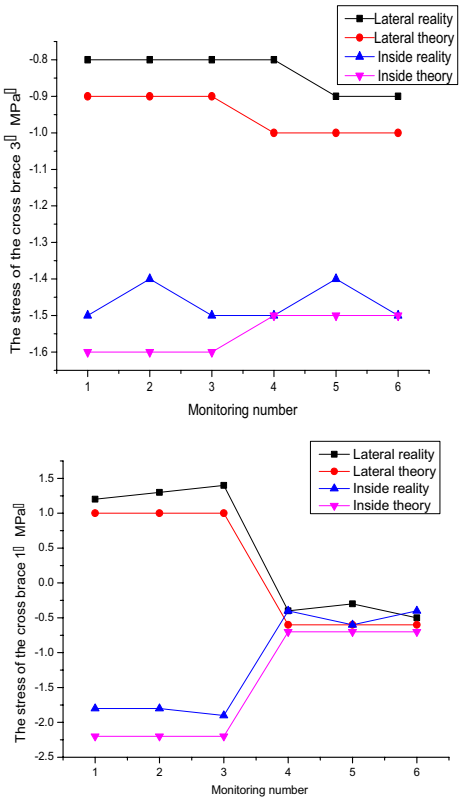


Fig.7 The sketch map of stress testing section

Using MIDAS finite element program of calculation and analysis, to establish the main tower model, the main tower is divided into 280 units and 310 nodes, in the process of construction, in the layout of the 4 tower root temporary active cross brace, the calculation model is shown in figure:

Before and after the four horizontal support, the stress of the measuring points was monitored, and the measured values were compared with the results of finite element analysis. The results of A-A section were compared, as shown in the figure:



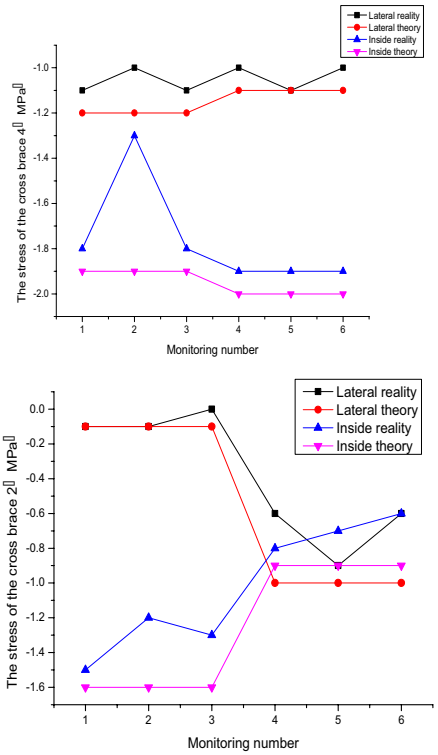


Fig.8 Comparison of internal and external stress measurement of section A-A

From the section above key construction stage on the tower stress test, cross brace 1,2,3 pushing measured before and after A-A and the measured stress is basically consistent with the theoretical value, the 4 cross brace pushing before and after A-A beta measured one point mutation value, but the value is smaller, less than the theoretical value. On the whole the tower, the stress theory and test are basically consistent, and consistent with the theoretical calculation, the structure is in the controllable state. All measuring points of the measured stresses are in good agreement.

5 Conclusion

The large cable stayed bridge pylon by formwork construction, on the pylon by hydraulic climbing formwork construction method.

1) Under the tower beam under the stress of special structure, for the construction of large volume concrete, and prestressed pipe is numerous, is the key of construction control. During the construction of cantilever beam under section, the tower section by molding method construction technology of construction, to ensure the safety of construction process is the key and difficult control from construction. Ordinary type brick tower, the tower is under the wall section, the section size is larger, belongs to the large volume of concrete pouring, pouring should be stratified and relatively reasonable, avoid disposable had hydration heat caused by internal and external temperature difference. The stress should be strictly monitoring the temperature of casting process,

prepared in advance and corresponding control measures and to avoid cracks.

2) In the tower to tilt to the inside of the pylon by hydraulic climbing formwork construction method in the construction of the tower in the gravity and the climbing frame under the action of additional moment will be produced and the unbalanced force, the inward tilt alignment will produce deviation, must set up the active cross brace. About the position and thrust force reasonable active cross brace to the tower tower after the final stress state. The simulation analysis can determine the number and location of the active cross brace, and can be determined for each active cross brace thrust force.

Acknowledgements

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STUDY ON SEISMIC DYNAMIC RESPONSE OF SUPER-LARGE CANTILEVER CONCRETE PYLON STRUCTURE OF LARGE CABLE-STAYED BRIDGE

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Keywords: large cantilever,masonry type tower, response spectrum analysis

Abstract

The large cable-stayed bridge main tower for diamond type tower, under the two ends of the beam are outward cantilever 7.15m at the same time, the tower bending point on the lower beam of 8.8 meters, is also a vertical cantilever pylon, as in the inflection point, the spatial cantilever structure of two large cantilever has become the bridge Tower of the diamond type the key part, using the general finite element program ANSYS to calculate the most disadvantageous load effect under the tower beam, beam and tower with the key by stress parts. Considering seismic design intensity, combined with seismic response spectrum analysis and single point excitation time history analysis of tower beam response behavior the dynamic force part of beam and tower column section combined with the key comparison, under constant loads tower stress, analysis of structural dynamic response under earthquake.

1 Introduction

The earthquake is a natural disaster afflicting humans, due to its randomness and sudden earthquake, it is difficult to predict, in essence is a kind of natural phenomenon, the unpredictability of human social life brought great trouble, disastrous consequences caused by the earthquake, often shocking. But our country increasingly rely on traffic the line is bigger, the construction and development of large span bridge is very fast, China is one of countries with frequent earthquakes in the world, more than 40% of the land is located in VII and above the earthquake zone, so as the key structure of large span bridge cable tower, explore its dynamic response characteristics is very necessary, especially in recent years, large-scale construction of a cable-stayed bridge, most frequently used in concrete diamond type tower structure, with complex structure stress, construction difficult, dynamic characteristics of the tower can provide reference for similar engineering Test.

2 Engineering background

The bridge main span of "78m+228 m +638m +228 m +78 m", while the 638 m span is the combination of domestic and foreign maximum span of cable-stayed bridge. The main tower with diamond type tower, the main pier fortification ship collision force is large, the south main tower reaches 4800t, in order to meet the requirements of the vessel collision force protection. The traditional tower tower under the double leg together increase the stiffness of the tower, resulting in the construction process under the tower beam at 7.15 meters in the tower and the cantilever bending point on the lower beam of 8.8 meters, is also a vertical cantilever pylon, as in the inflection point. The high tower 217m tower high 70m in upper joint beam thickness is 6m, in the tower beam cross bridge high 102.8m. to 46.3m wide, along the bridge to the length of 9.2m, as shown in Figure 1.

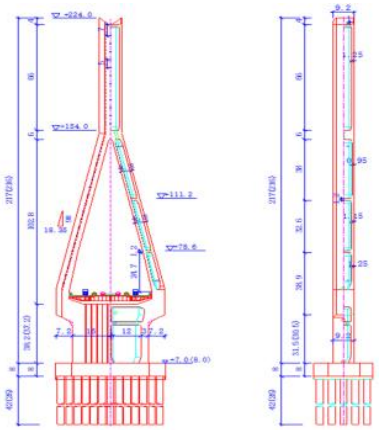


Fig.1 Schematic diagram of main tower

3 Establishment of finite element model

The general finite element program ANSYS is adopted in the calculation model, and the concrete is

made up of solid45 element, which is 1/2 model, and the bottom of the tower is consolidated.The whole bridge model to calculate the maximum axial force and bending moment of tower bear, aiming at the maximum axial force and maximum bending moment analysis, calculation and selection of parameters. The weight calculated by 2.6 kN/m^3, 8320kN bearing force, prestress load loss of prestress cable to 80%, on the most unfavorable load axial force of tower bottom load is 393342 kN. The longitudinal shear is 1939 kN, the longitudinal bending moment for the main tower of finite element model of 385415 kN · M. As shown in Figure 2.

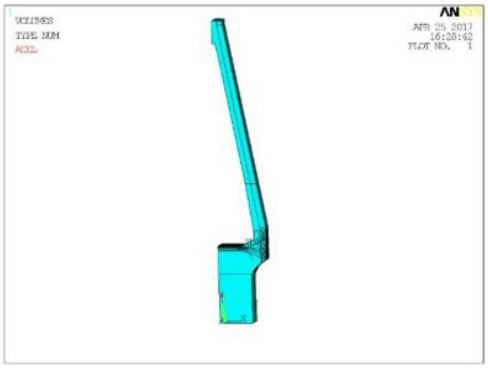


Fig.2 Finite element model of bridge tower

Under the tower made innovation than ordinary masonry type tower, the lower part of the ordinary double leg together as a whole wall section structure, relates to the mass concrete in construction, will produce a lot of heat of hydration, and in the process of construction, under the beam cantilever outward, to the tower in the construction period, until the first brace setting position in the tower is a vertical cantilever, the weight of root is lower beam and pylon in combination to generate additional moment, so the beam beam chamfer and the middle tower junction may appear relatively large stress, it is necessary to control the tensile stress in the the scope of the security. So it needs to be arranged on a lower beam transverse prestressed reinforcement beam under tension. The horizontal offset prestressed using 80 root diameter of 15-25 low relaxation steel strand, prestressed beam layout as shown in Figure 3.

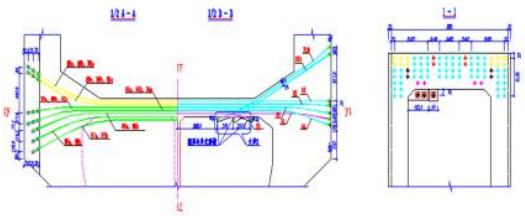


Fig.3 Prestress arrangement of lower beam

4 static analysis

Through the establishment of ANSYS tower model, static analysis, the results are as follows:

Fig.4 The normal stress distribution of the bridge pylon (Pa)

According to the analysis results, it is found that the normal stress distribution in the inner and outer side of the bridge tower is more uniform than that of the outside, and the numerical value is stable in a certain range.

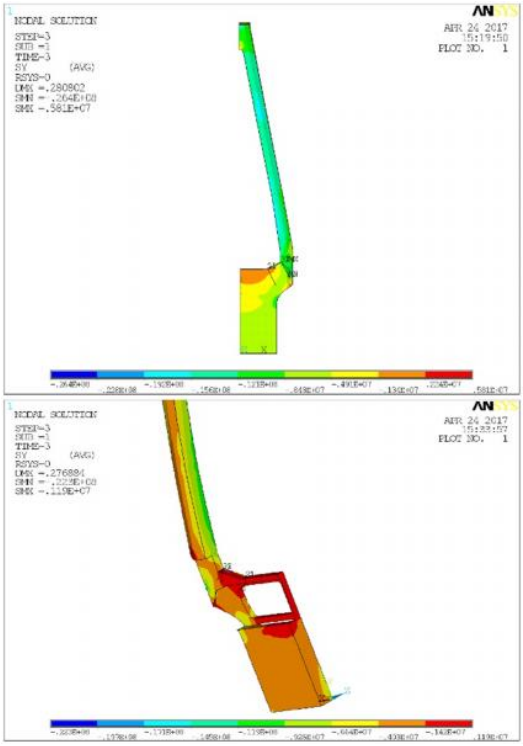


Fig.5 Vertical normal stress distribution of pylon (Pa)

The tower lateral vertical stress numerical distribution is not uniform, the interception of the tower inner stress distribution of tower under the tower roof beam found and cross beam tension stress value is small, the maximum tensile stress in the beam on the surface, about 1.19MPa.

Fig.6 Normal stress distribution along the bridge tower (Pa)

The numerical value of the normal stress distribution along the bridge tower is more uniform, and the tensile stress in the lower part of the cantilever is lower than that of the cantilever, and the maximum tensile stress is about 2.86MPa.

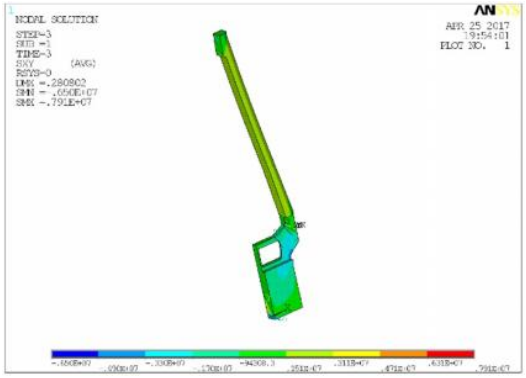


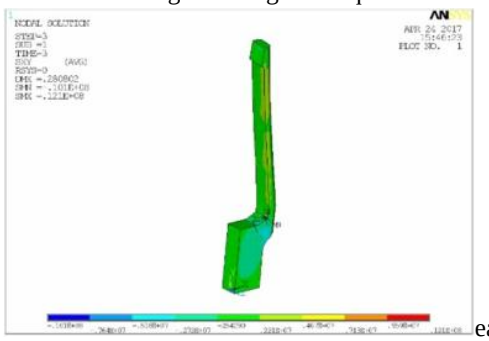
Fig.7 Distribution of shear stress in pylon (Pa)

The tower lateral shear stress value distribution is uniform in column shear stress values in the -0.94MPa ~ 1.51MPa range, under the tower in -3.3MPa ~ 1.5MPa.

To sum up, through the tower tower beam with prestressed structure layout, the tensile stress in the numerical control and a very small range, basically in the tower of compression only, that the reasonable layout of prestressed.

5 Dynamic response analysis

Considering the design earthquake intensity and



earthquake, the general finite element software ANSYS calculation in the most unfavorable loading conditions under the effect of tower beam, dynamic response force parts of the beam and tower column with the key, reveal the tower structure under the earthquake load dynamic characteristics.

First on the tower model for single point excitation response spectrum analysis, from the horizontal and vertical direction of the two incentives, understand roughly tower displacement, and then applied on the tower three to the artificial seismic wave, using the method of uniform excitation of seismic time history analysis.

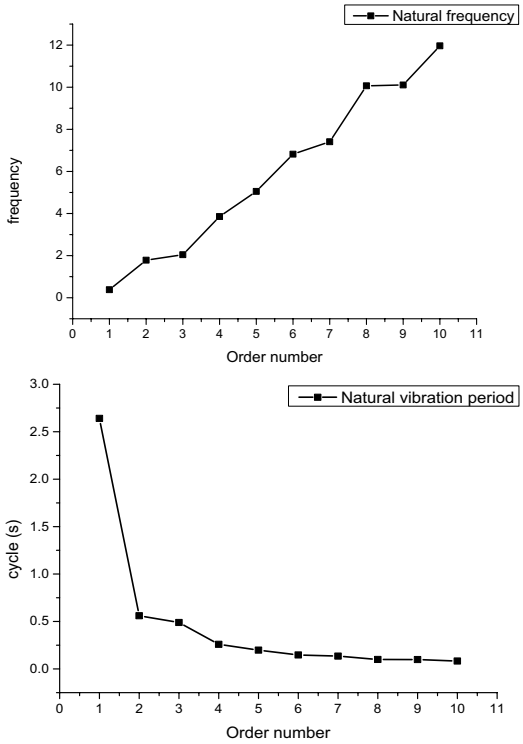


Fig.8 Natural frequency and natural vibration period of bridge tower

5.1 Response spectrum analysis

5.1.1 Lateral excitation

The application of ANSYS analysis method of seismic spectrum, obtained by modeling the modal solution, thus to obtain spectral solution, further expansion mode, finally combined with observation. The incentive direction is horizontal, and by changing the characteristic period of seismic influence coefficient for different towers were 8 degrees and 9 degrees under frequent earthquake and rare case of displacement the earthquake under the action of figure 9-10, in the 8 degree earthquake displacement of pylon distribution from the bottom to the top of the tower is increasing in intensity, the top of the tower is the most obvious incentive effect, the strongest, so the maximum displacement in the top of the tower, the other three cases is the same as the first. The distribution of the maximum displacement of the 8 degree earthquake tower value is 0.000858m; 8 degrees under the rare earthquake tower the maximum displacement value is 0.004827m; the maximum displacement of the 9 degree earthquake tower value is 0.001716m; The maximum displacement of the bridge under the condition of 9 degree earthquake is 0.007509m.

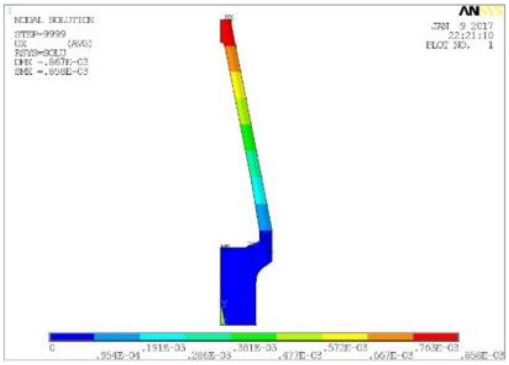
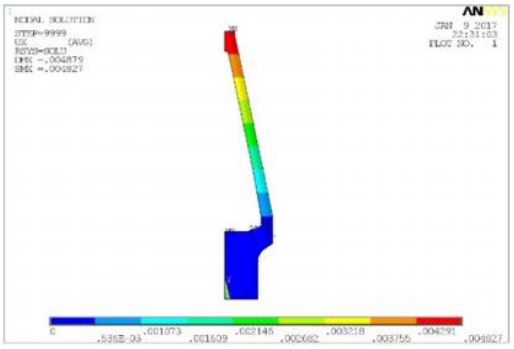


Fig.9 8 degree frequent and rare earthquake bridge tower tower displacement distribution (m)



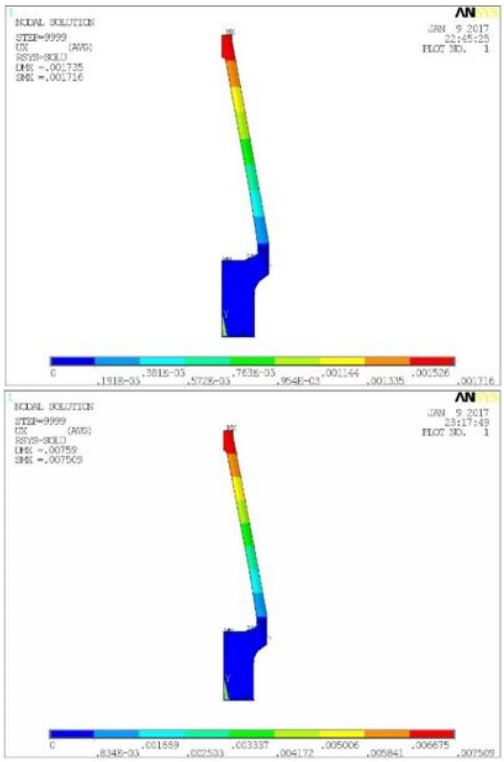


Fig.10 9 degree frequent and rare earthquake bridge tower tower displacement distribution (m)

5.1.2 Longitudinal excitation

The direction of longitudinal displacement excitation effect, numerical changes and distribution is consistent with the transverse excitation, the maximum displacement increases, the maximum displacement of the 8 degree earthquake tower value is 0.031022m; under the rare earthquake is 0.174499m; the 9 degree earthquake is 0.062044m, under the rare earthquake is 0.271443m.

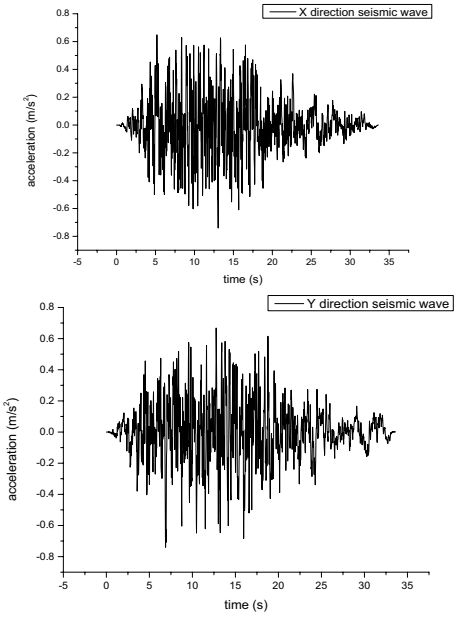
5.1.3 Results of response spectrum

The tower in different seismic intensity in different degrees, displacement is from the bottom to the top of the tower at the top of the tower increases, the incentive effect is most obvious, the strongest, and the displacement value than the transverse longitudinal excitation, the maximum displacement is at the top of the tower, the tower height of 217m, according to the maximum displacement of pylon the calculated displacement ratio to meet the requirements of specification.

5.2 Seismic time history analysis

5.2.1 Seismic wave generation

By applying ANSYS, earthquake response of the three directions of the model, applying seismic inertia loads on the structure, seismic inertia force by way of acceleration input into the structure, and then the quality of the structure together to form a dynamic load calculation of inertia, considering the seismic design intensity of 6 degrees, the site is classified as class II. The damping ratio is 0.02, the use of Simqke software to generate artificial earthquake wave, numerical as shown in Figure 11.



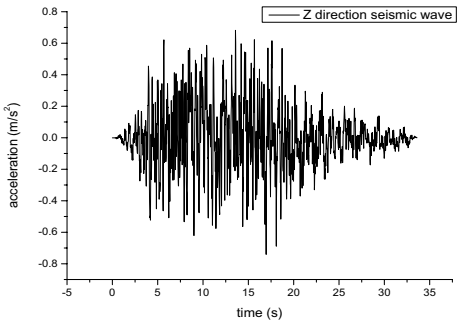
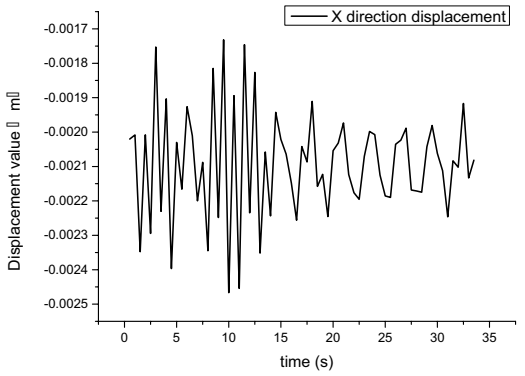
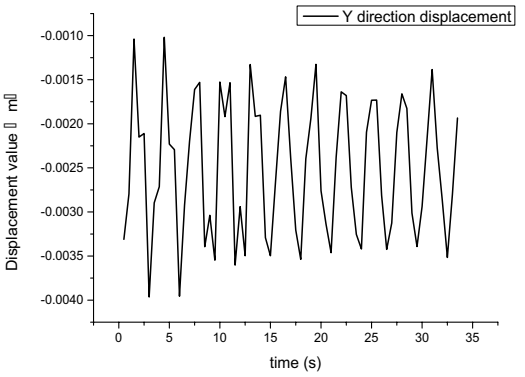


Fig.11 X, Y, Z direction seismic wave value

5.2.2 Seismic wave generation Analysis result

The seismic wave is added to the model in 3 directions in the process of operation of the earthquake, select tower beam under chamfer and the middle tower junction, the junction of the tower, the node displacement values of the two position of the comparison are as follows.





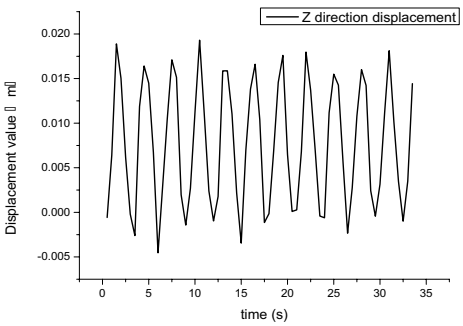


Fig.12 Numerical lower beam chamfering and middle tower junction node displacement

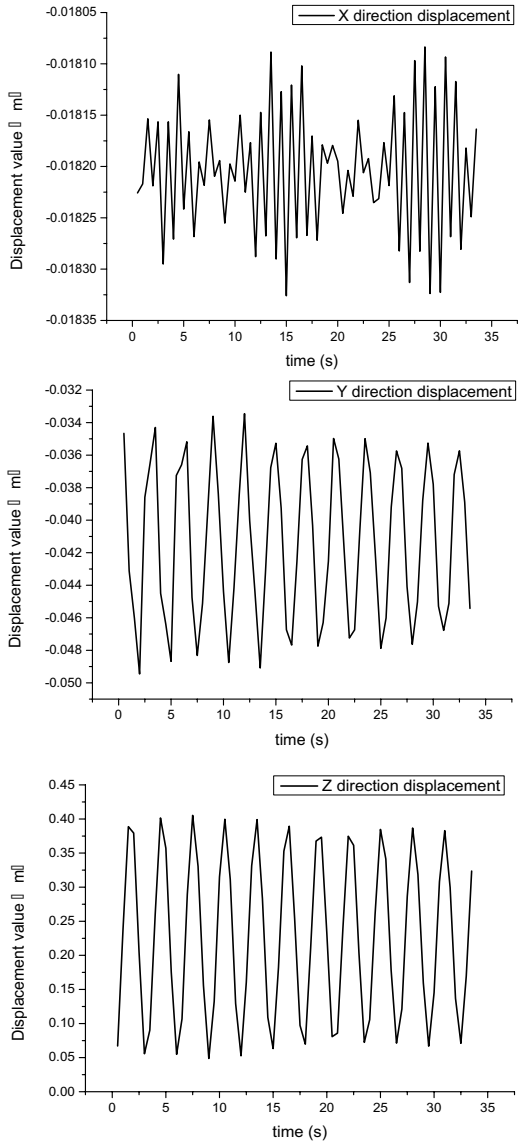


Fig.13 Numerical tower junction node displacement in

Found from the tower beam and pylon and chamfer junction in the tower at the junction of displacement, node, tower junction in the earthquake excitations is obvious, the displacement value is relatively large; the displacement excitation X direction two positions of the results are obvious, mutations, the former will inspire the attenuation, the latter has been fluctuating; displacement excitation Y two position direction and Z direction of roughly the same, is to keep a stable fluctuations, fluctuations in the value of the small range of the former, the latter in the larger range fluctuations.

The stress change of the nodes in the lower beam of the bridge tower is selected:

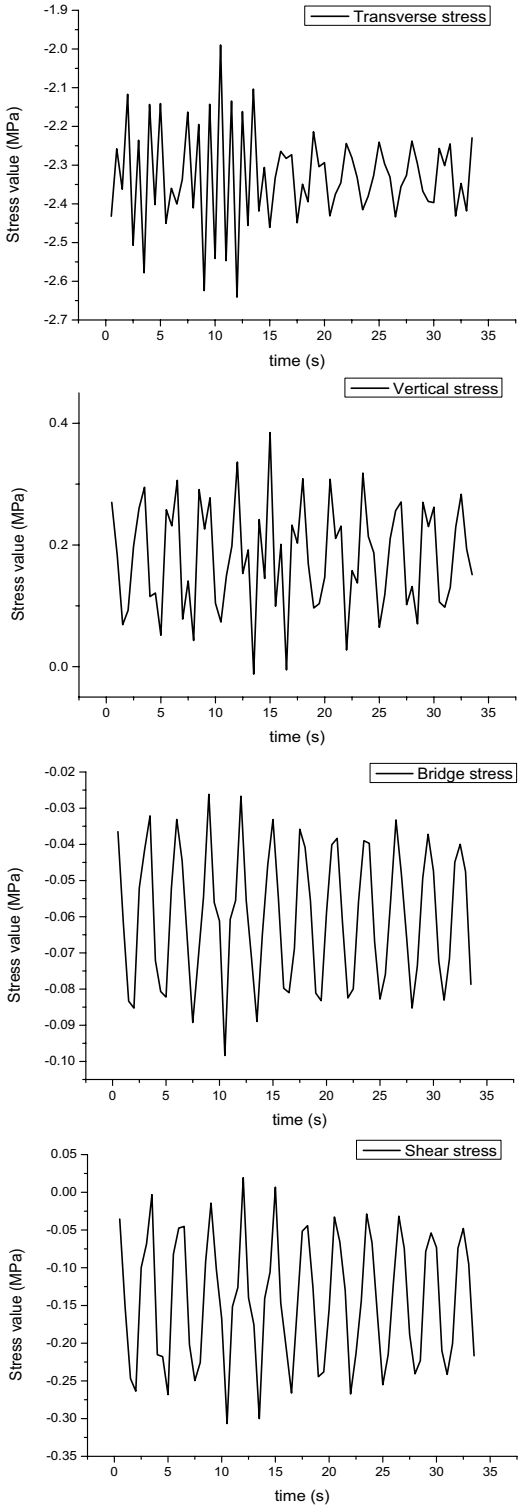


Fig.14 Numerical tower junction node displacement in

The node at the tower beam under the action of seismic wave, the transverse stress, vertical stress, longitudinal stress and shear stress values will fluctuate in a certain range, the transverse stress should exist numerical mutation and vertical and transverse stress in

the larger range of fluctuation, the maximum value is about 2.65MPa; compared with other three kinds of stress, vertical stress is tensile stress, but the value is small, the maximum value is about 0.4MPa; the longitudinal stress and shear stress are in the range of numerical stability in small fluctuations.

6 Conclusion

The main tower of a large cable-stayed bridge is a diamond tower, the tower model is established by using the finite element software ANSYS, and the static and dynamic analysis is carried out:

1) through static analysis, found the whole tower stress distribution, in addition to a small vertical beam under local and longitudinal tensile stress, only in compression, the stress value of the control in a very small range, also shows the rationality of prestressed reinforcement.

2) through the dynamic analysis, first of all, the spectral analysis shows that, under the action of the strong earthquake of 8 degrees and 9 degrees, the displacement change is from the bottom of the tower to the top of the tower

3) in the seismic time history analysis, found in the upper incentive effect is more obvious than the incentive effect of the beam, the displacement value is relatively large, the three directions of the incentive effect in the X direction (transverse) displacement numerical wave is obvious; the beam in three to the seismic wave were used, cross the bridge to stress and vertical stress fluctuation is obvious, only the vertical stress is tensile stress; on the whole, the tower has better seismic performance.

Acknowledgements

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URBAN DEVELOPMENT A CASE STUDY OF BEIJING'S URBAN CONSTRUCTION AND CULTURAL RENEWAL

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Key words: urban context; urban culture; cultural renewal.

Abstract

Nowadays, the urban images are becoming more and more complex and the cities tend to assimilate into one another. The issues such as the disappearance of urban characteristics become more prominent. It is necessary to protect and develop the historic buildings and the buildings with regional characteristics. Through discussing the relationship between city context and city culture, city context and city development, it is proposed that urban architectural culture development in Beijing should be based on updating the nation's traditional culture, absorbing foreign cultural essence, combining with the needs of modern social life to create new architectural culture with the characteristics of the times. It is pointed out that updating is the inevitable trend since cities need to develop.

1. Introduction

As the process of economic integration of the globe accelerates, the problems of global cultural convergence and the disappearance of urban features have become more prominent. Therefore, the protection of historical and cultural cities such as Beijing, respect for regional, national culture, and emphasis on the design concept of the urban context are the historical mission of urban planners and designers in new era.

Beijing, the capital, which is changing dramatically, is being watched by people all over the country and around the world. Architectural styles change constantly. In two decades, with the completion of some big projects such as the widening of the peace avenue, the completion of a number of commercial centers, the development of mass residential buildings and large-scale relocation of old cities ,Beijing is changeable rapidly. We also pay a huge price when the builders do their utmost to make Beijing a modern international metropolis. For instance, the walls and buildings, courtyards and hutong that represent Beijing's traditional architecture and cultural characteristics have vanished. Today, in Beijing there are only four city gate towers that are preserved, such as the Chengyang city gate tower and watchtower, Deshengmen watchtower, the southeast corner tower of the inner city, and there are two sections of the residual wall located on Xibianmen and the east street of the Beijing railway station. Instead, it is a complex of buildings with the modernist style which is tall, thick and devoid of regional features. The famous historical and cultural city in Beijing is losing its features.

2 Urban context and urban culture

The word "context" refers to "context" in literature. In a broader sense, it is the relationship between one thing and another in time or space. In urban design, context means the relationship between man and architecture, architecture and city and the whole city and its cultural background.

Urban context contains dominant and recessive forms, and the dominant forms include people, land and things, while the recessive forms refer to the situation and development of city that have potential and profound influence factors, including the city's politics, economy, historical events, cultural background, social conventions and psychological behavior and so on. [1] The innovation of the urban context, not only includes the building materials and architectural styles but also includes the urban structure, nature, topographic features, the continuity of the city life and demand, etc.

At present in China, a large number of copies similar to western architecture have poured into China intensively. Some cities' features are disappearing, and regional characteristics are weakening. In the face of the assimilation of contemporary city image and the mixture of city landscape, it is essential to protect and develop the historic buildings and buildings with local characteristics. This is the city's history and cultural memory.

Beijing, the ancient capital of the world, is made up of two main parts. The first part includes the Forbidden City and the former imperial gardens. The second part is Beijing residential area which occupies a larger area, which regards the imperial palace, the Forbidden City, as a center. The residential areas are made up of many hutong and siheyuan with go-pan-type structure. If hutong and siheyuan that reflect the style of ancient capital are gone, instead, there are big bank buildings and commercial shopping centers which are luxuriously decorated. Can you find out the culture with more than 3000 years history of city and more than 800 years

history of capital? If Beijing is just a replica of the skyscrapers of Hong Kong, New York and Tokyo, can it attract millions of tourists from all over the world every year? According to Xinhua's statistics, in the first half of 2011, the total number of foreign tourists visiting Beijing was 2.4 million. What are the most important things for tourists visiting Beijing for the first time? Of course, they focus on the historical and cultural relics that represent Beijing. Therefore, the historical and cultural relics such as the Forbidden City, the Temple of Heaven and the Great Wall are the preferred tourist routes of the foreigners. But every foreigner is more interested in the different culture compared with their own culture if they are able to probe into the connotation of the national culture.

Today some bottleneck problems in Beijing urban construction, such as the developing pattern which has been extended peripherally based on the old city as the center, seems to have become a hard problem to curb. Recalling the time when Beijing was set as the capital in the founding of new China, Liang Sicheng, in the "Suggestions for the central people's government administrative center ", claimed to protect and improve the old city of Beijing , and suggested building new district in the western suburbs as a administrative center, and said it was a " satisfactory solution". Mr. Liang and his wife, Lin Huiyin, also called for the protection of the walls and towers of Beijing and used them as a "city park" surrounded the central city. [2] The two Suggestions could both completely avoid the contradiction between protecting the style of the old city and the modernization construction to make the best of both worlds, and could preserve the important symbol of the old city walls and towers in Beijing. In those days, however, his efforts were futile. Then, decision making relied on the urban development of the old city is the main sticking point of Beijing's development which is in dilemma today.

The urban development of Beijing has undergone several major stages of development before becoming such a feature today. Take the design of the Beijing architectural design institute as an example, there were 18 public works completed between 1952 and 1956, including 13 ethnic and traditional forms, 72% of the total works [3], such as Friendship hotel, Jishuitan hospital and Beijing department store. By the 1980s, the two main architectural arts in Beijing were the ethos of modernism and national tradition. Although modernism occupies a certain dominant position, the proportion of national form buildings in all buildings is slowly rising. [3] In the 1980s, take "the top 10 buildings in Beijing in the 1980s" as an example, the traditional form of building has four of them, accounting for 40%.[3] This showed that the traditional form of the buildings occupied an important position in the construction practice in the 1980s. In the 1990s, the construction movement was "retaking the ancient style" due to the international and domestic "literary pulse". Instead of promoting the development and enrichment of architectural art, the movement has created a number of architectural failures. Beijing west guest station is the typical representative of the failures.

In the 21st century, Beijing has added several landmark buildings because of hosting the Olympic Games, such as the "bird's nest", "water cube" and "Pangu tower". In order to prosper the capital culture market, the national grand theatre and CCTV headquarters were built. Although these buildings were designed by foreign designers or architectural design firms, they were accepted and loved in different ways. "Bird's nest" and "water cube" are very popular with the masses because there are some Chinese elements in the design, but also form a harmonious symbiosis of square and circular shapes, becoming a beautiful scenery in Beijing. On the contrary, the national grand theatre seems to be out of place for its huge size and brightness of outer epidermis because of the complexity and the sensitivity of its location and the surrounding relation, which leads to the fact that cultural characteristics are gone. CCTV's new building is controversial because of its challenges of architectural structure, its high cost, its bizarre shape and its traffic problems. As in February 1950, Liang Sicheng and Charles Chen put forward in the "Suggestions for the central people's government administrative center position ", they mentioned that the buildings in a city can't be "immune", and they must cooperate to reconcile with the environment. [4]

The more abundant the building arts form in a city, the better. The buildings are certainly put in the right place. Mr. Zhang Qinnan has praised that Paris is the city of protecting general buildings and innovating landmark buildings. Meanwhile, he has also criticized that Beijing is the city of demolishing general buildings and modeling landmark buildings,[5] In terms of development trends, the modern form will be largely dominant, but it will complement the traditional form of the ethnic buildings.

We can learn from the protection and utilization of the historic buildings in Paris. Paris is an ancient historical city with a romantic passion for innovation and fantasy, such as the Eiffel Tower, the Pompidou arts center, the Louvre pyramid, La villette park, the new arc

de triomphe of La Defense, etc., For one hundred years, new buildings have added brilliant colors for the city constantly. The ancient city and the new district coexist and the ancient civilization and the modern civilization echo as if people can seek the context and the trace of the development of the famous European city. [4]

Since the reform and opening up, the urban development in our country has interpreted that the economic globalization leads to cultural globalization, which has had a negative effect on the traditional culture of the city. The influx of migrants into big cities, apart from the normal cultural mix, has had a big impact on the traditional culture of the big cities and the way people live. Old buildings, which are carriers of traditional cultural material, no longer exist. The indigenous inhabitants as the creators and inheritors of the traditional culture have also been removed. The life forms of history have been broken, the traditional network has been destroyed, and the "cultural identity crisis" of the city is very prominent. With the global economic integration, all ethnic groups, countries and regions increasingly link. Can urban cultural heritage continue to maintain its survival and independence of development? The immigration of outside population and the emigration of the main body bearing the weight of local intangible cultural heritage are evolving into a war between a strong economy and the "weak" culture without smoke of gunpowder and suspense. Can the architectural space, as material carrier of traditional culture, make a difference? Can the cultural renewal through the old buildings inherit the urban context organically? These problems are what we have to focus on.

3 Architecture and urban context

City is not only created to be artificial environment to meet the needs of human survival and development, but also a cultural carrier and containers, accumulating the rich cultural background, carrying the essence of human civilization. In this sense, the city itself is the cultural heritage and a complex series of value. The urban culture in every historical period is a trace of the urban culture. Therefore, the urban context is the atmosphere of history, culture, geography and the environment that human population in a city has formed in the long-term development, which reflects the unique existence and way of life. Buildings are the material carriers of the collective memory of the city and the place of urban occurrence. With the development of times, the old urban areas of some cities have lost their economic competitiveness, and they have become increasingly unfashionable in the process of development. But these urban areas are the core of material cultural heritage and intangible cultural heritage, and they are in the heart of the city culture, with a particular value of civilization. The old buildings in the city are not "out-of-date", and they also have a "propulsion" effect. These buildings take "the past" into "now", making the present people experience the "past", and they are able to accommodate different functions because of the time changing. As a result, the urban context of architectural space in historic regions of the old city should not exclude but should combine the industry layout and the transformation of urban functions to keep pace with the times. We should not only stay in the physical environment improvement and aesthetic level, but pay attention to fostering and introducing a new industry, and promote the regeneration of urban functions and energy a macro level to activate the city's society and local culture to create more jobs in order to improve the urban economy, urban finance, urban competitiveness, etc. Thus, the continuity of the urban context is dynamic development, not static protection, which conforms to the law of development of things.

4. How to update the development of urban architecture culture

In the culture renewal of the architectural space creative industries can be introduced. It is good for paying close attention to the culture and lifestyle of original residents, which benefits the protection of industrial buildings, historical blocks with traditional style and the traditional craft. It can maximize the protection of urban historical view, historical buildings to realize the combination of landscape protection and economic development, and it is beneficial to the continuation of the urban context. It is of great significance to solve the problems of the protection of city history and culture in the process of urban regeneration.

In respect of the traditional space and environment, the construction and new functions are added to meet the needs of modern people. In fact, it is not a contradiction to improve the living conditions of the inner city’s residents and the historical and cultural characteristics of Beijing. This can be learned from the experience of Paris. The specific measures are as follows:

4.1 The number and area of the Siheyuan reserves: need enlarging and connecting to pieces.

Important traditional homes need to be protected as a whole to ensure that the environment on which they depend is not broken. If the overall protection is not allowed, local protection and maintenance shall be carried out. Measures can be taken such as reducing the

population density in the old city, controlling the population of inner cities and moving some residents to new residential buildings outside the city. To restore the standard courtyard houses in inner city, the courtyard houses are turned into a single house with one family or a house with one courtyard.

4.2 Improving the infrastructure, public facilities and transportation facilities in urban areas and making inner cities a habitable zone

Residents living in urban areas have poor living conditions without available water and gas. There is no heating in the winter. If all these problems are solved, who wants to move to an apartment? So what the government needs to do is not only to protect the original culture texture of Hutong and courtyard houses, at the same time to make modern people see that living in the Hutong and courtyard houses is very comfortable and convenient.

4.3 The courtyard houses in inner city: free to buy and sell

If a house needs to be rebuilt, the owner needs to rebuild it at his own expense, but the look of the house needs to be rebuilt strictly in accordance with the kind of the specifications of gray walls and tiles and bungalow top, maintaining the look of Hutong and courtyard houses. The house can be decorated with modern style. The advanced residential areas can be formed similar to the rich zone of Ming and Qing dynasties.

4.4 The protection of the traditional courtyards houses: involving the transformation of its function and the reuse

An influential celebrity's old house or a representative residence can be converted to memorials. Doing this not only helps to improve the value of the protected residence, but also reflects the important role of the combination of matter and spirit. Small Siheyuan can be converted into dining rooms, because they are small, easy to emphasize the pleasant atmosphere, and easy to create a special family atmosphere, such as special tea houses, restaurants, cafes, bars and so on. Typical courtyard houses with a larger size and convenient transportation can be selected to use as communication places with purpose, which means better protecting courtyard houses to promote the architectural culture, and to endow the good way to give new life with traditional architecture, such as halls, clubs, antique shops, senior hotels, etc.

Every living city has its own past, present and future. This determines the importance of protecting the historical environment and the development of the Times. The city needs to develop continuously. The renewal is the inevitable trend. Historic buildings of the city as part of the urban space, on the one hand, witness the years, which need to protect, on the other hand ,they are carriers of the city, along with the development of the urban social economy, and this development is both orderly and organic. [6]

5. Conclusion

Urban development is an oxymoron. If the city is not metabolized, it will become unviable, so urban cultural renewal is inevitable. Whether urban culture survive or not depends on its ability to adapt to the transformation of urban function and the transformation of industry. With respect to the entry of strong capital culture from outside in the old central city, we should also pay enough attention, especially the development model will aggravate urban "culture identity crisis" because large-scale development of the property has forced indigenous residents to relocate. Cultural regeneration of architectural space is not only the continuation of the physical form environment, but also the continuation of life environment connotation. Therefore, the proper control of the floating population and maintaining the original residents' living characteristics are beneficial to the inheritance and renewal of the urban context.

The globalization competition of the new century is based on the competition of the economy, information, technology and culture of the city. The city is not just a building entity and it also marks a person's spiritual orientation and cultural depth. The charm of the city does not come from the technological strength of economic power, but from the cultural integration, which is manifested in its own uniqueness and regionalism.

Acknowledgements

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Detection and Evaluation of Post-Fire PC Box Girder of Bearing Capacity

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Keywords: Prestressed concrete bridge, fire, damage, Bearing Capacity, Safety evaluation

Abstract

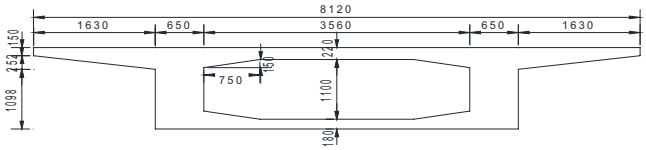
Based on the damage detection results of post-fire bridge, considering the influence of concrete strength decrease, the damage degree of the main section and the loss of prestress affecting factors, the finite element method is used to calculate the bearing capacity and compare the bearing capacity of the bridge before and after the fire, to evaluate the safety of the bridge. Some important conclusions are drawn as follows: The influence of fire on the bearing capacity of the bridge is very complicated, not only has the impact on the direct fire structural part, but also affect the bearing capacity of the non-burned parts. After the fire, the damage of the bridge is the key to the evaluation of the carrying capacity of the bridge. The evaluation of the carrying capacity of the bridge after the fire and the reinforcement of the post-disaster bridge are very important to ensure the safety of the bridge.

1 Introduction

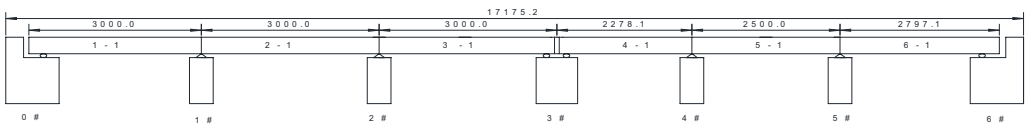
The fire of the bridge is divided into the bridge floor fire and the under bridge fire[1], the impact of the under bridge fire on the bridge is more serious. But no matter what kind of fire form will lead to different levels of damage to the bridge structure, the effect of fire on the structure of the bridge is related to the form of the bridge structure, the location and range of the fire, the degree of fire. The bridge after the fire behaves as durability damage and structural damage according to the different damaged parts and damage degree. Among them, the bearing capacity decline caused by structural damage directly affects the operational safety of bridge structure, it is also an increasingly concerned about and urgent problems for the Bridge management department to be solved. But the occurrence of bridge fire is accidental, due to the occurrence of fire data collection and collation is more lacking, the lack of research data about inspection and evaluation of bridge fire is the main problem of bridge safety assessment after fire at present[2]. This article takes a bridge in Beijing as an example, based on the test data of the bridge after the fire, the bridge safety is evaluated by analyzing the bearing capacity of the bridge after the fire.

2 Engineering background

A bridge in Beijing was damaged that the reason was the accumulation of fire, the fire part was composed of 2 connecting 3 pairs of prestressed concrete continuous box beams, its total length was 171.752 m and the clearance of span was 4.5m. The span combination were straight line (30+30+30) and curve segment (22.781+25+27.971).The total width of the bridge floor was 8.5m(Fig.1). Longitudinal prestressed steel beams were made of 9-7φ5 low relaxation steel strands, its standard tensile strength was 1860Mpa.The design load was Auto-super 20.



Cross section



Elevation map

Fig.1 Bridge layout （unit：mm）

3 Damage detection of bridge after fire

The fire caused a series of physical and chemical changes in the concrete material , such as cement stone、aggregate phase transformation、increased crack、loose porous structure、cracking and disconnect of cement - aggregate interface, the concrete material from the surface to the center gradually loosed and cracked[3]. The difference between the depth and the degree of damage caused by the difference between the temperature and the duration of the fire was very different. Therefore, the key to the fire damage detection and evaluation of concrete structure is how to accurately determine the damage degree after the fire. Considering that the concrete core drilling has a certain damage to the structure, according to the scope of the fire, the load-bearing components such as box beams, cap beams and pier columns were selected for non-destructive testing in the fire-affected areas[4,5].

3.1 Damage detection of bridge appearance

Fig.2 The left web of box beam appeared corrosion and spalling

Fig.3 The left bottom of box beam appeared corrosion and spalling

Fig.4 The right web of box beam appeared corrosion and spalling

Fig.5 Burn makers in the beam button overall

After the fire ,the main diseases of the bridge upper structure bearing component were as follows:3-1 Away from the 2# pier 6 meters ,the left web of box beam appeared corrosion and spalling ,its area was 2 m^2(Fig.2). 3-1 Away from the 2# pier 6 meters, the left bottom of box beam appeared corrosion and spalling , its area was 2.5m^2 (Fig.3). 3-1 Away from the 2# pier top 6 meters , the right web of box beam appeared corrosion and spalling , its area was 1.5m^2 (Fig.4). 4-1,2-1,3-1 beam bottom of the overall existence of burning marks (Fig.5) .

3.2 Strength test of the concrete

3.2.1 Presumption of strength of concrete structure

According to the damage of the concrete after fire, using the turbine to remove the loose layer, slurry, coating on the surface of concrete, and then clean the surface. The strength of concrete members is tested by rebound method, according to the same table[4] rebound curve of China, we get the conversion value of concrete compressive strength of the measured area *i*, then calculate the mean and variance, and finally calculate the constructive value of concrete strength. the rebound method is used for testing. Three components are selected for concrete strength test, results of test and evaluation [6,7] are shown in table 1.

Table 1 shows that the measured value of concrete strength of No. 0 abutment is higher than the design value, and the component strength is in good condition .The measured value of concrete strength of Box girder 2 - 1 and 3 - 1 is lower than the design value in fire zone, indicating that the bearing capacity and tensile strength of the structure are affected.

Table 1 Results of concrete strength test

Bridge member	Compressive strength of concrete （MPa）					Uniformity coefficient Of estimation strength	Uniformity Coefficient of average strength	Component strength state
	Design value	Measured mean value	Measured standard deviation	Measured minimum value	Measured estimated value			
2-1Beam	50	39.4	1.68	36.5	37.2	0.744	0.788	Bad
3-1Beam	50	38.8	1.52	35.2	36.5	0.730	0.776	Bad
0 Abutment	30	45.2	1.42	41.5	42.6	≧0.95	≧1	Good

3.2.2 Carbonation depth of the concrete

The concrete carbonization depth was measured by using 75% alcohol solution and white phenolphthalein powder at a concentration of 1 to 2%, the detection contents included the upper structural box girder and the lower structure abutment, 3 members were selected to test the carbonation depth of concrete in the rebound test area or where there indications that Corrosion of steel bar may exist, and then list the results[7] in Table 2.

The carbonization of box girder and abutment in Table 2 shows the ratio of measured concrete carbonation depth of bridge member to the thickness of protective layer is less than 0.5. The concrete did not lose effectiveness on the protection of the steel bar, and has little effect on the durability of the concrete.

Table 2 Test results of concrete carbonation

Bridge member	The average value of Measured concrete carbonation depth（mm）	Carbonation depth/ The thickness of Protective layer	Effect on the durability of the concrete
2-1 Beam	3.5	＜0.5	Little
3-1 Beam	3.5	＜0.5	Little
0 Abutment	3.5	＜0.5	/

3.3 Detection thickness of protective layer of reinforce concrete structure

The stress position or there are signs that the reinforcement may be corroded are the main positon to detection thickness of protective layer, three components were selected for the thickness of the concrete protective layer, and the results of test and evaluation [7] are shown in table 3.

Check component location	Thickness of protective layer （mm）				Eigenva lu-es value D_{ne}	Design value D_{nd}	D_{ne} / D_{nd}	On the durability of structural concrete
	Maximum value	Minimum value	Average value	Standard deviation				
2-1Beam	41	29	34.0	1.58	31.3	30	>0.95	Not significant
3-1Beam	44	28	33.5	1.65	32.8	30	>0.95	Not significant
0 Abutment	35	29	33.2	2.12	29.7	30	>0.95	Not significant

Table 3 Test results of concrete cover thickness

reinforce concrete structure of box girder and abutment is greater than 0.95, which shows that the protection of concrete to rebar is not effective, and has no significant influence on the durability of structural reinforcement.

4 Influence of Fire on Bridge

The prestressed concrete bridge is composed of concrete, steel bar, high strength steel wire or steel strand. After the fire, the structure is heated and the temperature is increasing. Due to the thermal inertness of the concrete material, the uneven temperature field is formed inside the structure. Correspondingly, concrete and steel construction material is deteriorated so that structural performance decreased, resulting in varying degrees of damage [3]. Due to the different aggregate and mix ratio of concrete, the mechanical and thermal properties of the fire are changed greatly after fire. The study shows[8,9] that prestressed tendons will suffer from prestress losses under high temperature, due to the high temperature effect leading to the relaxation and creep of prestressed steel, the shrinkage and creep of concrete, the difference between the thermal expansion of reinforced and concrete and the mutual influence of these factors. Therefore, combined with the bridge non-destructive testing data, considering the decrease of the concrete strength, the damage degree of the main section and the prestress loss on the bearing capacity of the bridge:(1) According to the appearance inspection in Figure 2.1 and Appendix B.1 , Table H.1, Table H.2 in the literature[10], when the temperature of concrete reaches 300 ℃, the strength of concrete peeling zone is reduced by 0.8, and 2-1 and 4-1 are at normal temperature and do not be reduced.

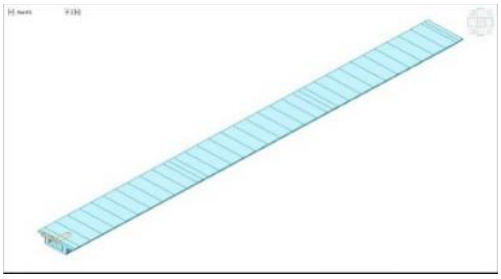
(2) From Table 3, it can be seen that the concrete has a peeling and the thickness of the concrete protective layer is 4.4 cm. According to the literature [11], the height of the cross section of the box girder 3-1 is reduced by 4 cm, and the height of the cross section is reduced by 2 cm.

(3) According to the appearance inspection in 2.1, the prestressed steel bar is not damaged, but the bridge will produce high temperature when the fire occurs, because the heat conduction leads to the irreversible deformation of the prestressed steel bars, the bearing capacity of the bridge will be affected. Combined with the literature [12] in Table 2.4-4 and literature [13] in Table 3.2-1, the prestressed strength of the box girder 3-1 is reduced by 0.927, and the prestressed strength of the box girder 2-1 and box girder 4-1 is reduced by 0.978.

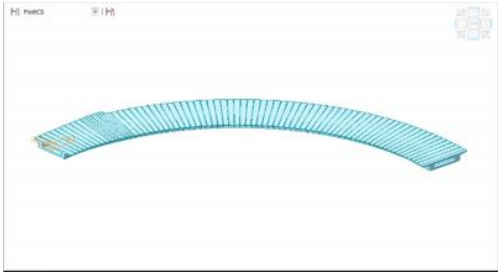
5 Analysis of bridge bearing capacity after fire

5.1 Finite element model

With the MIDAS/Civil 2015, the finite element model of the space bar is established ,as shown in Figure 6. The concrete is C50 and the weight is calculated according to 25 kN/m^3.The bridge deck pavement is applied in the form of linear load, and the prestressed reinforcement is adopted by Strand1860 and the proof stress 1339.2 MPa. The load combination consists of carrying capacity limit state load combination and the normal use limit state load combination.



（a） Straight line



（b） Curve segment

5.2 Data analysis

A finite element model considering three factors affecting fire on bridge is established. Two midspan moment are drawn in Fig.7 and Fig.8, and 1/4 span moment drawn in Fig.9, in Fig.10.

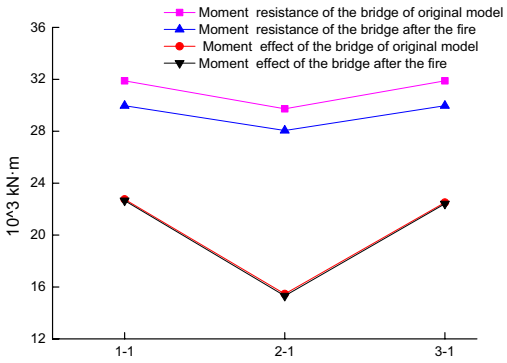


Fig.7 Bending moment variation of straight segment of bridge at the midspan

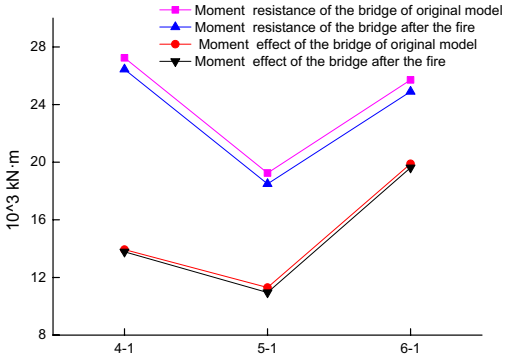


Fig.8 Bending moment variation of curve segment of bridge at the midspan

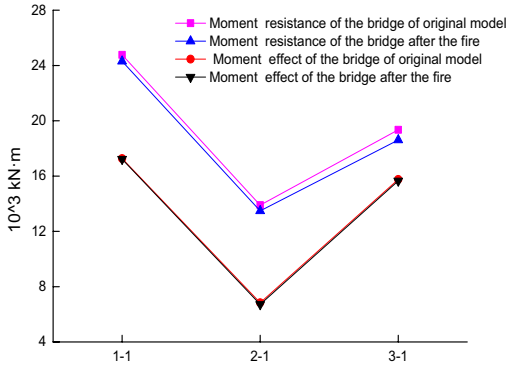


Fig.9 1/4 Bending moment variation of straight segment of bridge

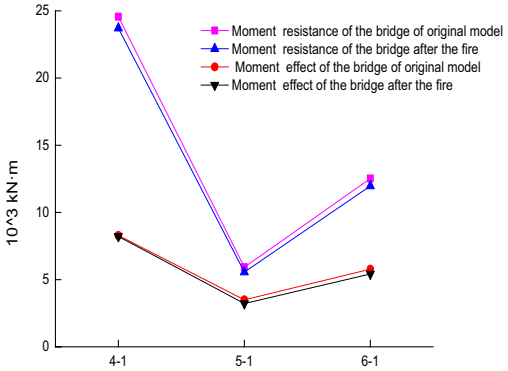


Fig.10 1/4 Bending moment variation of curve segment of bridge

It can be seen from the Fig.7~ Fig.8 that the bending moment resistance of the bridge changed greatly after the fire, but the bending moment effect was little changed. They show that the fire has a great influence on the bending moment resistance of the bridge. From Fig.7 and Fig.9, it can be seen that the mid-span bending moment resistance changed greatly. They show that the fire has a big impact on the bridge. From Fig.7 to Fig.10, it can be seen that the fire had an effect on the overall resistance of the bridge, but the basis of determining quantitative index of impact degree was slightly inadequate. The reason was: high temperature when the fire, the heat transfer effect also had an effect on the damaged area of the bridge, resulting in complex changes within the bridge structure, the influence of bridge after the fire on the bearing capacity of bridge is still to be studied deeply.

6 Conclusions

Through the analysis of the fire problem of a bridge in Beijing, the following conclusion is drawn:

(1) After fire, concrete and reinforcement mechanics are affected, using nondestructive testing method for the real reflect the actual state of the bridge is the basis for bridge safety evaluation、 maintenance.

(2) The fire effect on the bearing capacity of the bridge is very complicated, not only impact on structural part directly by the fire, but also impact on not the bearing capacity of the damaged area, therefore, the maintenance of the bridge considers the damaged parts, also should considers non-damaged area.

(3) The bridge bearing capacity and durability has impacted after fire, and the main influence on the performance of prestressed concrete in three sides: one is the weakening of the cross section of concrete, the second is to remain the influence of the prestressed, and three is the effect on the durability of concrete.

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Review of Seismic Behavior of Assembled Anti-lateral Steel Members

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Key words: anti-lateral steel members; seismic behavior; assembly

Abstract

The assembly structure will become one of the important development trends of the future construction industry, which has the advantages of energy saving, environmental protection and short construction period. As a component against horizontal lateral force, the anti-lateral force steel members play a particularly important role in the seismic construction of steel structures. This paper reviews the current research status of seismic performance of conventional anti-lateral steel members such as concrete-filled steel tubular columns, steel plate shear walls and steel supports. The advantages and disadvantages of the traditional anti-lateral steel members in resisting horizontal lateral force are introduced. The characteristics and development prospects of the assembled buildings are introduced. Combined with the production and construction requirements of the assembly building, the paper analyzes the reasons why the traditional anti-lateral steel members are not suitable for the assembled buildings. The paper puts forward the main research direction of the anti-lateral steel members for the assembled buildings in the future.

1 Introduction

Assembly architecture will become one of the important development trends of the future construction industry. Assembly of buildings refers to the factory production, assembly of the construction, integration of decoration, information management, intelligent applications, standardized design approach to achieve energy saving, materials saving, water saving, land saving and environmental protection. With the development of assembly buildings and steel structures in Europe, the United States, Japan and other countries, in recent years, China is vigorously developing the assembly of buildings. China had also introduced policies and in large reports mentioned to vigorously develop the assembly Type building. The demand for anti-lateral force steel members for assembly buildings is also increasingly urgent. This paper summarizes the research status of seismic performance of traditional anti - lateral steel members, such as concrete - filled steel tubular columns, steel plate shear walls and steel supports, and introduces the advantages and disadvantages of traditional anti - lateral steel members in resisting horizontal lateral force. This paper introduces the characteristics of assembly architecture and its development prospect. Combined with the production and construction requirements of the assembly building, the paper analyzes the reasons why the traditional anti - lateral steel members are not suitable for the assembled buildings. The paper puts forward the main research direction of the anti - lateral steel members for the assembled buildings in the future.

2 Seismic behavior of anti - lateral force steel members

2.1 Review of seismic behavior of concrete filled steel tubular columns

P. Gajalakshmi, H. Jane Helena[1] reported the results of an experimental study in the study, which found that the filled steel column subjected to quasi-static load cumulative damage. The concrete-filled steel tubular columns were studied by using the diameter-thickness ratio of the steel pipe and the two types of fillers. The common concrete and steel fiber reinforced concrete were studied.

Shan, J.H. and Chen, R.[2] designed a kind of confined concrete-filled steel tube, which is a new type of lattice column. In order to verify the bearing capacity and ductility of the column, the authors carried out static and quasi-dynamic loading tests. Experiments show that the column shows good performance during loading. In addition, the performance of the column and the traditional concrete-filled steel tubular column under the same impact load is compared, and it is found that the

column has better mechanical performance than the concrete-filled steel tubular column.

Sun Chao, Chen Baochun[3] studied the influence of eccentricity on the short columns of concrete-filled steel tubular lattice is studied, and the influence factors are studied experimentally. Among them, the specimen includes eccentric compression specimen and eccentric tension member. After the test analysis, it is found that the failure type of the specimen is divided into the type of crush and the limb. When the eccentricity is small, the eccentricity is small. And when the eccentricity is large, the component is pulled bad. And for the tensile specimen, it is pulled bad. This study analyzes the calculation methods associated with the design specifications of concrete-filled steel tubular structures in China. At the same time, the calculation equations related to the bending moment of CFST short columns are put forward.

Chen Baochun, Ou Zhijing[4] studied the ultimate bearing capacity of concrete-filled steel tubular column and its influencing factors, the ultimate bearing capacity of concrete-filled steel tubular column was studied. The slenderness ratio and eccentricity were used as the main parameters of the test. The results show that the two main parameters are independent of the ultimate bearing capacity of the column. On this basis, the bearing capacity of the lattice column is obtained. The results show that the two main parameters are independent of the ultimate bearing capacity of the column. The relationship between the approximate reduction formula of the force and the reduction ratio of the slenderness ratio and the eccentricity reduction coefficient. The effects of slenderness ratio and eccentricity on the ultimate bearing capacity of lattice columns are described in a more comprehensive way.

Chen Bowang et al.[5] Studied the failure behavior and seismic behavior of the concrete-filled steel tubular column structure in the low cycle cyclic reciprocating load. At the same time, it better explained the overall performance of the two different cross-sections on the concrete-filled steel tubular column Impact. The failure modes and seismic performance of the two kinds of concrete-filled steel tubular columns were obtained. The failure modes of the two models were similar to the seismic performance. However, in the case of energy dissipation and stiffness degradation In this paper, the influence of the cross section of the circular steel pipe on the whole steel column is obvious. Finally, the application of the concrete-filled steel tubular column is discussed.

Weng Chengchiang et al.[6] Proposed an experimental study on the seismic behavior of rectangular reinforced concrete (SRC) columns with new multi-spiral cage. The multi-spiral cage refers to the center having a large helix, with four small helix "5-helix" in the corner, and the application of the 5-helix to the SRC column is to take advantage of its advantages in concrete confinement and automation efficiency for full-size SRC The results show that the newly proposed five-helix can be successfully applied to the prefabricated rectangular SRC column. The results show that the SRC column can be used in the SRC column.

Wang Tiecheng et al.[7] Established a finite element analysis model in order to study the effect of steel strength on the seismic performance under the condition of steel content, slenderness ratio and axial compression ratio. At the same time, the quasi-static loading test of concrete-filled square steel tubular column was carried out. The finite element analysis model can be applied to seismic analysis. The influence of the three influencing factors on the seismic performance of the concrete-filled steel tube is analyzed. It is found that the axial compression ratio is relatively large and the slenderness ratio is small and the steel content is larger in the range of certain research. Reflect the ability to better energy consumption.

2.2 Review of seismic behavior of steel plate shear wall

Jebelli, H. and Mofid, M.[8] The numerical simulation analysis of nine thin steel plate shear walls were carried out. The shear wall was made of 160Mpa, 200Mpa and 240Mpa thin steel plates respectively. The results show that, under the effect of the use of low yield point of the energy consumption of thin steel plate more.

Sabouri Ghomi Saeid. And Sajjadi Seyed Ramin Asad[9] are modified on the basis of thin steel plates to increase the shear stiffness of thin steel plates and improve the energy consumption of thin steel plates. The experimental results show that the ductility and energy dissipation capacity of the steel plate with special treatment are greatly improved, and the experimental results are similar to the numerical simulation results, and the mechanical performance is stable.

Nie Jianguo, Zhu Li[10] mainly studied the quasi-static test of steel plate shear wall, which mainly focuses on its seismic performance. The variation parameters considered during the test process mainly relate to the setting of the opening and the middle of the wall panel, and the comparative specimen is set up. The results show that the shear wall structure of the steel plate exhibits

better bearing capacity, ductility and energy dissipation capacity under the action of the earthquake. Both the parameters of the shear wall structure in the stiffness and bearing capacity of the steel plate are helpful to improve the shear wall structure performance.

Guo Yanlin et al.[11] Experimental study on the shear wall of anti-buckling steel plate embedded in two-layer steel frame. The purpose of this study is to study the interaction between the anti - buckling steel plate shear wall and the method of numerical simulation analysis. Through the research, it is shown that the buckling steel plate shear wall shows good ductility and energy dissipation capacity in the process of the experimental study, which is obviously better than the comparative specimen.

Guo Yanlin, Dong Quanli, Zhou Ming[12] studied a new type of anti - lateral force component, which is suitable as a high - rise building structure, is studied in this paper. Two kinds of analytical methods, such as theoretical analysis and quasi - static test, are studied. The ultimate bearing capacity, the failure mode, the ductility and the hysteretic behavior of each specimen were studied, and the stress mechanism of each specimen was revealed. The results show that the bearing capacity of each specimen is higher than that of the specimen. And the characteristics of the force, summed up and evaluated the steel wall of the mechanical performance.

2.3 Review of seismic behavior of steel support

Eehrouz Asgarian and Hamed Rahman Shokrgoza[13] analyzed a new support structure for offshore platforms. The strength of two different directions of the offshore platform is monitored by the incremental dynamic method. At the same time, the FOD detection system is used to simulate the full-scale model. The required IM and DM values are obtained, The results show that the support in the marine platform configuration shows good operational capacity.

Shi Yongjiu et al.[14] Studied the eccentric support steel frame structure. The parameters such as the failure mode and the hysteresis performance of the frame are obtained by the experimental phenomena and the experimental data. Based on this, the eccentricity of the eccentric support steel frame structure under the earthquake is obtained. The basic performance of the steel frame is supported to provide some necessary experimental data for the experimental study of the eccentric support steel frame structure system, which is more convenient for the systematic study of the system.

Cheng Guangyu, Ye Liping, Xu Xiuzhen[15] proposed a buckling energy consumption steel support. The results show that the energy consumption of the steel can be used to test the tensile and compressive strength of the supporting material in the test process. And the design of unbonded construction measures of steel support and concrete also meets the requirements. Finally, the end structure of the steel is put forward by the test results.

Wu Hui et al.[16] Studied the anti-buckling support, and this as the anti-lateral force and energy consumption unit for the study of the quasi-static test study, through the test is to verify the anti-buckling support under the action of the earthquake can occur first yield damage It can dissipate a large part of the energy before the yield of the concrete frame, and improve the seismic performance of the structure, and the anti-buckling support can be replaced in the application process. After the replacement, the seismic performance of the structure is still good.

3 Review of assembled buildings

Chen Jianbo[17] summed up the United States in the assembly of steel structure of residential research. At present, the United States multi-storey residential, villas and other basic use of light steel structure. The United States insists on modularization and standardization of components, realizing the versatility of the main structure components, forming a relatively perfect steel structure residential system.

The United States in the last century 70s during the energy crisis began to implement the construction of parts and mechanized production. US Department of Urban Development has introduced a series of strict industry standards, has been in use ever since. American urban residential structure is basically to the factory, concrete assembly and steel structure-based, reducing construction costs, improve the plant versatility, increased construction of the operability.

Lu Junfan et al.[18] Introduced the European and American assembly of steel structure mainly used light steel keel system, the system has the following advantages:(1) light weight, basic costs and transportation and installation costs;(2) all kinds of

accessories can be factory production, high precision, good quality;(3) room space is large, flexible layout;(4) good wind and seismic performance;(5) construction and installation of simple construction speed, less construction waste, easy to recover the material;(6) indoor water and electricity pipelines can be hidden in the wall and floor structure, to ensure that the indoor space integrity;(7) do not need a second renovation. Lu Junfan also introduced China's assembly-type steel structure of the main structural forms are:(1) thin steel steel residential system;(2) new layered assembly support steel structure system;(3) many, high-level light steel assembly-style residential.

In 1891, France had been the implementation of the assembly of concrete construction, so far has been 120 years of history. The French construction industry is mainly composed of concrete system, steel and wood structure system, and adopts frame or plate column system, and gradually develops to large span. In recent years, the French construction industry is characterized by: (1) welding and other dry work popular; (2) structural components and equipment, decoration works separately, reduce the embedded, making the production and construction quality; (3) Prestressed concrete frame structure system.

Liu Meixia[19] summed up the development of assembly buildings in Japan. Japan's development of the assembly of the building mainly used the following four measures:(1) the development of relevant laws and systems;(2) with a standardized way to promote the large-scale production of parts parts;(3) clear institutional functions;(4) issued a number of support policies.

In 1968, Japan put forward the concept of assembly-style housing. In 1990, Japan adopted the component, factory production mode, high production efficiency, residential internal structure is variable, to meet the needs of diversification. In addition, Japan has adopted legislation to ensure the quality of concrete components, in the assembly of residential areas to develop a series of principles and policies and standards, but also formed a unified modular standards to solve the contradiction between the standardization, mass production and diversification needs.

4 Discussion

The anti-lateral force steel component is a component that is used in building structures to play an important role in seismic action. Common anti-lateral force components are steel support, steel plate shear wall and concrete pipe and so on. As mentioned above, for steel support, steel plate shear walls and concrete-filled steel tubular columns, both in terms of seismic performance and structural aspects have many advantages, but not for standardized design, factory production , The assembly of construction. Therefore does not apply to the construction of buildings. Can not meet China's urgent need to vigorously develop the requirements of the assembly of buildings. To a certain extent, limits the process of China's industrial construction, restricting China's current development of assembly-style buildings and steel Structure development. Therefore, in the future for the assembly of the anti-lateral strength of steel components should be able to meet the components can be standardized design, factory production, assembly and construction requirements to promote the development of China's assembly-style buildings.

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PERFORMANCE OF SELF-CENTERING STEEL FRAMES WITH INTERMEDIATE COLUMNS CONTAINING FRICTION DAMPERS UNDER HIGH-MAGNITUDE SEISMIC ACTIONS

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Keywords: Intermediate Column with Friction Dampers, Resilient Prestressed Steel Frame, Seismic Performance.

Abstract

A self-centering steel frame system with intermediate columns containing friction dampers (hereafter known as ICSCF) has been proposed for the use in a large span, seismically active regions and a high-rise building. Analysis on investigating the seismic performance of the self-centering steel frame system with intermediate columns containing friction dampers when it subjected to the seismic actions with high magnitudes, i.e., the peak ground acceleration value may reach the 0.62g and 0.8g, is carried out, by adopting the numerical simulation method through the software Abaqus. Additionally, analyzing results such as the slippage of the intermediate column friction damper, the gap-opening, the base shear, the story drift and the energy dissipating condition are compared with those of the self-centering steel frame (SCF). The results show that an ICSCF effectively controls the story drift, dissipates more energy through the intermediate column friction damper (ICFD) and protects the components from developing plasticity relatively earlier compared with SCF. A resilient prestressed steel frame system with intermediate columns containing friction dampers has a small post-seismic residual story drift, demonstrating that it may realize the target of self-centering after undergoing seismic actions with high magnitudes1 Introduction

Prestressed self-centering steel frame structures can fully leverage the advantages of prestressed steel. It can reduce structural damage, reduce or eliminate residual deformations, and is easy to repair after a strong earthquake. Therefore, it has received widespread attention from researchers and has been extensively studied (Ricles et al 2001; Garlock et al 2003; Kim et al 2008;Iyama et al 2009; Wolski et al 2009) [1]~[6].To accommodate a prestressed steel frame structure system with a large span and in a high intensity seismic zone, a prestressed steel frame system with intermediate column containing friction damper is proposed (hereafter known as intermediate column prestressed steel frame or ICPSF). Finite element model of the overall structural of an ICPSF is built up and a comparative analysis is performed with the seismic performance of a prestressed steel frame with beam web friction dampers (Tsai et al 2008; Lin et al 2013; Zhang,A.L. et al 2013; Zhang,Y.X. et al 2014) [7]~[13] (hereafter known as prestressed steel frame or PSF). The properties of the ICPSF, such as the floor acceleration, the base shear, the story drift, the post-seismic structure recovery capability and the energy dissipation capability are investigated.

2 Sample structure and modal analysis results of 2 types of structures

2.1 Sample structure

Sample frame structure has 10 floors; the height of the first floor is 3.6 m, floors 2-10 are 3m high, there are 5 spans widthwise and 5 spans lengthwise, and each span measures 10m. The plane diagram of the structure is shown in Figure 1. The illustration of the finite element model towards the overall structure is shown in Figure 2.

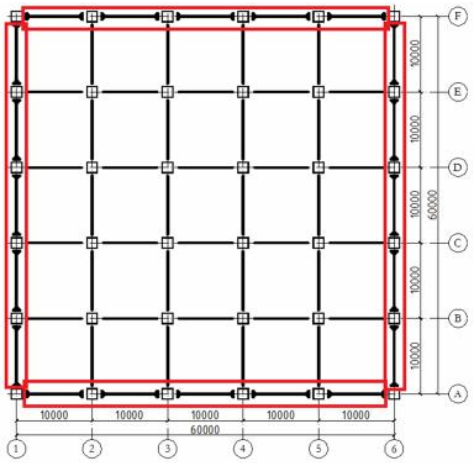


Fig.1 Plan of the structure

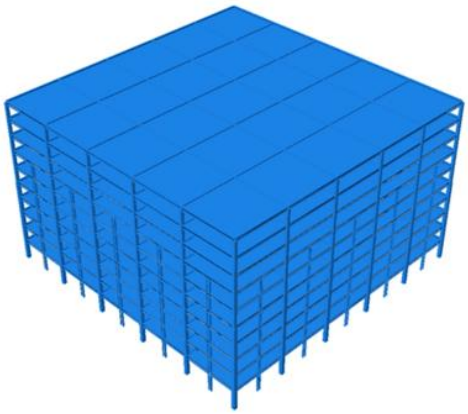


Fig.2 Finite element model

The red sections are the ICSCF at floors 1-6 and SCF at floors 7-8; the other parts are hinge systems. The fortification strength was degree 8. The constant floor load, which included the weight of the floor plate, was 7.0kN/m^2, the variable floor load was 2.0kN/m^2, the variable building surface load was 2.0kN/m^2. The main parameters of the structure are defined as follows.

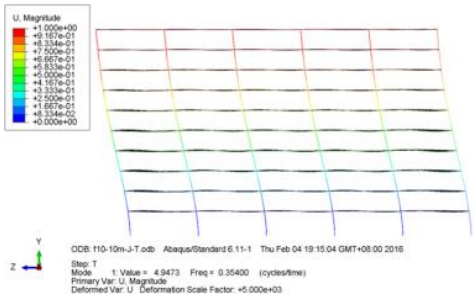
Table1 Dimensions of beam and column sections of ICSCF and SCF

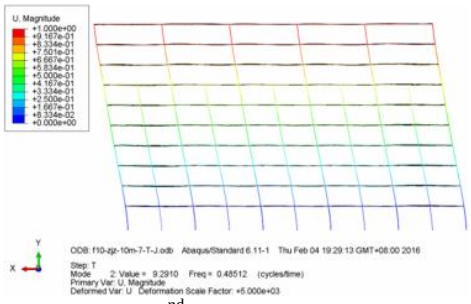
2.2

Floor	Section	Dimensions（mm）	Section	Dimensions（mm）
6-10 Floor	prestressed columns hinge columns	500×500×20	prestressed beams hinge beams	600×300×14×24
3-5 Floor	prestressed columns hinge columns	550×550×24	prestressed beams hinge beams	600×300×14×24
1-2 Floor	prestressed columns hinge columns	600×600×28	prestressed beams hinge beams	600×300×14×24

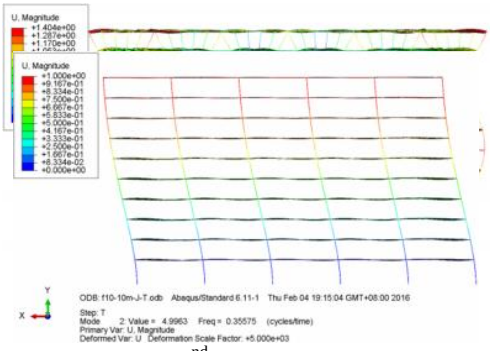
Modal analysis results of the 2 types of structures

The finite element software ABAQUS has been adopted to carry out the model analysis toward the ICSCF and SCF, which the first 6 vibration modes can be achieved as shown in Figure 3. It can be seen that the first 6 vibration modes of ICSCF are basically coincident with those of SCF. Each natural period calculated from the frequency is shown in Table 2. The data show that, compared with the SCF, the period of the ICSCF is significantly reduced, i.e., the first three natural periods have reduced amplitudes of at least 13%. This result indicates that the use of the ICFD in the design of the ICSCF provides lateral rigidity and torsional rigidity that is significantly higher than those of SCF, therefore contributing significantly to the overall rigidity of the structure.

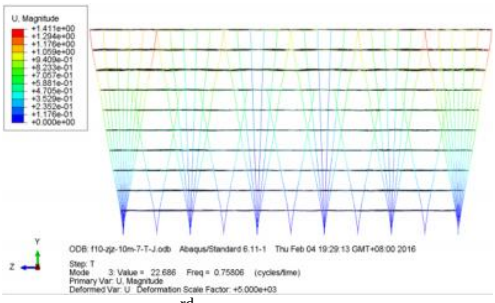




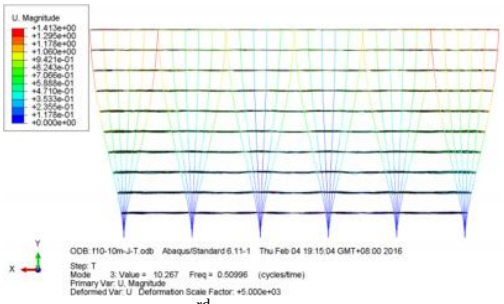
(c) 2nd mode of ICSCF



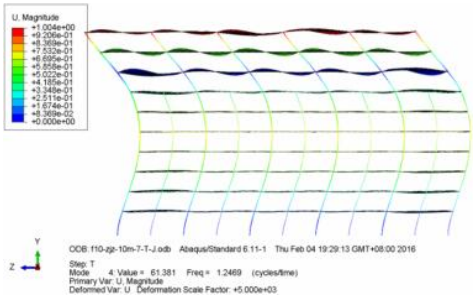
(d) 2nd mode of SCF



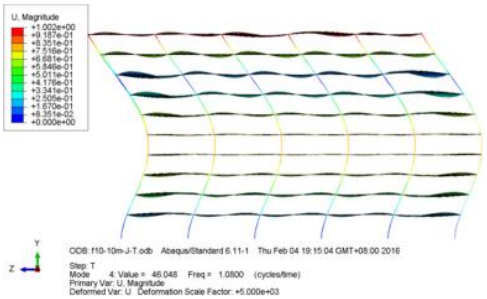
(e) 3rd mode of ICSCF



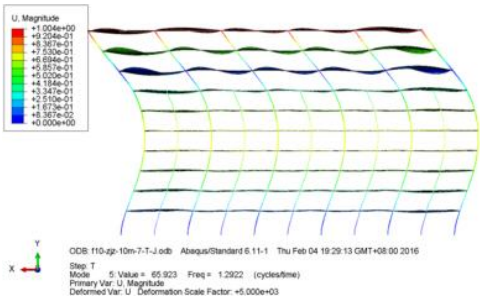
(f) 3rd mode of SCF



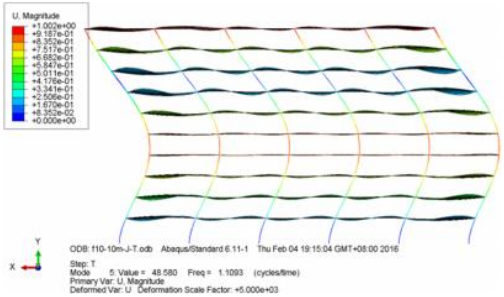
(g) 4th mode of ICSCF



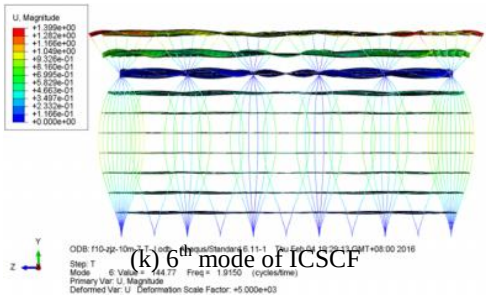
(h) 4th mode of SCF



(i) 5th mode of ICSCF



(j) 5th mode of SCF



(k) 6th mode of ICSCF

(l) 6th mode of SCF

Table 2 Comparison of natural vibraion mode of ICSCF and SCF （s）

Mode	1	2	3	4	5	6
SCF	2.82	2.81	1.96	0.93	0.90	0.64
ICSCF	2.08	2.06	1.32	0.80	0.77	0.52
Percentage of the difference	26.42%	26.67%	32.73%	13.39%	14.15%	18.67%

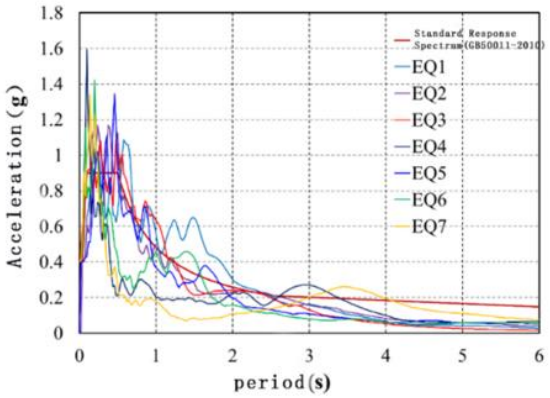


Fig. 4 Acceleration spectrums of 7 ground motions

Table 3 Basic data of 7 ground motions the time-history analysis take

Serial number	Ground motions	Magnitude	Years	Name	Seismic platform	Influence coefficient difference
1	EQ1	6.9	1995	Kobe,Japan	Shin-Osaka	8.44%
2	EQ2	6.5	1987	Superstition Hills	Poe Road(temp)	-4.74%
3	EQ3	7	1940	Imperial Valley	EL-Centro Array#9	-5.40%
4	EQ4	7.5	1999	Kocaeli,Turkey	Duzce	-11.82%
5	EQ5	7.4	1952	Kern County	Taft Lincoln School	-31.34%
6	EQ6	7.3	1992	Landers	Yermo Fire Station	-39.15%
7	EQ7	6.5	1979	Imperial Valley	EI-Centro Array#11	-55.40%

3.2 Time history dynamic analysis results under 8-degree rare ground motion (PGA=0.4g)

3.2.1 Slippages of the friction damper and connection gap opening

Table 4 appears that the maximum slippages of the friction damper of ICSCF in both directions under 8-degree rare ground motion(PGA=0.4g). Table 5 shows the maximum connection gap opening of both ICSCF and SCF. According to the table, the slippages of ICSCF reached 61.57mm under the ground motion EQ4. The gap opening reached 10.63mm in X directon under EQ6 that is relatively small compared to the slippage, which indicates that the friction damper can provide greater friction energy consumption during resulting in a large slippage.

Table 4 Maximum slippage of the intermediate column friction damper in ICSCF （mm）

Ground motions	EQ1	EQ2	EQ3	EQ4	EQ5	EQ6	EQ7
Z direction	38.22	41.35	31.38	52.46	16.85	41.59	12.75
X direction	40.12	12.95	43.04	61.57	22.50	49.21	24.69

Table 5 Maximum gap-opening of 2 frames （mm）

Ground motions		EQ1	EQ2	EQ3	EQ4	EQ5	EQ6	EQ7
Z direction	ICSCF	3.66	1.38	4.02	5.67	1.84	4.00	2.51
	SCF	5.33	2.45	6.46	9.49	2.21	5.94	4.93
X direction	ICSCF	3.81	4.15	3.21	4.40	2.10	3.97	2.22
	SCF	2.95	5.14	3.69	4.48	2.95	10.63	3.00

3

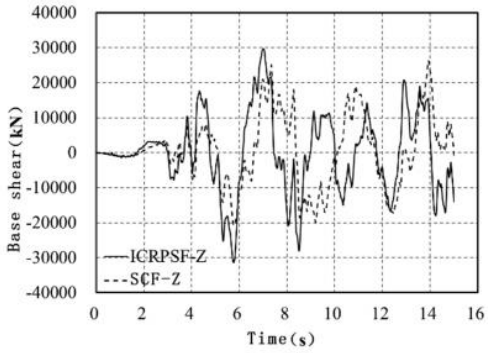
.2.2 Comparison of the base shear

Figure 5 shows the base shear time history comparisons of ICSCF and SCF under the ground motion EQ2 and Figure 6 appears the the base shear time history comparisons of ICSCF、ICSCF-IC、ICSCF-C and SCF. ICSCF-IC means the base shear that only beared by the intermediate column, ICSCF-C denotes the base shear beared by prestressed columns and hinged columns without the intermediate column. Table 6 lists the maximum base shears of the ICSCF、ICSCF-IC、ICSCF-C and SCF in the primary and secondary directions under the above seven ground motions The data in Figure 5、6 and Table 6 shows that, in both directions, the base shears of ICSCF are greater than those of SCF except under the action of ground motion EQ6, and the maximum increase is 50.04%, which indicates that the intermediate column can prove lateral rigidity of the prepressed steel frame that has smaller structural period and bigger base shear than SCF.

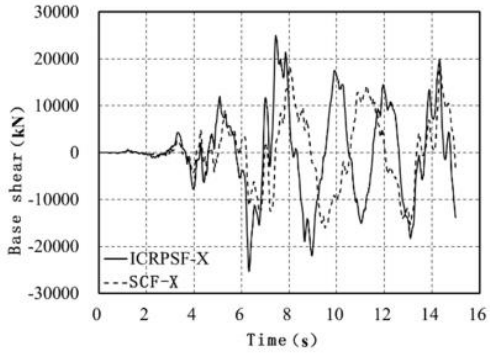
The base shear of ICSCF-IC in both directions is 20%~25% of the ICSCF. The base shear of ICSCF-C in the main direction of most ground motions is less than or close to the SCF. Under 8-degree rare ground motion (PGA=0.4g), a large slippage occurs in the intermediate column friction damper that makes large friction energy consumption, which makes ICSCF-C has less base shear indicating that the ICFD can protect the main structure.

3.2.3 Comparison of the story drift

Figure 7 appears the envelope values of the story drifts of ICSCF and SCF in the primary and the secondary directions under EQ2, besides, the maximum story drifts of each floor are shown in Table 7~8. The maximum of the values are marked underscore. According to the data,most of the maximum story drift values of both directions appear in the 4th floor. The maximun story drift value of ICSCF is less than SCF without EQ1、EQ4 of main direction and EQ5 of secondary direction. The maximum story drift of SCF is 4.116%(1/24) of EQ6, which exceeds the limit of the elastic-plastic interlayer displacement angle of 1/50[14]. Meanwhile, The value of ICSCF is 1.878%(1/53) that meets the requirements of seismic specifications ,which indicates the ICFD can prove lateral rigidity of the structure.

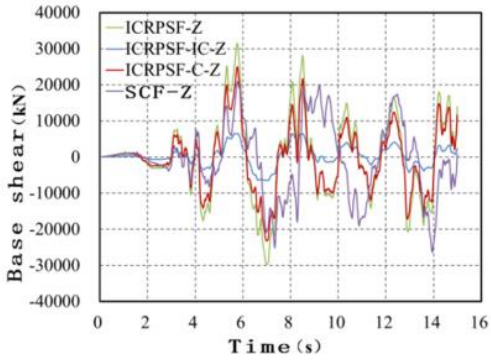


(a)0.4g main direction

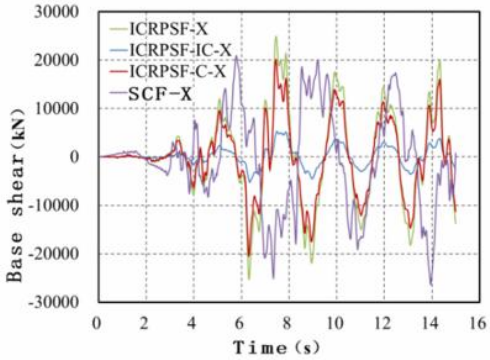


(b) 0.4g secondary direction

Fig.5 Comparison of base shear time-history curves for frames in two directions under EQ2



(a)0.4g main direction



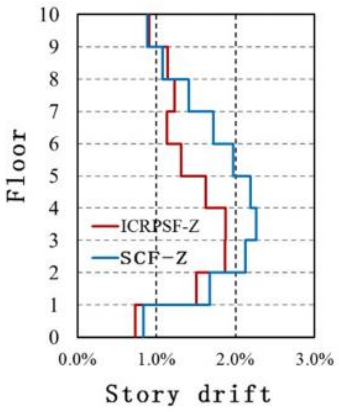
(b) 0.4g secondary direction

Fig. 6 Comparison of base shear time-history curves for ICSCF, ICSCF-IC, ICSCF-C and SCF in two directions under EQ2

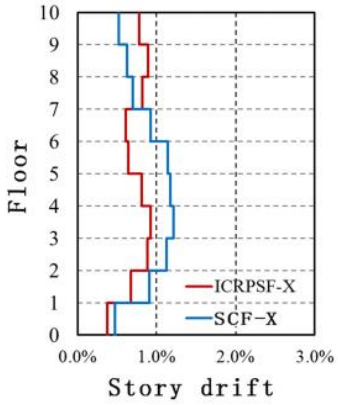
Table 6 Comparison of the maximum base shear of two frames （kN）

Ground motions		EQ1	EQ2	EQ3	EQ4	EQ5	EQ6	EQ7
Z-direction	ICSCF	32511.6	31453.7	34680.2	27010.4	26365.1	32434.7	22222.5
	ICSCF-IC	6488.2	6490.3	6471.6	6430.7	6434.6	6475.6	4554.5
	IC/ICSCF	20.00%	20.60%	18.70%	23.80%	24.40%	20.00%	20.50%
	ICSCF-C	26023.4	24965.1	28209.1	20584.2	20139.5	25959.1	17740.1
	SCF	24976.1	26413.1	26372.2	25947.7	22819.4	32842.9	19358.5
	amplification	30.17%	19.08%	31.50%	4.10%	15.54%	-1.24%	14.79%
X-direction	ICSCF	31516.3	25298.2	32425.1	33671.9	24571.2	25434.4	27255.7
	ICSCF-IC	6489.9	5287.5	6515.1	6602.9	5328.3	6415.1	6435.1
	IC/ICSCF	20.60%	20.90%	20.10%	19.60%	21.70%	25.20%	23.60%
	ICSCF-C	25026.4	20149.9	25910	27088	19349.5	19019.5	20821.5
	SCF	23672	18472	23739.2	30905.4	16376.6	28547.7	20169.4
	amplification	33.14%	36.95%	36.59%	8.95%	50.04%	-10.91%	35.13%

Note: IC/ICSCF means percentage of the base shear of ICSCF-IC/ICSCF



(a)0.4g main direction



(b) 0.4g secondary direction

Fig. 7 Comparison of maximum story drift for 2 frames under EQ2

Table 7 Maximum story drift in the major direction of each floor for 2 frames under 8-degree rare seismic actions

(% rad)

Ground motions		1st	2nd	3rd	4th	5th	6th	7th	8th	9th	10th
EQ 1	ICSCF	0.650	1.331	1.689	1.747	1.559	1.165	1.076	1.280	1.317	1.185
	SCF	0.675	1.266	1.443	1.457	1.394	1.445	1.386	1.205	1.237	1.109
EQ 2	ICSCF	0.733	1.505	1.867	1.874	1.623	1.314	1.137	1.229	1.138	0.906
	SCF	0.837	1.673	2.120	2.263	2.184	1.973	1.722	1.406	1.081	0.882
EQ 3	ICSCF	0.651	1.147	1.239	1.198	1.261	1.397	*1.404*	1.465	1.556	1.323
	SCF	0.623	1.180	1.378	1.593	1.704	1.697	*1.724*	1.727	1.730	1.516
EQ 4	ICSCF	0.609	1.186	1.380	1.482	1.611	1.953	2.101	2.025	1.766	1.477
	SCF	0.616	1.239	1.527	1.612	1.617	1.819	2.016	1.955	1.750	1.412
EQ 5	ICSCF	0.484	0.920	1.053	*1.064*	1.027	0.989	0.933	0.972	1.150	1.104
	SCF	0.525	1.044	1.306	1.426	1.400	1.353	1.315	1.177	1.043	0.876
EQ 6	ICSCF	0.671	1.355	1.715	1.817	1.878	1.859	1.672	1.709	1.585	1.458
	SCF	2.400	3.583	4.069	4.116	3.850	3.349	2.853	2.550	2.201	1.757
EQ 7	ICSCF	0.345	0.568	0.797	*0.932*	0.931	0.863	0.803	1.101	1.219	1.070
	SCF	0.521	0.994	1.230	1.415	1.456	1.298	0.918	0.866	0.914	0.801

Table 8 Maximum story drift in the secondary direction of each floor for 2 frames under 8-degree rare seismic actions

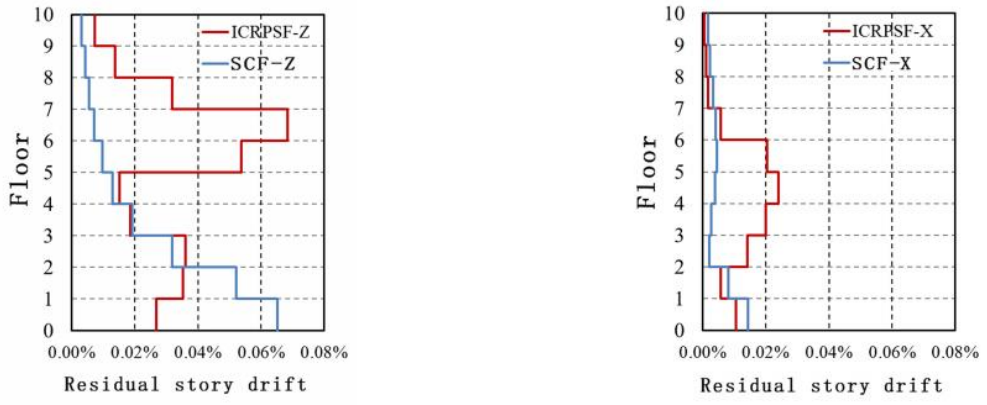
(% rad)

Ground motions		1st	2nd	3rd	4th	5th	6th	7th	8th	9th	10th
EQ1	ICSCF	0.657	1.304	1.630	1.750	1.732	1.597	1.344	1.248	1.133	0.941
	SCF	0.675	1.391	1.849	2.162	2.298	2.355	2.259	2.032	1.752	1.396
EQ2	ICSCF	0.381	0.675	0.885	0.924	0.814	0.646	0.615	0.820	0.890	0.784
	SCF	0.475	0.906	1.130	1.215	1.174	1.139	0.925	0.697	0.628	0.523
EQ3	ICSCF	0.802	1.560	1.862	1.816	1.582	1.291	1.329	1.429	1.222	0.907
	SCF	0.818	1.667	2.292	2.664	2.700	2.464	1.906	1.330	1.033	0.748
EQ4	ICSCF	1.050	1.988	2.416	2.454	2.245	1.946	1.609	1.393	1.046	0.818

EQ5	SCF	1.946	3.144	3.681	3.830	3.689	3.359	2.707	2.011	1.461	1.125
	ICSCF	0.391	0.662	0.749	0.739	0.823	0.993	*1.103*	1.141	1.054	0.917
EQ6	SCF	0.448	0.892	1.109	1.134	1.015	0.891	1.031	0.987	0.798	0.600
	ICSCF	0.493	0.919	1.208	1.361	1.602	1.916	1.948	1.746	1.314	0.964
EQ7	SCF	1.366	2.245	2.532	2.494	2.301	2.006	1.683	1.630	1.301	0.938
	ICSCF	0.531	1.026	1.272	1.284	1.243	1.134	0.909	0.992	0.859	0.638
	SCF	0.764	1.532	1.996	2.189	2.123	1.893	1.487	1.076	0.731	0.497

3.2.4 Comparison of the residual story drift

Figure 8 shows the comparison of the maximum residual story drifts between ICSCF and SCF of each floor under EQ2. The accurate values under different ground motions are listed at Table 9. According to the data, Residual story drifts of ICSCF are colse to those of SCF, the maximum post-seismic residual story drifts of the ICSCF and SCF are 0.1636% and 0.4518% under the 8-degree design ground motions(PGA=0.4g). However, the values are small enough to be ignored.



(a)0.4g main direction (b) 0.4g secondary direction

Fig. 8 Comparison of maximum residual story drift for 2 frames under EQ2

Table 9 Comparison of the maximum residual story drift for 2 frames （% rad）

Ground motions		EQ1	EQ2	EQ3	EQ4	EQ5	EQ6	EQ7
Z direction	ICSCF	0.0989	0.0683	0.0678	0.1636	0.0645	0.0376	0.1215
	SCF	0.0861	0.0239	0.0577	0.2063	0.0419	0.0688	0.0776
X direction	ICSCF	0.0171	0.0651	0.0172	0.0281	0.0031	0.0696	0.0465
	SCF	0.0125	0.0143	0.0857	0.4518	0.0028	0.1010	0.0577

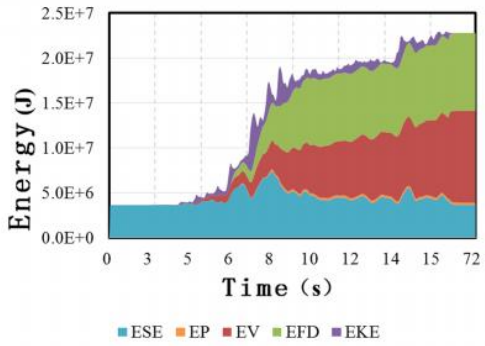
Table 10 lists the maximum residual slippage of the friction damper in both directions under the 8-degree rare ground motions. According to the table, the residual slippage in X direction is less than Z direction, which is relative large in EQ4 and EQ7. This will have a few influences on the overall performance of the structure. However, after an earthquake, the high strength bolt can be relaxed to restore the ICFD to its original position.

ble 10 Maximum residual slippage of the ICFD in ICSCF （mm）

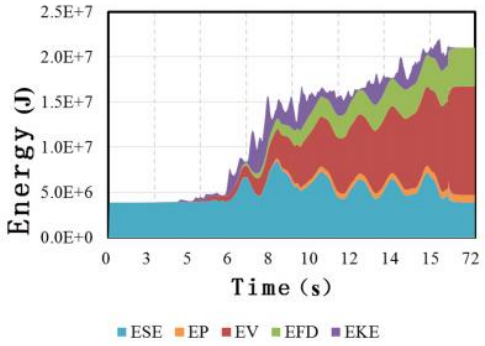
Ground motions	EQ1	EQ2	EQ3	EQ4	EQ5	EQ6	EQ7
Z direction	9.33	6.71	7.66	12.98	6.2	4.62	10.19
X direction	7.94	3.25	5.01	15.92	4.08	6.1	7.02

3.2.5 Comparison of the energy dissipation

Figure 9 shows the energy dissipation comparison of both frames under EQ2. Each symbol represents the following meanings: EKE—kinetic energy, EFD—friction energy dissipation, EV—damping energy dissipation, EP—plasticity energy dissipation, ESE —elastic strain energy. Table 11 shows the energy dissipation values in detail. Under different ground motions, the total energy absorption WK of the ICSCF structure is greater than that of the SCF without EQ6 and EQ7. The ESE of both frames is 12%~39% of WK, EKE is less than 14% of WK, EV is 28%~54% of WK, EP is less than 21%of WK. The EFD of ICSCF can reach 20%~50% of WK that is larger than SCF, which is less than 20%. The above indicates that the ICFD not only improves EFD but also reduces EP, which reduces plastic deformation of structure.



(a) 0.4g ICSCF



(b) 0.4g SCF

Fig. 9 Comparison of enenrgy dissipation of 2 frames under 8-degree rare seismic actions

Table 11 Energy table of the ICSCF and SCF

Ground motions		WK(J)	ESE/WK	EKE/WK	EP/WK	EV/WK	EFD/WK
EQ1	ICSCF	25756100	15.91%	1.70%	1.52%	38.99%	41.74%
	SCF	20229800	21.89%	3.50%	2.26%	53.65%	18.55%
EQ2	ICSCF	22869300	17.97%	3.31%	1.08%	39.68%	37.73%
	SCF	21127200	21.70%	9.81%	3.75%	45.86%	18.74%
EQ3	ICSCF	35140000	11.59%	1.51%	1.58%	38.91%	46.27%
	SCF	26299100	16.91%	2.11%	4.76%	54.14%	21.93%
EQ4	ICSCF	36632700	16.99%	4.67%	2.05%	28.00%	48.13%
	SCF	34077700	38.91%	2.80%	11.38%	30.58%	16.07%
EQ5	ICSCF	22353400	21.18%	2.95%	0.26%	52.00%	23.34%
	SCF	17785800	24.05%	6.90%	1.22%	56.63%	11.04%
EQ6	ICSCF	30357700	13.78%	2.06%	0.95%	43.77%	39.22%
	SCF	53455500	22.02%	0.43%	21.36%	35.84%	20.06%
EQ7	ICSCF	14673100	25.40%	4.63%	0.28%	42.56%	26.81%
	SCF	17541500	36.64%	14.05%	2.31%	35.96%	10.76%

note：WK—the total energy dissipation；ESE—elastic strain energy；EKE—kinetic energy；EFD—friction energy dissipation；EV—damping energy dissipation；EP—plasticity energy dissipation.

Under 8-degree rare earthquakes, the PEEQ for the column bases and beams of both frames is shown in Table 12. The data in Table 12 show that the PEEQ occurs to the most common column base of both frames, and most values of ICSCF is less than SCF. A little PEEQ also occurs to the intermediate column, the frame beam still remains in an elastic state without EQ4. The data indicates that the intermediate column can reduce the plastic deformation of frame column and hinged column.

Table 12 Equivalent plastic strain of the 2 frames under seismic actions

Ground motions		Prepressed column	hinged column
EQ1	ICSCF	1.11×10^{-3}	1.73×10^{-3}
	SCF	8.84×10^{-5}	1.83×10^{-3}
EQ2	ICSCF	1.13×10^{-4}	9.48×10^{-4}
	SCF	6.76×10^{-4}	2.99×10^{-3}
EQ3	ICSCF	1.93×10^{-3}	2.41×10^{-3}
	SCF	2.26×10^{-3}	3.79×10^{-3}
EQ4	ICSCF	1.10×10^{-3}	1.19×10^{-3}
	SCF	9.43×10^{-3}	8.74×10^{-3}
EQ5	ICSCF	0	5.68×10^{-4}
	SCF	0	1.15×10^{-3}
EQ6	ICSCF	1.08×10^{-3}	1.41×10^{-3}
	SCF	2.33×10^{-2}	2.33×10^{-2}
EQ7	ICSCF	0	2.16×10^{-4}
	SCF	0	9.39×10^{-4}

3.3 Dynamic time-history analysis under high magnitude seismic actions

According to the comparison of response to ICSCF and SCF under EQ3(PGA=0.62g, PGA=0.8g) to research the Seismic performance of ICSCF.

3.3.1 Slippages of the friction damper and connection gap opening

Table 13 shows that the maximum slippages of the friction damper of ICSCF in both directions under high magnitude seismic actions (PGA=0.62g, PGA=0.8g). Table 14 lists the maximum connection gap opening of both ICSCF and SCF. According to the table, the slippage and gap opening increase with the increase of PGA. The slippages of ICSCF reached 113mm, the gap opening just reached 12.556mm, which indicates that the friction damper can provide greater friction energy consumption during resulting in a large slippage.

Table 13 Maximum and residual slippage of the ICFD in ICSCF （mm）

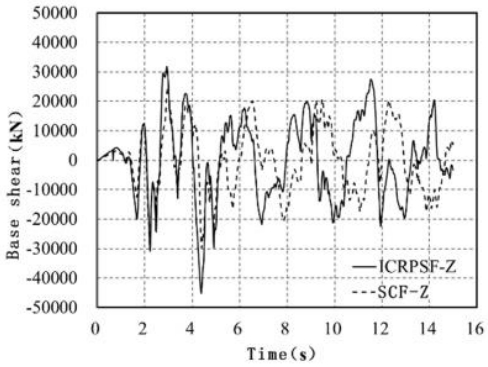
Ground motions	Z direction		X direction	
	Maximum slippage	Residual slippage	Maximum slippage	Residual slippage
0.62g	54.2	9.5	81.8	9.1
0.8g	63.0	13.8	113.0	18.2

Table 14 Maximum gap-opening of 2 frames （mm）

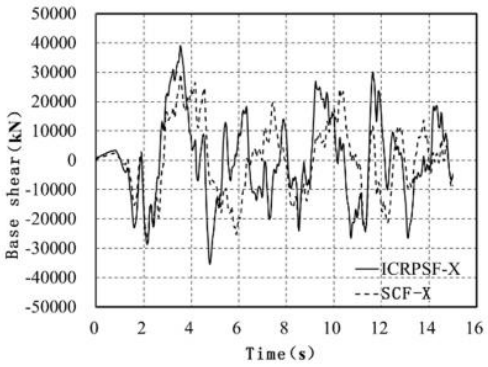
Ground motions	Z direction		X direction	
	ICSCF	SCF	ICSCF	SCF
0.62g	7.426	9.936	5.053	6.366
0.8g	7.856	12.556	6.483	10.154

3.3.2 Comparison of the base shear

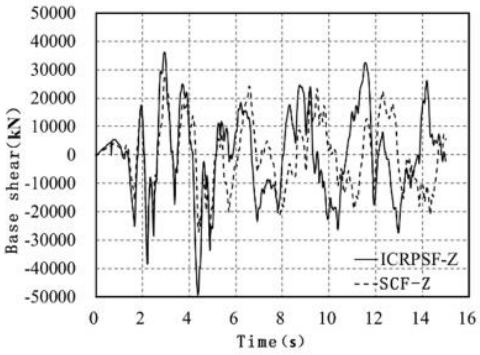
Figure 10 shows the base shear time history comparisons of ICSCF and SCF and Figure 11 shows the the base shear time history comparisons of ICSCF、ICSCF-IC、ICSCF-C and SCF. Table 15 shows the maximum base shears of the ICSCF、ICSCF-IC、ICSCF-C and SCF in the primary and secondary directions under the above seven ground motions



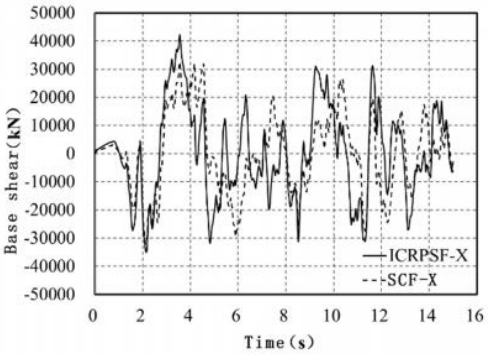
(a) 0.62g main direction



(b) 0.62g secondary direction

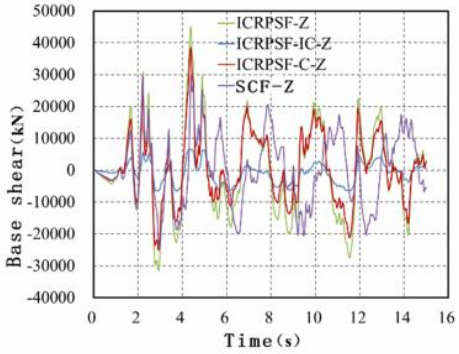


(c) 0.8g main direction

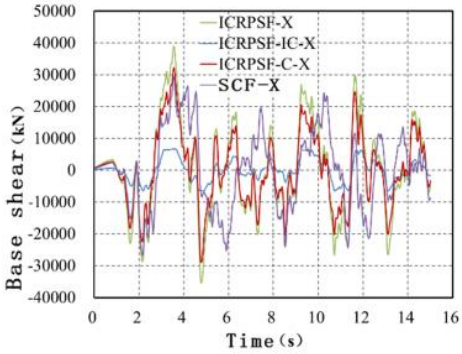


(d) 0.8g secondary direction

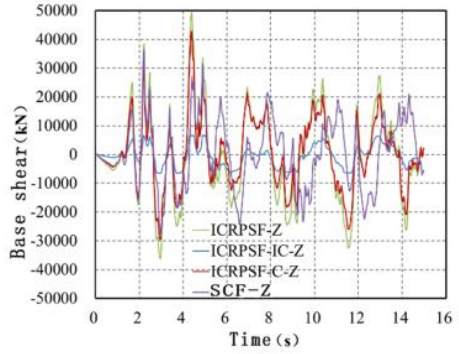
Fig. 10 Comparison of base shear time-history curves for ICSCF and SCF in two directions



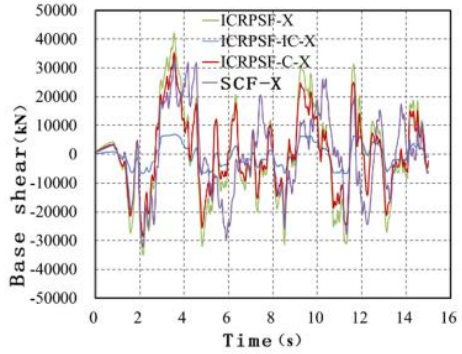
(a) 0.62g main direction



(b) 0.62g secondary direction



(c) 0.8g main direction



(d) 0.8g secondary direction

Fig. 11 Comparison of base shear time-history curves for ICSCF, ICSCF-IC, ICSCF-C and SCF

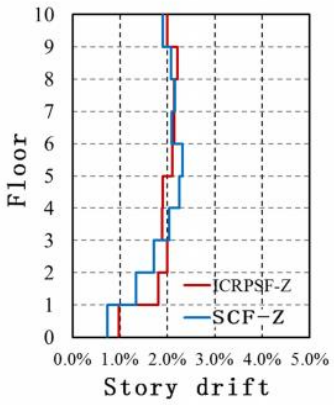
Table 15 Comparison of the maximum base shear of two frames （kN）

Ground motions		ICSCF	ICSCF-IC	IC/ICSCF	ICSCF-C	SCF	增幅
0.62g	main direction	45125.04	6522.14	14.45%	38602.9	29831.8	51.26%
	secondary direction	39021.26	6730.25	17.25%	32298.2	29410	32.68%
0.8g	main direction	49358.14	6683.58	13.54%	42809.3	36398.8	35.60%
	secondary direction	42261.34	6907.49	16.34%	35367.8	32700.4	29.24%

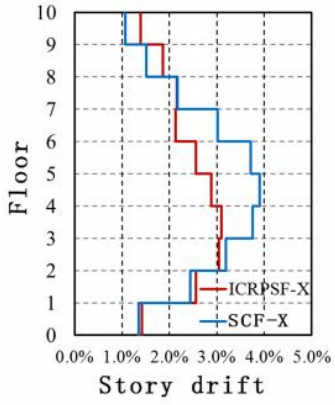
Note: IC/ICSCF means percentage of the base shear of ICSCF-IC/ICSCF

3.3.3 Comparison of the story drift

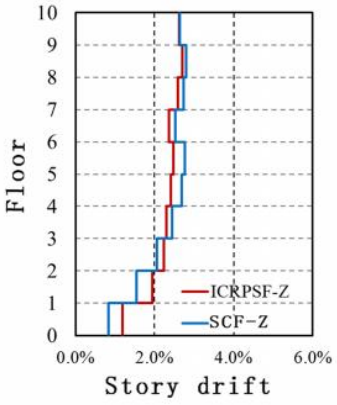
Figure 12 lists the envelope values of the story drifts of ICSCF and SCF in the primary and the secondary directions and the maximum story drifts of each floor are shown in Table 16. According to the data, story drift increases with the increase of PGA.The maximun story drift of ICSCF is less than SCF. The maximum story drift of SCF is 4.807%(1/21), meanwhile, ICSCF is 3.971% (1/25), which indicates the ICFD can prove lateral rigidity of the structure.



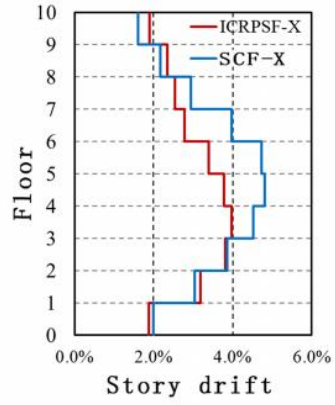
(a) 0.62g main direction



(b) 0.62g secondary direction



(c) 0.8g main direction



(d) 0.8g secondary direction

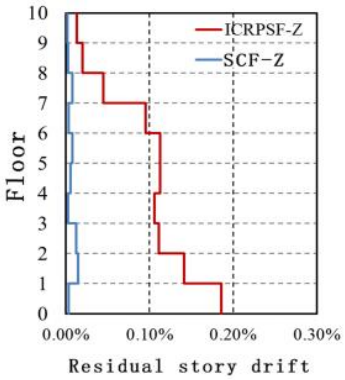
Fig. 12 Comparison of maximum story drift for 2 frames

Table 16 Maximum story drift of each floor for 2 frames
(% rad)

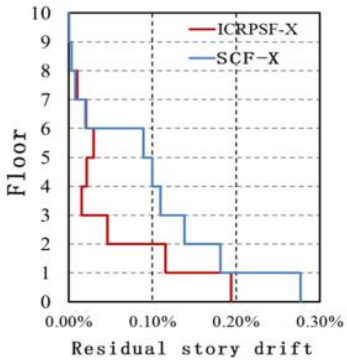
Ground motions		1st	2nd	3rd	4th	5th	6th	7th	8th	9th	10th
Z direction											
0.62	ICSCF	0.985	1.812	2.001	1.886	1.901	2.110	2.141	2.145	2.205	1.997
	SCF	0.736	1.344	1.711	2.043	2.248	2.317	2.090	2.160	2.078	1.901
0.8	ICSCF	1.186	1.949	2.241	2.303	2.419	2.484	2.365	2.589	2.695	2.625
	SCF	0.838	1.542	2.054	2.448	2.681	2.772	2.532	2.732	2.799	2.641
X direction											
0.62	ICSCF	1.398	2.513	2.981	3.008	2.764	2.432	1.991	2.096	1.843	1.377
	SCF	1.353	2.434	3.191	3.750	3.899	3.717	3.016	2.177	1.512	1.067
0.8	ICSCF	1.878	3.185	3.809	3.971	3.772	3.392	2.790	2.547	2.348	1.890
	SCF	1.993	3.047	3.864	4.527	4.807	4.727	3.967	2.948	2.166	1.614

3.3.4 Comparison of the residual story drift

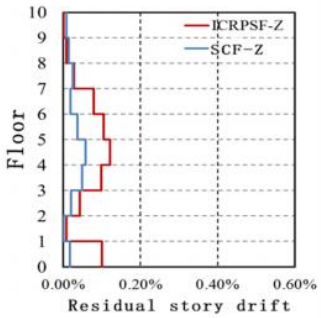
Figure 13 shows the comparison of the maximum residual story drifts between ICSCF and SCF of each floor. The accurate values are shown at Table 17. According to the data, Residual story drifts of ICSCF are larger than those of SCF. Figure 8 appears the residual slippage of friction damper, which shows that residual slippage is about 15% of maximum slippage, however, the high strength bolt can be relaxed to restore the ICFD to its original position.



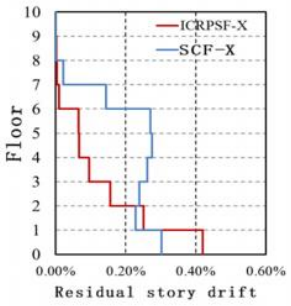
(a) 0.62 main direction



(b) 0.62 secondary direction



(c) 0.8 main direction



(d) 0.8 secondary direction

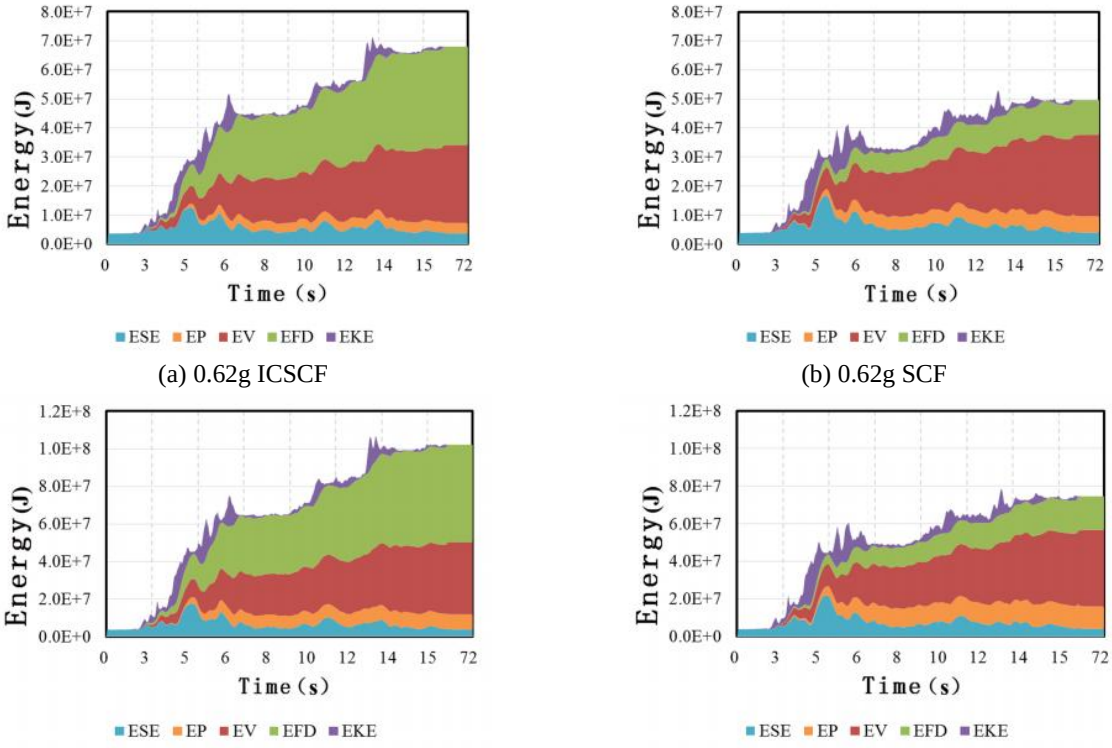
Fig. 13 Comparison of maximum residual story drift for 2 frames

Table 17 Comparison of the maximum residual story drift for 2 frames （% rad）

Ground motions		Z direction	X direction
0.62	ICSCF	0.185	0.183
	SCF	0.014	0.277
0.8	ICSCF	0.121	0.419
	SCF	0.058	0.303

3.3.5 Comparison of frame plasticity and energy dissipation

Figure 14 shows the energy dissipation comparison of both frames. Table 18 shows the energy dissipation values in detail. The WK increases with the increase of PGA and the ICSCF is larger than the SCF. ESE has fallen to 10% and EKE to 5%. The EFD of ICSCF reach 50% of WK that is larger than SCF, which is about 25%,and the EP of ICSCF reachs 10% is a half of SCF. The above indicates that the ICFD not only improves EFD but also reduces plastic deformation of structure.



(a) 0.62g ICSCF (b) 0.62g SCF

(c) 0.8g ICSCF (d) 0.8g SCF

Fig. 14 Comparison of enenrgy dissipation of 2 frames

Table 18 Energy table of the ICSCF and SCF

Ground motions		WK	ESE/WK	EKE/WK	EP/WK	EV/WK	EFD/WK
0.62	ICSCF	68192500	6.22%	1.92%	5.24%	37.01%	49.55%
	SCF	49812900	8.62%	2.92%	11.26%	53.29%	23.71%
0.8	ICSCF	102461000	4.08%	1.77%	7.84%	35.92%	50.34%
	SCF	74899700	5.78%	3.10%	15.97%	51.47%	23.49%

note：WK—the total energy dissipation；ESE—elastic strain energy；EKE—kinetic energy；EFD—friction energy dissipation；EV—damping energy dissipation；EP—plasticity energy dissipation.

The same rule can also be seen from Table 19 that lists the value of PEEQ. According to the data, the PEEQ of hinged column and beam in ICSCF is less than SCF, and the PEEQ of prepressted column of ICSCF is close to SCF.The data indicates that the intermediate column can reduce the plastic deformation of body structure with the increase of friction dissipation capacity.

Table 19 Equivalent plastic strain of the 2 frames

Ground motions		Prepressed column	hinged column	intermediate column	beam
0.62	ICSCF	1.03×10^{-2}	8.54×10^{-3}	1.18×10^{-3}	5.81×10^{-5}
	SCF	9.23×10^{-3}	1.17×10^{-2}	-	1.07×10^{-3}
0.8	ICSCF	1.87×10^{-2}	1.45×10^{-2}	5.18×10^{-3}	1.38×10^{-3}
	SCF	1.76×10^{-2}	1.95×10^{-2}	-	2.58×10^{-3}

4 Conclusion

(1)Compared with the SCF frame, the ICSCF has a shorter period. The existence of the ICFD results in a significant increase of the lateral rigidity and torsional rigidity of ICSCF compared with the SCF. The intermediate column contributes to the overall translational and rotational rigidity of the structure significantly. Additional rigidity provided by the ICFD causes the greater maximum base shear of the ICSCF than that of the SCF. However, The intermediate column will produce a large friction energy consumption under 8 degree rare earthquakes, which plays an important role in protecting the main structure.

(2) The lateral rigidity of ICSCF is higher than that of SCF which contributes to the maximum story drifts of ICSCF are smaller than those of SCF. The maximum residual story drift of ICSCF is also smaller than SCF under the action of 8 degree rare earthquakes. The residual slippage of ICFD is relatively large, however, after an earthquake, the high strength bolt can be relaxed to restore the ICFD to its original position.

(3) The total absorbed energy and friction energy dissipation in the ICSCF are always greater than those of the SCF, while its non-elastic energy dissipation is always less than that of the SCF. Under the earthquake conditions of high earthquake magnitudes, the friction energy dissipation of the ICSCF is larger than SCF. ICFD contributes to the overall seismic energy dissipation of the structure significantly with less plastic deformation.

(4) The relative stiffness difference between ICSCF and SCF reduces with increase of PGA and slippage of ICFD under higher degree earthquake. The stiffiness of ICFD reduces after slippage occuring, and the shear of intermediate column also reduces with increase of PGA, which contributes to control the story drift and dissipate more energy, slow down the plastic development of the components . In conclusion, the resilient prestressed steel frame with intermediate column containing friction dampers is very suitable for the resilient prestressed steel frame structure with a large span, which may realize the target of self-centering after undergoing seismic actions with high magnitudes

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view of Application of BIM in Project Management of Urban Utility Tunnel

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Key words: BIM; urban utility tunnel; project management; full life cycle

Abstract

Utility tunnel can effectively improve the utilization of urban underground space and avoid the re-excavation of urban roads in pipeline maintenance and reconstruction. To introduce BIM technology in urban utility tunnel project can improve the design and construction efficiency and ensure the quality of the project. In the design, construction, maintenance and operation process, this paper summarizes the current application of project management in utility tunnel based on BIM both in domestic and abroad. However, the application of utility tunnel project management based on BIM are still in their early stages. Therefore, this paper also give a rational analysis and prospect to the combination between Utility tunnel based on BIM and some other areas such as laser scanning, cloud technology and QR code technique.

1 Introduction

At present, China's economy and urbanization construction is developing rapidly. Due to the increasing population of the city, the original urban design capacity has been unable to meet the needs of the existing urban population in the rapid development of the city. In the process of urban expansion, it is necessary to upgrade or rebuild the pipeline. In the new planning of urban construction, in order to avoid the later transformation, it is necessary to plan the use and construction of the underground space in advance.

At present, most of the city's municipal pipelines in the latter part of the transformation process needs to be carried out after the excavation of the pipeline construction, it brings not only inconvenience, but also huge property damage to the normal operation of the city[1]. City municipal integrated corridor is an effective way to solve the unreasonable layout of urban underground space and improve the utilization rate of urban underground space. Urban municipal integrated corridor, as an intensive municipal pipeline solution, can effectively avoid pipeline transformation on secondary excavation to the urban road , protection of urban environment, to ensure the smooth operation of urban transport.

BIM technology is based on the three-dimensional model of the construction project, the integrated project life cycle information, data, with its visualization, coordination, simulation, optimization and graph ability gradually in the construction industry to obtain a large number of applications, BIM makes The construction project significantly improves efficiency and reduces risk in its entire process.

Based on the characteristics of BIM and municipal corridor, BIM technology can be applied to the municipal integrated corridor, which can improve the management, production, construction and operation of the municipal corridor, improve the construction efficiency and ensure the quality of the project. The main application process is as follows: Form an environmental model→Create an existing pipeline model→Analysis of Corridor Section Demand→Corridor and Pipeline Prefabricated Split→Factory prefabricated→Prefabricated combination→Site construction→Operation and maintenance management[2] .

2 Research status

2.1 Application of BIM Technology in the Design Stage of Urban Comprehensive Corridor project

2.1.1 3D modeling

In the engineering design phase,Using the corresponding three-dimensional CAD software directly to the municipal corridor project professional design,The three-dimensional design model (BIM) with attribute information is established, The BIM data can be shared between professional designs to avoid duplicate data entry errors.For example, create a template file for use in Revit for the project,Establish the corresponding elevation, import CAD drawings. Along the centerline of the corridor to generate tubular tunnels, and then split the tubular tunnel, the formation of pipe section and related information.

2.1.2 Engineering quantity statistics

According to the BIM model, according to different screening conditions for the management of the project volume statistics, form different types, different models, different periods of time the amount of work,and provide the data basis for the measurement and responsibility cost analysis of the project.In the construction process,It can provide materials in a timely manner, eliminating the cumbersome manual calculation, improve work efficiency. In the Revit model, According to the set parameters and filter conditions, Sub-category, sub-time, sub-location output of the corresponding amount of work, and exported to EXCLE file for engineering measurement and budget.

2.1.3. Analysis and real simulation

You can use the specialized software to automatically check whether the BIM design meets the specification requirements. The BIM data can be automatically imported into various analysis software (eg.energy analysis, green building analysis, sunshine analysis, wind environment), Quickly carry out various analysis and real simulation.

2.2 The currently application of BIM technology in the city integrated corridor project construction stage

2.2.1. Construction simulation

Option 1: Based on the DELMIA software from Dassault, the project will be connected with the construction progress and the construction resources, and then the 4D construction information model will be established. The construction organization plan will be simulated and analyzed in the virtual environment. On the one hand, in the simulation process, identify the irrational and problematic construction method in the design process, and on the other hand, in the process of simulation, the construction method and the construction method of the construction method are analyzed, It is beneficial to timely adjust the construction method before the construction of the project to obtain the best construction plan, optimize the construction resources (personnel, machinery, materials, etc.) configuration, shorten the construction period, save the cost, To achieve the purpose of fine construction[3].

Option 2: In Autodesk Navisworks Manage 2015, the schedule is made into an Excel spreadsheet or Project schedule, exported as a .csv file, using the Timeliner function in Navisworks, the associated duration of the .csv file, attached to the model by name Among them, conduct 4D construction simulation.Related to the ODBC database, you can carry out engineering animation roaming, through the third person perspective, you can roaming the tunnel and ground, and observe the relationship between the form after the completion of the various parts of the project and the correspondence between the tunnel and the ground building[4].

2.2.2 Impact checking

The collision inspection includes the interior of the corridor of the professional collision inspection and corridor with the built or planned underground space, rail traffic collision check, corridor internal collision check. Integrate the pipe layout corridor model, the professional pipeline model and the affiliated facility model, and use Autodesk Navisworks to automatically check the collision between the professional pipeline model and the corridor and the collision between the professional pipelines. Construction unit according to the software generated by the collision report on the BIM platform and the owners, designers to communicate with each other to develop a reasonable pipeline layout program and modify the adjustment model to form the construction design stage of the overall management of the BIM model and process reports, in accordance with the corresponding corridor Information model delivery standards are provided to owners and other participants.

2.2.3. Construction progress management

Based on Dassault's ENOVIA software to build BIM construction management platform. Before the project starts, according to the project implementation organization plan, pre-arranged construction plan, construction process, combined with the project actual situation, the project construction process refinement, in order to facilitate the construction site management, more operational, and secondly, In the construction process, the construction progress of each construction task is tracked and the construction progress management platform is updated to the construction progress management platform. The project leader can grasp the progress status of the project by grasping the update status of the project WBS, and compare the actual construction schedule with the design and quick Deviation analysis, and timely take appropriate measures to achieve the dynamic management of the project construction[5]. In other words, BIM technology can make progress control dependable, data control, project management can accurately control each work for the normal progress of the project to provide protection.

2.2.4 Monitoring and measurement

Open up the BIM construction management and the existing monitoring and measurement platform between the data interface, the integration of existing monitoring and measurement platform, without increasing the workload of the field technician under the premise of the BIM model in real time to view the corridor construction process Measuring data, deformation curve and state. For the measurement of deformation over limit, timely notify the project management staff, for the safe construction of the project to provide a reference, but also for managers to provide the basis for decision-making[6].

2.2.5 Dynamic management of transport information for underground corridor production based on BIM

In the process of splitting the corridor of the components and pipelines named after the identification of the unique identity of the components, the use of RFID and other radio frequency technology, prefabricated components embedded in the chip card, using a handheld scanning terminal scan chip card, you can view the corresponding components all Front operator, operating time[7]. After confirming the current process, the confirmation information will be sent back to the server, the component production information, component quality acceptance information, and the transport location information of the component are synchronized to the BIM model in real time. The system will reflect the production, transportation, Install information, and the relevant information, intuitive feedback to the appropriate management staff hands.

In the management platform can view the real-time factory production, the storage capacity of the warehouse, the transport of components and the current project corridor has been completed installation, for each component can view the components from the production to the construction of the various stages of information , Including component production time, the acceptance of the various stages of the results, admission time, installation time, to facilitate the entire process of integrated information management project.

The specific application process is: to create a component coding a component information, enter the system a print component identification and encoding to the RFID radio card→engineering production planning→embedded scanning entry, stripping scan entry, inspection scan entry, storage scan entry, loading scan entry, unloading scan entry, installation scan entry→synchronization update to the integrated platform.

Especially in the construction process, the project construction staff can scan the components to check the quality of the information components, when the discovery of components should be in the construction of non-standard pipe section, the system can query the non-standard pipe section of the site storage location, so as to avoid the construction of the wrong pipe section led to the late pipeline and the corridor can not be constructed.

2.3 The currently application of BIM technology in the operation and maintenance phase of the city integrated corridor project

2.3.1 Corridor lighting analysis

Corridor lighting is mainly the patio natural lighting with corridor interior lighting, in the corridor design process, you can use the BIM model for corridor lighting analysis, in the Revit software by setting a different time, weather conditions and the power of the lighting equipment to simulate the analysis of the corridor inside the lighting conditions to meet the maintenance requirements of lighting. Combined with the results of simulation analysis of the corridor lighting adjustment and optimization, until the requirements so far.

2.3.2. BIM-based operation and maintenance management

The design, production and construction of all the information, are integrated into the tube project BIM model, the realization of digital integrated delivery, the late use of the completion of the delivery model for the operation and maintenance of the corridor to achieve corridor operation and management of information, electronic and digital[8].

During the operation and maintenance process, the staff can inquire about the relevant information of any component through the model, including the manufacturer of the pipeline, the geometric parameters, the person in charge of the management and the warranty telephone number. After the maintenance and maintenance is completed, Model, as the latter part of the operation and maintenance of management data, but also in the BIM model to each pipeline to set the next maintenance of the time plan, near the maintenance time, the system will notify the appropriate management of the corresponding pipe in the pipe Update maintenance, to ensure the smooth implementation of the late operation.

3 Summary and Prospect

The emergence of BIM technology is a subversive change in the field of civil engineering, the design of the past as the core of the design and construction model to model as the core design.

In the process of comprehensive underground corridor planning, BIM technology core modeling software can be used as a design means, to play its excellent fast and accurate modeling advantages, improve the design efficiency. The design process is the production process of the three-dimensional model, any local details through the selection of different angles of the model cut, you can

get a detailed construction plan. Second, we can play a collaborative working mechanism, through the use of external reference, data shortcuts and object references, etc., to achieve the coordination and communication between professional designers to ensure that the actual map to meet the construction requirements to reduce the construction process changes. The Different professional underground pipe network between the impact of BIM software can be brought by the pipeline collision detection to adjust.

In the process of integrated corridor construction, we can also simulate the construction, taking full account of the roadside buildings, bridges, urban ridge on the late integrated corridor construction and pipe set within a variety of pipe network construction, fully found late construction possible The problems encountered in the process of modeling the existing problems, responsible for all aspects of the design engineers can be resolved through consultation. Through the refinement model to simulate the construction schedule, reasonable arrangements for construction personnel, construction materials, the preparation of reasonable and feasible construction schedule. Through the continuous development of the corridor model, we can use BIM and create the relevant interface to build and refine the original building model, decompose into a small building model with detailed construction data, facilitate the organization of factory production and shorten the construction progress. According to the design unit to provide the building model, the construction information into the building model, the establishment of a comprehensive integrated corridor file. At any time sampling the construction process of the construction data, call out the parameters of the building model to control, to ensure the construction quality.

During the completion of the integrated underground corridor, the BIM model will be fully refined as the construction is completed. A large number of engineering information in place, all transferred to the integrated underground corridor management unit for the late integrated corridor lighting, ventilation, fire and drainage facilities maintenance and alteration to provide infrastructure basic information. This model platform can also provide technical information for some kind of pipeline construction or maintenance in the later integrated underground corridor. Combined with other BIM operations and management software, more efficient to play a comprehensive underground corridor of the advanced design concepts and comprehensive, efficient, economical and practical functions[9].

Underground corridor operation and management enterprises. Should be from the planning, design and construction phase began to set up relevant institutions, can not be organized by the underground pipeline units at random construction, this is not conducive to play the advantages of integrated corridor, resulting in confusion between the relevant units, and related public facilities damage, and even affect To the integrated underground corridor of various pipeline safety issues. BIM model of the use of virtual construction advantages, in the entire corridor to install surveillance cameras to ensure that the underground pipeline without dead ends. Summary, BIM technology has been used in many international mature projects, there are many large domestic or large projects on the application of the success stories; underground integrated corridor as China's large area of the promotion of the project, its design is very forward-looking, so The rational use of BIM technology to the planning, design, construction and post-operation of underground integrated corridors will greatly facilitate the development of the project and provide reliable model information for post-operation management.

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Review of Cloud and BIM-based Collaboration in Construction Management of Subway Engineering

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Abstract: With the development of BIM subway construction, there comes a problem of project coordination. It is hard to deal with data from different major as well as different working step, which means a low efficiency of coordination. As the cloud technology come into use widely, it is an effective way to use cloud platform to deal with the problem of data sharing, for cloud can gather information of all major and the extraction of information is very easy and flexible. Throughout recent research and the exploration of current BIM cloud platform, the weakness in function and framework of BIM cloud platform can be found and make up by some programming. In this way, a new platform which contain the function in whole major of subway construction can be established and the difficulty in coordination is to be solved.

Key Words: subway construction; BIM-cloud; coordination; information sharing

1 Introduction

Subway construction is a complex project which investment is large, time cycle is long, risk is high, technique is difficult and schedule is tight, as well it needs a multi-specialty coordination including architecture, structure, water, heating, electricity and so on[1]. It will be very difficult working in current mode based on 2D CAD modeling to realize the comprehensive cooperation of various specialties. However, BIM (Building Information Modeling) will give us a way for improving our modeling as a new type of architectural design and management concept, throughout the calculation and simulation of information associated with 3D models. BIM have characteristics of high degree of visibility, consistency and completeness of information, which means it can deal with much amount of data as well build more accurate models, provide an entire information of site layout and more optimal solution, and it can realize the real-time management in the project. However, there are still many deficiencies in the existing BIM application in subway construction. BIM includes a large amount of software, models in different software adopt different data formats, so that the model in can not be read freely modified in different software. The existing BIM software presents a higher performance requirement for computers, which means increasing the cost of equipment for companies. For different users, the model transmission often through flash drive or hard drive, which means the speed of transmission is very low and the integrity, security of the data cannot be guaranteed[2]. However, cloud computing will give us a solution. The most obvious feature of cloud is it can gather all kinds of data into the same platform, users can share and acquire information on demand and flexibly. The cloud platform can also break the requirements of the high performance of the computer, and users need only the average level performance of computer to complete the work. Finally, the cloud also realizes the transformation of different data formats, thus makes each specialized information data generalized in a same format, and make the synergy of all specialties of projects in whole life cycle much easier.

2 The advantages of sharing and collaboration in projects using cloud platforms

2.1 Disadvantages and defects of traditional BIM design

Current BIM design in whole process needs to be carried out in a certain order, and it can be seen obviously of the order of work in different specialty. For example, equipment and electrical work cannot get started until the architectural design is determined. This means that the design period is uncontrollable, when the architecture design is modified, other specialty like structure, water, electricity also needs to be redesigned, this can incredibly increase the time of work, even make the actual work period exceed expectations. In addition, the lack of information communication between the various specialty will result in conflicts and more risks between different models, that means extra work time and increase the cost of design as well[3].

2.2 Mode of cloud platform collaboration

Cloud platform can make all kinds of resource centralized. To integrate multiple server in the system can get the function available that normal computer can't realize. In such way, large amounts of data will no longer be an obstacle to BIM development. The cloud platform can integrate all professional work resources. It is a point-

to-face mode that users can acquire resources on demand easily, whereas traditional mode users have to get information in a point-to-point mode, which makes the work more of duplicates. (see figure.2). For a design programme cloud can allow all participants to work simultaneously, which can take into account the views of

all participants, thereby minimizing the error and risk of the programme. The problems encountered during the work of each specialty can be fed back to the cloud platform timely, so that all parties can solve the problem together in time.

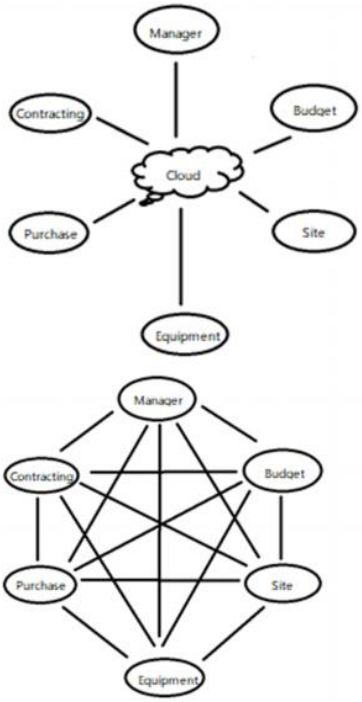

Fig.1 Comparison between mode of collaboration with or without cloud

3 Works of subway construction in BIM cloud platform

3.1 Tasks in whole life span of program

The whole life span of subway project includes planning and design stage, bidding stage, construction and completion stage, operation and dimension stage. The planning and design stage refers to the feasible plan of the project from the initial working to the final process, including on-site investigation, construction method, project design, construction period simulation, cost estimation and final construction blueprint. Bidding stage is based on the planning and design of the project to carry out the calculation and engineering budget, generate the corresponding tender list, and the budget content to be audited. In construction phase, cost, material, equipment and human resources is needed, and it is important dealing with the unexpected incidents that is outside the plan. The completion stage is to carry out the project settlement and the project delivery. The operation and maintenance of the project are included in the operational stage. At the same stage, there are some quantitative relations and steps in different functions, and the work of different phases is interlocking and forming a complete flow. For BIM, the most obvious feature is the model informationization, which models are seen as the main line of the whole life span of the project, the 3D graphics information in the model can be applied to all phases of the project, other information is based on the needs of different stages. Therefore, the management framework of the whole life-span of subway project based on BIM should be constructed around the main line, and the collaborative work of the project is realized at all stages.

Stage	Function needed	Model
Planning	Scheme comparison, Investment estimate	Estimation model
Designing	Drawing design, Budget design, Collision check, Design review	Design model
Bidding	Engineering quantity calculation, Tender List, Contract price, Construction budget, Budget review	Bidding model
Construct	progress control, cost control, Change management, equipment management, Material management, Human resource management	Construction model
Completing	Engineering settlement, Project delivery	Completing model
Maintenance	Operation control, Operation monitoring, Maintenance management	Maintenance model

Chart 1 BIM based subway engineering life cycle management framework

3.2 Schema of subway engineering cloud platform service

The construction of subway project involves a number of specialty division of labor and cooperation, at the same time to the construction site information real-time feedback to put forward a higher request. Therefore,

the design of BIM Cloud platform should consider the characteristics of multi-professional, real-time and coordination. This paper proposes a system framework of BIM cloud platform, including function output layer, platform service layer, data management layer and information gathering layer. Platform service layer is the core of cloud platform framework, as the hub of information input, information processing and output. The functional output layer is the information needed to generate the user after processing the information, such as 3d browsing, information management, cost management, progress management, etc. The Platform service layer is processing the collected data and generates the result data corresponding to each function. The data management layer is the data distribution center, where the external equipment or the raw data of manual input is storage and managed, easy to call for use of platform services. There are two ways to gather information: the one is through external devices, and the other is through manual input, the information of both methods is transmitted to the data management layer. This platform architecture with "cloud Platform service" as the core can satisfy the information gathering, processing and real-time interaction of the project in the whole life cycle, and it also puts forward a high requirements for the hardware and software of the cloud platform. The platform needs both sufficient hardware and software performance. For hardware, the security of data should be ensured. For software, the interface and operation should be optimized according to the actual need of users. In addition, the platform's operation needs to be perfectly fused with the field sensor equipment to ensure that the site information is fully acquired.

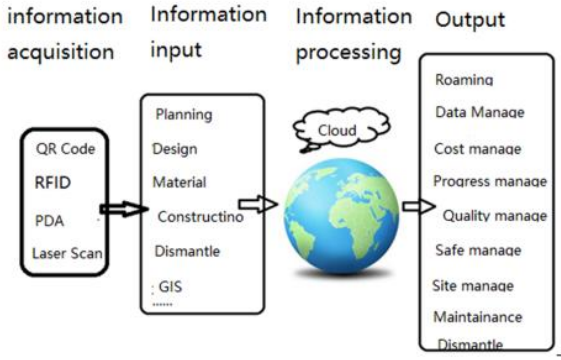

Fig. 2 Schema of subway engineering cloud platform service

4 Comparison of the features of the existing BIM cloud platform

The existing cloud platform can be classified into two categories according to the application mode: one is stand-alone platform, one is secondary development platform. Stand-alone platform is the kind that the project construction of most of the functions is implemented through a complete set of system, such as Autodesk360, Microsoft Projectwise; Development platform is the kind which its function is attached to BIM modeling software, it use plug-ins instead of complete set of system, such as Hung Ye Bim and the Yi Zhu BIM, both of them are plug-ins attached to Revit. For stand-alone platforms, its function is relatively complete, the file is available in whole function of the platform, but when meeting the challenge that function is not supported by the platform, it will be a great difficulty because other platform cannot read the file. For development platform, it can use model from BIM software directly and it can eliminates the repetitive modeling process. However, function is limited because of the amount of plug-in is limited, so that some complex function will not be well supported, and there are many defects in model recognition as well.

Software	Type	Function				
		how	Attribute	Amount	rash	Modify
Archicad BIMx	Platform	√	√	√	√	×
Bentley	Platform	√	√	√	√	×
Autodesk 360	Platform	√	√	√	√	×
Dassault	Platform	√	√	×	√	×
Glodon	Platform	√	√	√	√	×
Luban	Platform	√	√	√	√	×
Hongye	Plug-ins	√	√	√	√	√
Yizhu E-BIM	Plug-ins	√	√	√	√	√
Pengyu Cheng	Plug-ins	√	√	√	√	√

Chart 2 Comparison of the feature of existing BIM cloud platform

As can be seen from the above chart, the existing BIM cloud platform mostly supports 3d display, attribute view, calculate and statistic. For independent platform, basically supports model collision check function, but for modeling modifications it cannot be directly modified in the cloud platform, that means users still need to install modeling software to operate. This indicate that BIM

cloud platform still have many work to make improvement.

5 Further development of BIM Cloud Platform based on subway project

Data formats between the existing BIM cloud platforms also require complementary compatibility. Secondary development can utilize the function of different cloud platforms to realize complementary, solve the information sharing problem between different platforms simultaneously, and it will make BIM cloud Platform available for the model modification. WebGL can realize the preview of the model in the cloud and the function of object property management, the principle is to make the BIM model and property data to generate the HTML recognizable data format according to certain algorithm, and finally realize the display on the network side of the model and its property. Hadoop technology can be used as the core of cloud Platform data management, as it is relatively a mature open source system of cloud computing, it can meet the challenge of massive data retrieval and storage System Management, in a reliable, scalable and the high efficiency way, its cluster of server can expand to tens of thousands. It is especially suitable for subway construction. ThinkerCAD is an online model editing platform developed by Autodesk company, which is originally designed for 3d printing services. The model can be edited directly on the network side by ThinkerCAD. The BIM model can be converted into ThinkerCAD data format by using the transformation tool, so that the BIM model is modified directly on the network side.

The IFC (industrial file base standard) format is an important general format standard for model data interchange, and is also an important intermediate file for BIM models in network breaking. While BIM model being preserved in the IFC format, then the IFC format being converted into obj format, the display of the BIM model can be realized in the webpage. Other commonly used modeling software such as SketchUp, Archicad, AutoCAD can generate into IFC format. Therefore, this method can realize the process of transforming the BIM model to the cloud platform in a larger scale.

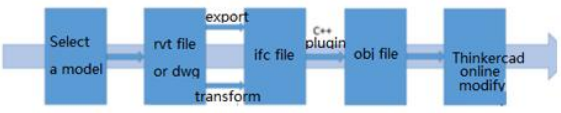

Fig.3 Method for realizing web page preview by Revit model

6 Conclusions

The conclusions are: (1)BIM cloud platform can solve the problem of format universal, which provide a better collaboration quality in subway construction. (2)BIM cloud platform can also achieve many functions which cannot realized only by personal computer, for it integrates a large number of hardware which performance can catch up with thousands of computers. (3)Through the platform, many construction achievement can be saved and use again when necessary. (4)It is good for subway construction as the progress of design and management will be more intelligent and automate, which can improve the efficiency and lower the cost.

In the future, The development of subway engineering and the combination of BIM and clouds can exert a strong productivity, and the future integration of subway engineering and more technology can revolutionize the design and management. At the same time, the BIM cloud will be used in the construction of subway engineering in all the functions of life[4] , especially the operation and maintenance stage[5] . The degree of data sharing among the professions is obviously improved, and the information is exchanged in real time, and the level and efficiency of collaborative management can be enhanced as well[6] . The use of lightweight model display technology can ensure the reduction of the computer hardware requirements on the premise of display effect. And through cloud computing, the pervasive computing of intelligent analysis based on BIM model enables users to focus on business rather than data processing[7-9] , as well as a storage platform for model and engineering libraries, to form the BIM subway project "Knowledge Network" as a template and reference for future engineering design[10-12] . BIM will also integrate with other high-tech, further increase the breadth and depth of synergy, such as through the wireless equipment, virtual reality, geographic information system to achieve more automated construction and management, to promote the construction of subway Engineering Intellectualization[13-14] .

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Safety and Reliability Evaluation of Ancient Wood Structure Based on Dynamic Method

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Keywords: dynamic method, ancient wood structure, reliability

Abstract: Chinese ancient buildings with wood-based architecture enjoy a high reputation in the world. Its long history, the vast amount of complex structure, scientific and rich cultural connotation, and good seismic performance are very famous. Wood structure system is the main part of ancient architecture of China. However, the used wood of ancient wood architecture in the service process occurs decay, insects, cracking, and other aging problems, which will be resulting in attenuation of physical and mechanical properties of wood, eventually leading to the occurrence of damaged wood members, seriously affecting on the use of the structure, and some even leading to structural damage. Most damage of ancient buildings originated in wood members damaged. Currently, the evaluation of how the security architecture of ancient wooden structure, reasonable renovation program has become a common concern. Therefore, the durability of ancient architecture and evaluation of damage is important. This paper will use a dynamic method to test and evaluate the durability analysis of ancient architecture of ancient timber buildings in the service environment. It is can provide the basis of design and construction of repair of ancient buildings. It is also for scientific and systematic protection of the ancient buildings.

Conference topics: 2.Structural mechanics and technologies

1. Introduction

Chinese ancient buildings with wood-based architecture enjoy a high reputation in the world. Its long history, the vast amount of complex structure, scientific and rich cultural connotation, and good seismic performance are very famous[1]. Wood structure system is the main part of ancient architecture of China. However, the used wood of ancient wood architecture in the service process occurs decay, insects, cracking, and other aging problems, which will be resulting in attenuation of physical and mechanical properties of wood, eventually leading to the occurrence of damaged wood members, seriously affecting on the use of the structure, and some even leading to structural damage[2-3]. Most damage of ancient buildings originated in wood members damaged. Currently, the evaluation of how the security architecture of ancient wooden structure, reasonable renovation program has become a common concern[4]. Flexural elastic modulus of wood refers to the ratio of stress and strain in the elastic phase when the wood is bent by load. And the elastic modulus of wood is proportional to the bending strength, so it is important to determine the elastic modulus of wood. Zhang Hongjiang et al [5] through the formation of hanging test system experiments were made in the use of dynamic method to measure the elastic modulus. The supports of the vibration signal and test results have no effect on the conclusions. The results show that the dynamic elastic modulus obtained by the vibration method is positively correlated with the static elastic modulus and the bending strength [6]. Lin Lanying, measured the elastic modulus of various methods. It was proved that the bigger density, the better elastic modulus, which is consistent with one of the results of this experiment [7]. At present, the method of rapid determination of elastic modulus is less. In this experiment, the elastic modulus of the specimen is measured at the frequency of vibration. The method has the advantages of small drawing amount and convenient operation. It was explored the wood cracks and other common wood defects on the elastic modulus of wood impact, which found the relationship between wood defects and elastic modulus.

Therefore, the durability of ancient architecture and evaluation of damage is important. This paper will use a dynamic method to test and evaluate the durability analysis of ancient architecture of ancient timber buildings in the service environment. It is can provide the basis of design and construction of repair of ancient buildings. It is also for scientific and systematic protection of the ancient buildings.

2. Experimental

In this experiment, the Xing'anling larch specifications with different types of defects and no defects are selected as experimental samples. Before the experiment, the samples are placed in constant temperature and humidity control, to its moisture to its equilibrium moisture content, about 12%.

2.1 The elastic modulus of Wood tested by dynamic method

Elastic Modulus of Wood is tested by dynamic excitation method. This experiment is used solid material elastic modulus tester by the China Building Materials Certification and Testing Center developed. The mechanical exciting on the specimen are achieved by automatic excitation or manual hammering. The method utilizes a pulse actuator to energize a rectangular cross-section beam sample to measure the bending or twisting frequency of the sample. The vibration of the sample is obtained by the signal receiver at the top of the sample, and the few frequencies of free vibration are obtained by fast Fourier transform. First, the elastic modulus of the specimen is calculated by the fundamental frequency of the bending vibration. The shear modulus is calculated using the torsional vibration main frequency. Since the fundamental frequency of the free vibration of the beam specimen is uniquely determined by the sample size, modulus of elasticity and sample quality, the elastic modulus can be calculated when the fundamental frequency has been measured. The elastic modulus depends on the bending response frequency.

The formula for calculating the bending modulus of the rectangular section beam is:

$$E = 0.9465 \frac{mf_f^2}{b} \left(\frac{L}{t}\right)^3 \left[1 + 6.585\left(\frac{t}{L}\right)^2\right] \quad (1)$$

(Where m is the sample mass, ff is the bending response frequency of the specimen, Hz; L, b, t are the length, width, thickness of the sample, mm)

2.2 The natural frequency of Wood tested by dynamic method

The experimental samples are the wood beams of rectangular cross-section. The cross-sectional dimensions of wooden beams are 26.77mm*40.68mm. The wood beams vibrate free ly on the sponge. The wood beams are free to vibrate on sponge. The hammer method based on variable time base technique is used to Modal testing. Due to the small size and light weight of the structure to be measured, the non-contact sensor-sound pressure sensor was used as the response and the small elastic hammer MSC-1 was used to excite the samples. By excited vibration, the natural frequency and mode can be obtained by the instrument.

3. Results and discussion

3.1 The Dynamic Elastic Modulus Results

Dynamic Elastic Modulus Test Results Value of Non-Defective Sample are shown in Table 1. According to the literature[8], the elastic modulus of larch is between 8-12Gpa. From the results of Table 1, elastic modulus test value of this experiment is also in this range.

Table 1 Dynamic Elastic Modulus Test Results of Non-Defective Sample

Sample No.	Width / mm	Height / mm	Length / mm	Mass / g	Elastic Modulus / GPa
1#	30.25	39.35	201.4	100.91	10.19
2#	27.08	42.08	201.3	108.74	11.14
3#	27.07	42.17	201.3	108.45	11.09
4#	26.89	42.38	201.3	99.45	10.58
5#	27.04	41.92	209.7	111.78	12.95
6#	30.05	40.5	201.1	103.63	9.98
7v	30.21	40.43	200.1	115.08	8.42
8#	30.03	40.25	201.5	100.49	9.37
9#	29.99	40.27	201.0	114.39	9.10
10#	36.50	67.02	202.0	206.99	9.06

In this paper, the effects of crack and moth on elastic modulus are experimentally analyzed. In this experiment, the effect of cracks on the elastic modulus of wood was studied in two parts. First, the length of the crack formed on the wood was measured, and then the elastic modulus was measured. This was compared with the wood without cracks. The effects of cracks on the elastic modulus of wood were studied. Second, using the relevant tools to destroy the wood surface, and then the elastic modulus of wood after the destruction was tested. The specific method is to destroy the surface of the test

Sample 1#, 8#, 9#with the knife. In the process of destruction, there were no mass losses. Using the same method to test their elastic modulus, measured data are shown in Table 2. According to Table 2, it can be found in the elastic modulus will increase with the surface cracks decreased.

Table 2 Dynamic Elastic Modulus Test Results of Sample with cracks

Sample No.	Elastic Modulus Before the destruction/GPa	Elastic Modulus Before the destruction/GPa	Decline/ %
1#	10.19	9.95	2.36
8#	8.94	8.42	5.82
9#	9.10	8.7	4.4

Moth attacking is also a factor affecting the elastic odulus of wood. In this experiment, moth attacking was also carried out a simulation through the opening groove. The sample NO.2# was opened groove along the length direction. The sample NO.4# and NO.5# were opened groove along the width direction. The dimensions of all grooves are width of 7 mm and depth of 3 mm. After testing, the data was shown in Table 3. We can see that after the destruction of the elastic modulus of the sample are reduced, the decline nearly 10%.

Table3 Dynamic Elastic Modulus Test Results of Samples with Moth attacking

3.2 The Dynamic natural frequency Results

In this experiment, the natural frequency and its modal of wood beams are mainly tested. The natural frequency of the wood beams in different circumstances with good condition, a crack, two cracks, was measured and compared in Fig.1, Fig.2, and Fig.3. The results were shown in Table 5. The results show that the crack will cause the main frequency of the wood beam to decline. especially Wood beam modal test results in different condition are shown in Fig.4 and Fig.5. From Fig.4 and Fig.5, the higher modal and frequency change is more obvious with cracks.

Table 5 Comparison of wood frequency changes in three states

Wood State Frequency	**Frequency** 1/Hz	**Frequency** 2/Hz	**Frequency** 3/Hz	**Frequency** 4/Hz
Intact	153.32	761.328	1183.98	1671.09
1 crack	148.633	750.977	1176.37	1655.08
2 cracks	146.094	741.797	1174.80	1633.79

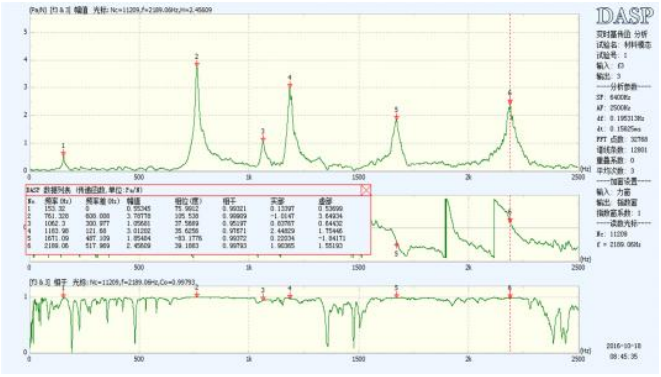


Fig.1 Frequency of wood beam in good condition

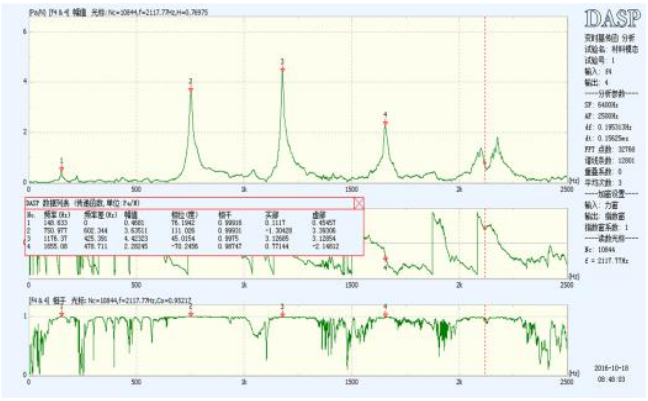


Fig.2 Frequency of wood beam with 1 crack

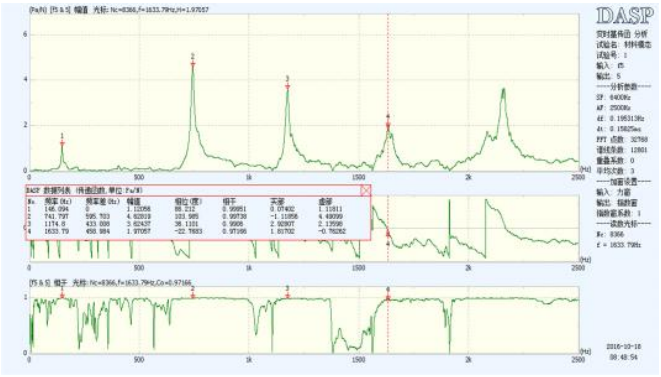
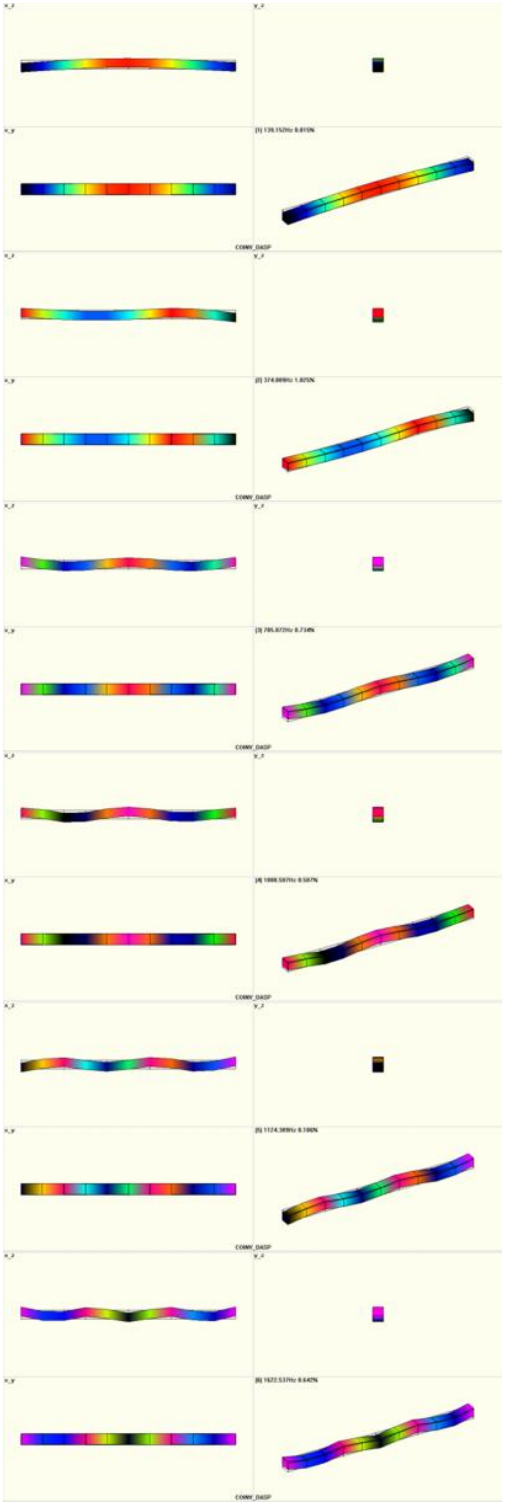


Fig.3 Frequency of wood beam with 2 cracks

从□1,□2，□3中比□看出可以看出**Frequency of wood beams的□化**



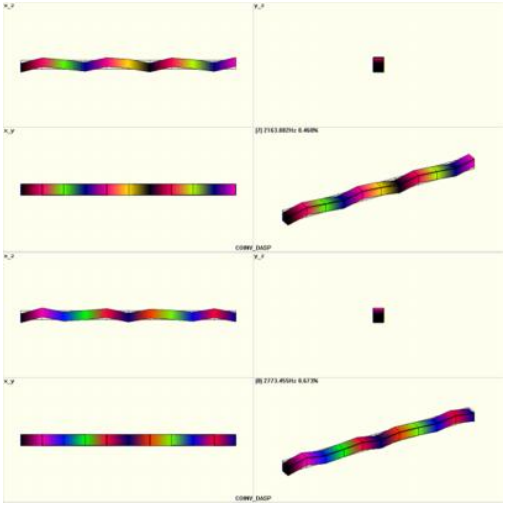
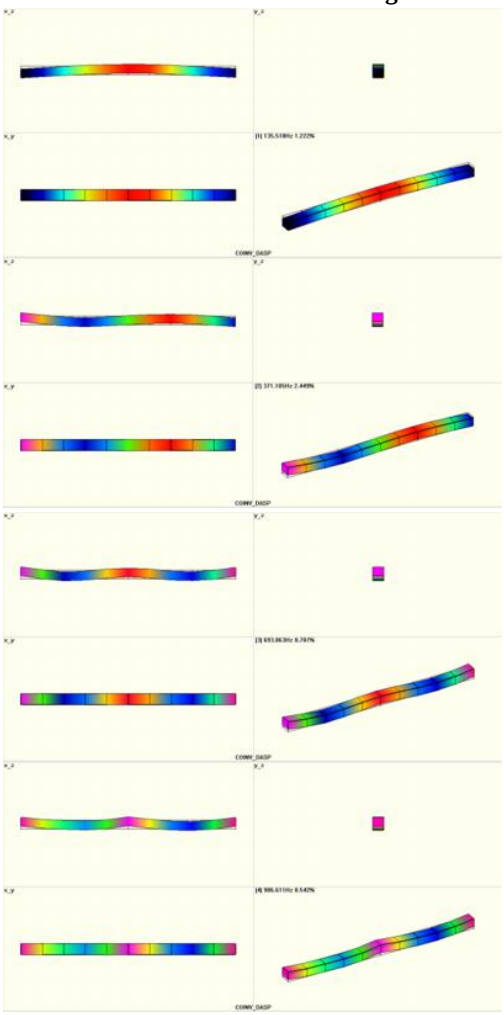


Fig.4 Wood beam modal test results in good condition



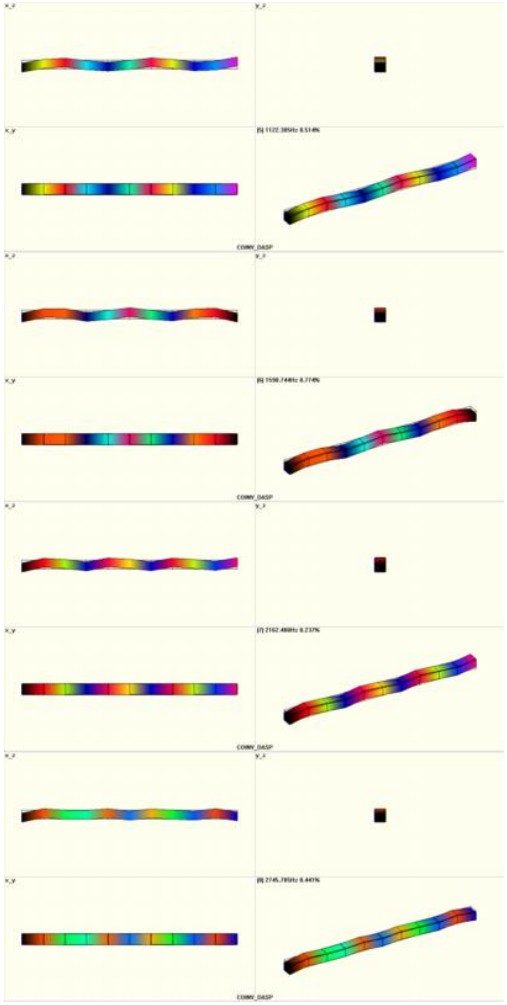


Fig.5 Wood beam modal test results with a crack

4. Conclusions

(1) The elastic modulus test of the ancient building wood can be carried out using the dynamic method of nondestructive testing. Cracks can affect the change in elastic modulus. The elastic modulus will increase with the surface cracks decreased.

(2) By simulating the effect of the moth on the elastic modulus, the Moth attacking can cause the elastic modulus of the wood to decrease, the decline nearly 10%. So the elastic modulus can be easily detected by the dynamic elastic modulus method.

(3) By testing the effect of the crack on the frequency and modal, a conclusion is that crack will cause the natural frequency to decrease. Therefore, the dynamic method can test the natural frequency of Chinese ancient architecture wood structure, and can be used to monitor the safety and reliability of ancient buildings.

Acknowledgments

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STUDY ON THE VIBRATION RESPONSE OF THE EXISTING HIGH SPEED RAILWAY TUNNEL WITH CAVITY DEFECTS IN LINING

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Key words: high-speed railway; train load; cavity behind lining; numerical simulation

Abstract The typical characteristics of the disease in the existing high speed railway tunnel - cavities behind the lining, numerical simulation analysis using the finite element software ABAQUS to study the influence of containing cavities behind the lining of the tunnel structure under the vibration load of train. Both in the vault in empty and void size change that the high speed railway tunnel dynamic response analysis. The results show that the vault area of different sizes appear empty, with the increase of the void size, the lining stress more unfavorable, and the train loads in the empty part of the corresponding structural dynamic amplification. Results to reveal the dynamic mechanical behavior of the existing high iron tunnel under the vibrating load, the it is of great significance to improve the ability of high iron operation safety coefficient and disaster prevention and mitigation.

1 Introduction

China is a mountainous country, mountains which accounted for part of China's total land area of about 70%, due to the factors caused certain difficulty for China's transportation construction, in order to improve the traffic and improve the railway speed, in the construction of high-speed railway tunnel when you need to build a wide range. So since 2005, high speed China railway construction speed is increased rapidly, and the construction of high-speed railway tunnel surge in Mengjin. However, according to China's 5800 seat tunnel in service inspection survey most in service tunnel due to technical level construction, affect the operation and maintenance of geological conditions or late factors, 93.3% of the existing tunnel disease problems in different degree, the disease is mainly manifested in the cracks of lining structure, lining seepage Water Leakage, cavities behind the lining, lining containing holes, survey and design is not reasonable, construction is not standardized. This problem The existence of these diseases seriously threatens the service life of high-speed railway tunnel and the safety of train operation[1].

Wang Chunjing, Lei Mingfeng and other [2]~[4] from less than the thickness of the lining, lining the cavity behind other aspects of numerical simulation and analysis of the safety performance of the structure, the research of tunnel lining structure existing force structure holes changed obviously, threatening the bearing capacity of structure safety. Liu Yonghua[5]is simulated and analyzed in the lining thickness and when there are cavities behind the lining structure stress, the structural reliability index, the different depth is the thickness of the lining has a critical value. When less than this critical value, will adversely affect the bearing capacity of the structure. Liu Haijing, summer only elementary[6] geometry from the empty set of different, rectangular and half round two hole shape, that hole geometry on stress influence structure is not obvious. She Jian, He Chuan [7]~[8] et al in tunnel of a highway in Chongqing as an example, through the indoor 1:2 Model test 5, study in the surrounding conditions are not the same, the empty position is not at the same time, the stress situation and degradation mechanism of the tunnel structure. The study shows that the worse surrounding structure, bearing capacity is weak; the level of pressure, the crown is hollow, with the increase of pressure, cracks appeared in the hole and edge spandrel position; vertical loads, the most serious harm is empty of medium size. We can find that the research on tunnel lining structure holes are concentrated in the static load, numerical simulation, model test, considering the influence of high speed railway vibration load is less, the roof structure of tunnel site there are holes of different sizes, and then fully considering the interaction of concrete

damage, the load of the train in the tunnel structure, the dynamic response of tunnel.

2 Establishment of damage model

2.1 Establishment of concrete damage model

In this paper the tunnel structure model of surrounding rock and concrete lining is one of the diseases in the presence of complex stress state of the tunnel structure, tensile stress exists on the lining structure, and compressive stress, so as to define the damage variable is a double scalar damage variable tension, expression of concrete damage the damage formula can better, this paper uses the Huang Juan[9] established damage evolution equation, starting from the point of view of this study, the damage formula can be well considered concrete and surrounding rock body form. Considering the equivalent ideal material strain stress and effective stress caused by the strain of ideal material, namely damage the model of the relationship between stress and strain can be effective ideal material stress conversion. Because in the engineering of equivalent strain assumption will be able to meet the requirements, so directly in this paper Strain energy efficiency:

$$\begin{cases} \varepsilon(\sigma, d) = \dfrac{\sigma}{E} \\ \bar{\varepsilon}(\bar{\sigma},0) = \dfrac{\bar{\sigma}}{E_0} \\ \bar{\varepsilon} = \varepsilon \end{cases} \quad (1)$$

Damage evolution equation:

$$\begin{cases} \partial d = \dfrac{\varsigma_i}{\varepsilon_{ei}} \left[\dfrac{\varepsilon_e}{\varepsilon_f}\right]^{[\varsigma_i/(1-d_0-\varsigma_i)]} \partial\varepsilon_{ei} \\ d = \dfrac{\varsigma_i \varepsilon_f (1-A_i)}{\varepsilon_{ei}^2} + \varsigma_i \dfrac{A_i B_i}{e^{[B_i(\varepsilon_e-\varepsilon_f)]}} \partial\varepsilon_{ei} \end{cases}$$

$$0 \le \varepsilon_e < \varepsilon_f$$

$$\varepsilon_f \le \varepsilon_e \quad (2)$$

Where: ς is material parameters; ε_e is equivalent strain; A_i, B_i according to the concrete pull and pressure to determine the material parameters.

According to [10] research literatures, the D-P criterion is a criterion used in the study of concrete, it can well describe the concrete failure characteristics. In the long cycle under train load, the lining structure damage expansion, structure stress decreases, it is necessary to consider on the yield surface and damage of concrete correction.

Modified D-P criterion formula:

$$F = \sqrt{\alpha^2 k^2 (1-d)^2 + \overline{J_2}} + a\overline{I_1} - k(1-d) \quad (3)$$

Where: $\overline{I_1}$ is first stress invariant; $\overline{J_2}$ is second stress invariant; α, k, a is parameter.

Which:

$$\overline{I_1} = \overline{\sigma_1} + \overline{\sigma_2} + \overline{\sigma_3} \quad (4)$$

$$\overline{J_2} = \frac{1}{2}\overline{s_{ij}}\,\overline{s_{ij}} \quad (5)$$

$$\overline{s_{ij}} = \overline{\sigma_{ij}} - \frac{1}{3}\overline{\sigma_{kk}}\delta_{ij} \quad (6)$$

$$\alpha = \frac{2\sin\varphi}{\sqrt{3}(3+\sin\varphi)} \quad (7)$$

$$k = \frac{6c\cos\varphi}{\sqrt{3}(3+\sin\varphi)} \quad (2\text{-}8)$$

$$a = 1 - \frac{\alpha f_t}{k} (0 \le a \le 1) \quad (2\text{-}9)$$

Where: $\overline{s_{ij}}$ is partial effective force tensor; c is cohesive force; φ Internal friction angle; δ_{ij} $i = j$, $\delta_{ij} = 1$; $i \ne j$, $\delta_{ij} = 0$.

2.2 finite element model

The tunnel structure is simulated by using ABAQUS finite element software, the first release of stress, then the simulation of excavation, excavation and then added the initial stress in the lining; then invert correctly applied basal train vibration load, analysis of the dynamic response of lining structure containing voids, we need to extract data.

Calculation procedure:

(1) the establishment of three-dimensional model of surrounding rock, the surrounding rock around the tunnel were 4 times of the diameter of the tunnel, about 50m, thickness of upside soil tunnel from actual field detection section thickness 35m, tunnel base with 30m. tunnel depth vertical thickness range of 50m calculation, the rock properties defined by C4R8; mesh, mesh unit, number 4116, the number of nodes is 32928;

(2) the calculation of ground stress equilibrium under the initial condition of surrounding rock;

(3) excavation simulation, the excavation of the first phase of the release of the ground stress in the application of the lining structure (this part in order to facilitate the simulation, the structure of the lining of a

simple calculation of the thickness of the lining of the two times without considering the initial lining) and then release the of 40%;

(4) the physical properties of the material are given in table 2-1, which defines the viscoelastic dynamic boundary conditions, the train vibration load spectrum calculated by the given formula, the train vibration load, the grid partition, and the calculation.

Table2-1 Physical properties of the material

	Unit weight (kN/m3)	E(GPa)	Poisson ratio	Internal friction angle (°)	Cohesive force (MPa)
IV grade surrounding rock	21	2.7	0.2.8	35°	0.3
Secondary lining	25	30.8	0.2		

2.3 train vibration load determination

Train vibration load is caused by vehicle itself factors (such as train, rail, axle load itself) track geometric irregularity, train speed and so on. And the train load is determined, based on literature is theoretical analysis and field testing. The effect is now widely recognized based on an empirical formula for beam can be [11]~[12] et al. In considering the wave train vibration load on the cause of formation and fully considering the wheel rail force on the line and keep moving in a certain area of memory in the stack and the rail material dispersion and rail surface geometry factors on the vibration caused by track irregularity problems, the former has summed up the train vibration load. Vibration formula is modified, the modified formula for:

$$P(t) = k_1 k_2 (P_0 + P_1 \sin\omega_1 t + P_2 \sin\omega_2 t + P_3 \sin\omega_3 t) \quad (10)$$

Where: P_0 is vehicle static load; P_1, P_2, P_3 is vibration load corresponding to the three parameters in the geometric irregularity,(kN), See table 2-2; k_1 —— The superposition coefficient is usually 1.2~1.7, this paper takes 1.5; k_2 ——The dispersion coefficient, usually in the range of 0.6~0.9, is taken as 0.8.

Train quality M_0, Then the vibration load corresponding to different geometric irregularity control conditions is:

$$P_i = M_0 \alpha_i \omega^2 \quad (11)$$

Where: α_i is (i=1,2,3) Corresponding to the vector height of three geometric irregularities in the table 2-2; ω_i ——The corresponding vibration frequency in three kinds of geometric irregularities.

The formula of the circular frequency in 11 formula is:

$$\omega_i = 2\pi \frac{v}{L_i} \quad (12)$$

Where: v is vehicle speed（km/h）; L_i is wavelength corresponding to three geometric irregularities（m）.

In this paper, the calculation of the geometric irregularity of the three control conditions is based on a large number of experiments in the United Kingdom summed up the 200km/h on the train speed in the track geometry irregularity management standards, see table 2-2.

Table 2-2 Geometric Irregularity Management Standards (UK)

Control condition	wavelength (m)	Versine (mm)
According to driving stability	50	16
	20	9
	10	5
Dynamic additional dynamic load on line	5	2.5
	2	0.6
	1	0.3
Waveform wear	0.5	0.1
	0.05	0.005

According to the actual field test showed that the load of the train in the tunnel section duration is about 2S, so the analysis is not set, is set to 2S in accordance with the train vibration load time. The calculated formula of 2-10 foundation 300km/h speed train vibration load curve of figure 2-1.

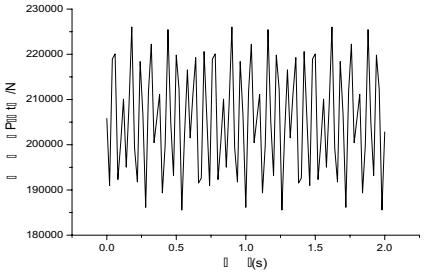

Fig.2-1 300km/h train vibration load time history curve

3 Dynamic response analysis of tunnel lining vault

According to the above a series of analysis to establish the finite element model of tunnel was simulated and analyzed. The requirements of the model parameter is selected according to table 2-1, local dynamic boundary load dynamic simulation can be better under the infinite boundary by boundary conditions. The tunnel cross-section form by finite element simulation based on V standard grade rock section, set three different conditions of lining containing holes, hole size according to the actual detection of lining thickness is 10,20,30cm, the length is set to 2m, see table 3-1. tunnel lining structure section size, cavity size and mesh as shown in figure 3-1~3-2.

Table 3-1 The lining structure contains a void size setting

working condition	Tra in travel direction	Tran sverse width	Verti cal thickness of section
working condition 1	2m	30°	30cm
working condition 2			20cm
working condition 3			10cm

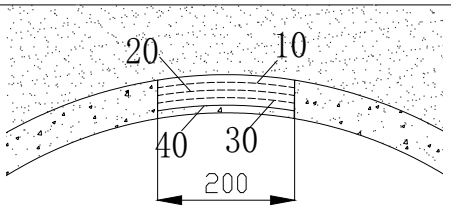

Fig.3-1 Lining thickness setting in the vault position

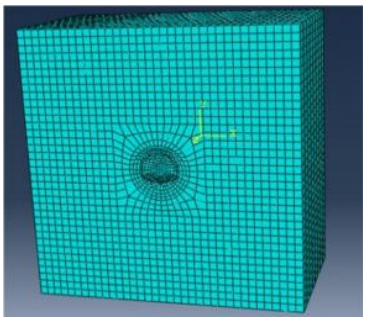

Fig.3-2 Model grid diagram

3.1 analysis of structural dynamic stress under different conditions

The calculation of the stress time history curve of the top of the lining structure is calculated when the lining structure of the lining structure is in the vault of the working condition 1. 3-3.

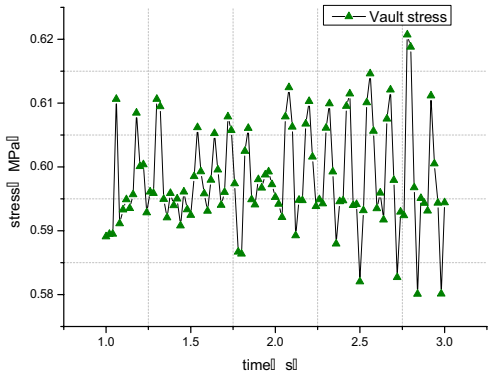

Fig.3-3 Vest position dynamic stress (MPa)

The calculation of the stress time history curve of the top of the lining structure is calculated when the lining structure of the lining structure is in the vault of the working condition 2. 3-4.

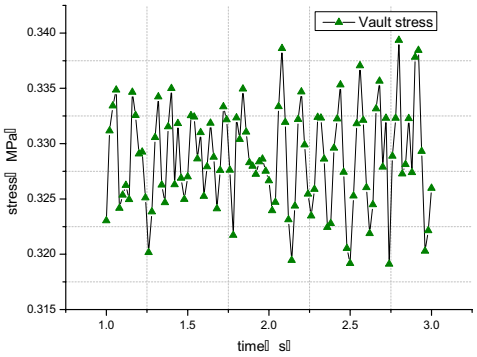

Fig.3-4 Vest position dynamic stress (MPa)

The calculation of the stress time history curve of the top of the lining structure is calculated when the lining structure of the lining structure is in the vault of the working condition 3. 3-5.

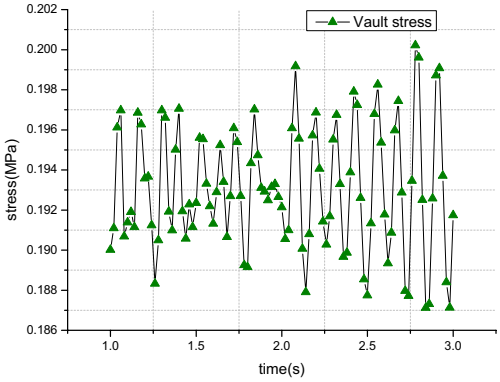

Fig.3-5 Vest position dynamic stress (MPa)

The stress of the arch in the lining of the lining structure is different from that of the case when there are different sizes of cavities

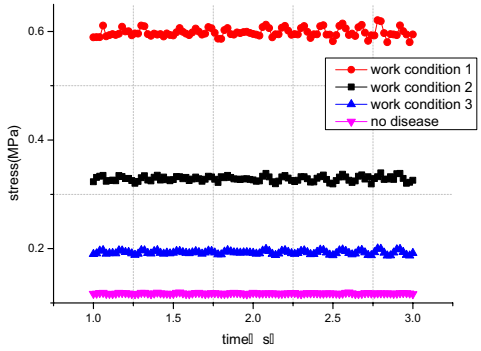

Fig.3-6 Vest position dynamic stress (MPa)

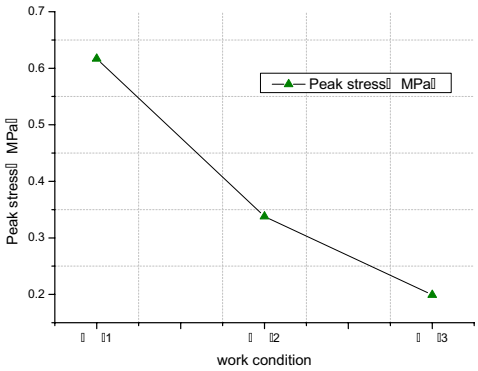

Fig.3-7 The corresponding dynamic stress peaks corresponding to different void sizes (MPa)

We can see from figure 3-3~3-7 in the position of lining vault containing different size cavities under the condition of the vault lining structure dynamic stress showed: in case 1 cases, the load of the train structure dynamic stress peak value is 0.61MPa, the status of the 2 corresponding to the peak value of the dynamic stress is 0.338MPa, 3 case of corresponding conditions the dynamic stress peak value is 0.199MPa, different conditions corresponding to the peak value of the dynamic stress reached the ultimate strength of the concrete tensile strength of 43%, 24%, 14%. dynamic stress peak value is in no case of disease has increased by 0.43MPa, 0.156MPa and 0.071MPa, thus the lining structure containing cavity greater influence of vibration of lining structure under train loads the more obvious.

3.2 structural displacement analysis under different conditions

The calculation of the displacement time history curve of the vault of the lining structure is calculated when the lining structure of the lining structure is in the vault of the working condition 1. 3-8.

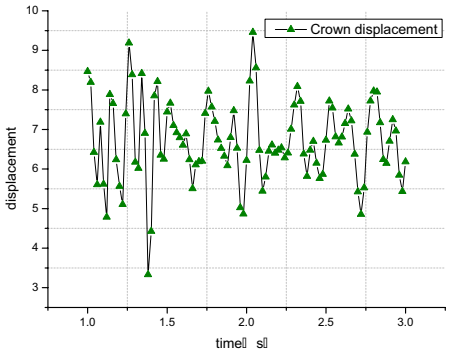

Fig.3-8 Dome position displacement (mm)

The calculation of the displacement time history curve of the vault of the lining structure is calculated when the lining structure of the lining structure is in the vault of the working condition 2. 3-9.

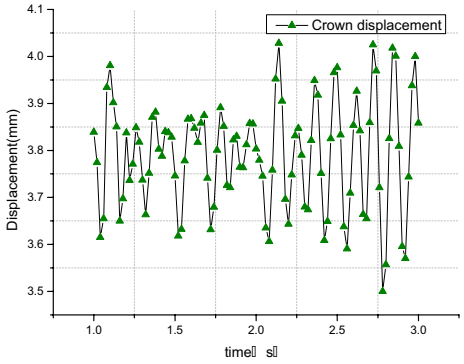

Fig.3-9 Dome position displacement (mm)

The calculation of the displacement time history curve of the vault of the lining structure is calculated when the lining structure of the lining structure is in the vault of the working condition 3. 3-10.

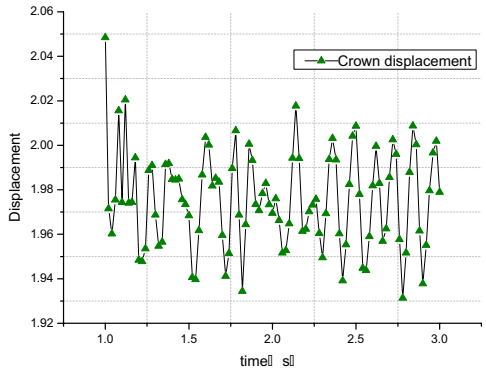

Fig.3-10 Dome position displacement (mm)

The stress of the arch in the lining of the lining structure is different from that of the case when there are different sizes of cavities

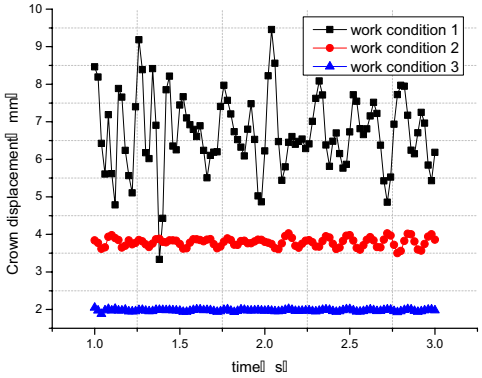

Fig.3-11 Dome position displacement (mm)

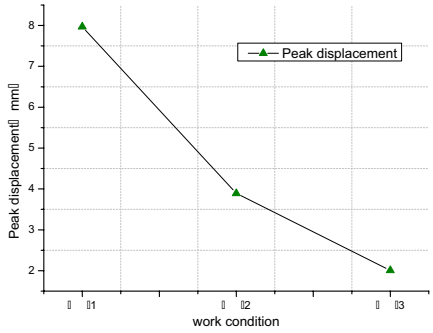

Fig.3-12 Dome position displacement (mm)

According to the 3-8~3-12 diagram of lining structure exists insufficient thickness in vault case, time history curves of vault displacement curve describing the timely contrast map, peak displacement contrast can be seen in the condition of 1 cases, the load of the train when the peak displacement structure of 7.97mm, the status of the 2 corresponding to the peak value of the dynamic stress conditions for 3.89mm 3 cases corresponding to the peak value of the dynamic stress is 2.01mm, corresponding to different conditions the lift curves are basically the same displacement force peak corresponding to the different conditions when compared with the allowable deformation of the lining structure is too small, but can be seen in the lining cavity in the greater displacement response performance of the more obvious, and with the increase of cavity lining, the displacement increases more.

According to the position of lining vault containing different sizes of empty set in 3 different positions of the vault condition contrast dynamic stress and displacement comparison, we can find that under the vibration load of train, changed a lot of stress of lining structure containing voids with and without the disease, but also a displacement without disease the case has increased 9.5 times, and the dynamic stress and displacement is greater with the lining cavity containing more dynamic stress, although these are still in the numerical control limit value range.

4 Conclusion

(1) different conditions corresponding to the peak value of the dynamic stress reached the ultimate strength of the concrete tensile strength of 43%, 24%, 14%. dynamic stress peak value is in no case of disease has increased by 0.43MPa, 0.156MPa and 0.071MPa, thus the bigger the cavity containing lining structure, lining structure vibration under train loads is more obvious.

(2) exist in vault thickness of lining structure under time history curves of vault displacement curve describing the timely contrast map, peak displacement comparison, we can see different conditions corresponding to the dynamic curve of displacement should be consistent in different conditions corresponding to the peak stress, compared with the allowable deformation of the lining structure is too small however, as can be seen in the inner lining cavity is large displacement response performance of the more obvious, and with the increase of cavity lining, the displacement increases more.

(3) the position of lining vault containing different sizes of empty set in 3 different positions of the vault

condition contrast dynamic stress and displacement comparison, we can see that great changes have taken place in the stress of lining structure containing voids with and without the disease, but also less disease under the condition of displacement increases by 9.5 times many, and the dynamic stress and displacement is greater with the lining cavity containing more dynamic stress, although these values are still in the limit control range, but in the long-term train vibration loads and the deterioration of lining structure of tunnel, damage to the structure cannot be ignored.

Acknowledgements

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Experimental study on the effects of crack width and depth on the chloride penetration of concrete

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Key words: Concrete; Crack; Chloride penetration

Abstract: In order to study the effects of crack width and depth on chloride penetration, concrete specimens were made with artificial cracks by using steel sheets, and Rapid Chloride Migration testing(RCM) were carried out. The highest of penetration depth was obtained around the crack. Away from crack leads to smaller penetration depth. The software analysis of SPSS showed that the influence of crack depth on the maximum penetration depth and diffusion coefficient of chloride ion was more significant than that of crack width. The main factor that affects the maximum penetration depth and diffusion of chloride ion is the crack depth.

Conference topic : Civil engineering, Sustainable engineering structure, Materials.

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1. Introduction

Concrete structures are exposed to the atmosphere, soil, water and sea water at all times, and are subjected to changes of temperature, humidity, water level and invasion of harmful chemical media. Cracks will become water or other chemical media intrusion channel,which results in the destroy of concrete cover[1]. In recent years, the study on concrete microstructure and plenty of engineering practices proved that concrete cracks are inevitable[2].If the crack resistance of concrete structure is too strict, it is bound to pay an unbearable economic cost, so the study of the influence of cracks on concrete permeability is of great practical significance.

There are many methods to measure the impermeability of concrete, among which water resistance test and chloride ion penetration test have been commonly adopted in the experimental study. The anti permeability performance is enhanced along with the increase of concrete strength grade. It is difficult to measure the permeability of concrete by anti water permeability test. At present, the chloride ion penetration test is put to use at home and abroad. The permeability of concrete varies in different degrees because of the different crack width and depth. However, most studies and actual measurement only pay attention to the crack width while ignoring the depth, which is obviously unreasonable.

In order to study the influence of chloride ion penetration in cracked concrete, the first step is to prepare the cracks. Different methods were used to create cracks. ACOBSENS et al[3]. made the random cracks which forming in the freeze-thaw cycle environment. The Splitting method is used to make the fixed-width cracks[4-6]. MARSAVINA et al[7]. used copper slice to create controllable cracks in width and depth. In these methods, only the last method can control the depth of crack. In this paper, improving method is used to make cracks.

2. Experimental Program

Cylindrical concrete specimens with cracks of size 100 mm diameter and 50 mm height have been adopted in this test.

2.1.Materials

Cement, coarse/fine aggregate, fly ash,water reducer and water are basic materials. The cement is ordinary Portland cement, P•O Type 42.5, which is produced by Beijing jinyu Liulihe cement co., ltd.The coarse aggregate is crushed gravel with continuous grading which ranges from 5 mm to 10 mm and 10 mm to 20 mm from Sanhe Fuwang gravel field.The fine aggregate is river sand from Qinhuangdao Cuizhuang sand field. The secondary fly ash produced by Luannan Baotong fly ash building materials Co. Ltd.,Polycarboxylate Superplasticizer produced by Shanxi Katie Building Material Co., Ltd. ,and potable water are used. The average compressive

strength of concrete 28d is 49.1MPa. The mix proportions of concrete are listed in Table 1.

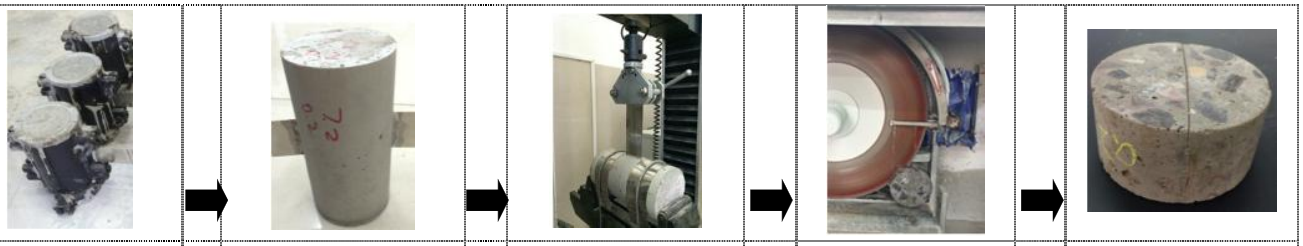
Table 1.Concrete proportioning（kg/m^3）

cement	fly ash	sand	Gravel(5-10mm)	Gravel(10-20mm)	water reducer	water
298	99	761	274	821	3.6	143

Concrete Samples Cracking

In order to get concrete specimens with artificial cracks, thin steel sheets are precisely fixed in the center of the cylindrical moulds with diameter of 100 mm and height of 200 mm, followed by concrete casting. These steel sheets have a width of 50mm and thickness of 0.1 mm, 0.2 mm, 0.3 mm. All specimens are demolded at one day and stored in a climate room at 20℃ and more than 95% RH for 28 days. The test specimens undemoulding were shown in Figure 1(a) and demolded were shown in Figure 1(b).

As shown in Figure 1(c), A crack in the specimen along the diameter direction is appearing, after the steel sheets were carefully removed from the specimens by universal testing machine.The specimens are cut into 2 standard specimens which had a height of 50mm and diameter of 100 mm.The crack width of the standard specimens are determined by the thickness of the steel plates, and the depth of the crack is determined by the position of the cutting.The process of cutting specimen is shown in Figure 1(d), and the prepared standard specimen is shown in Figure 1(e). The combinations of crack width and depth studied in this research are given in Table 2.



(a) Making specimens with steel sheet | (b) Specimen with steel sheet (Φ100mm×200mm) | (c) Pulling out steel sheet | (d) Cutting specimen | (e) specimen with crack (Φ100mm×50mm)

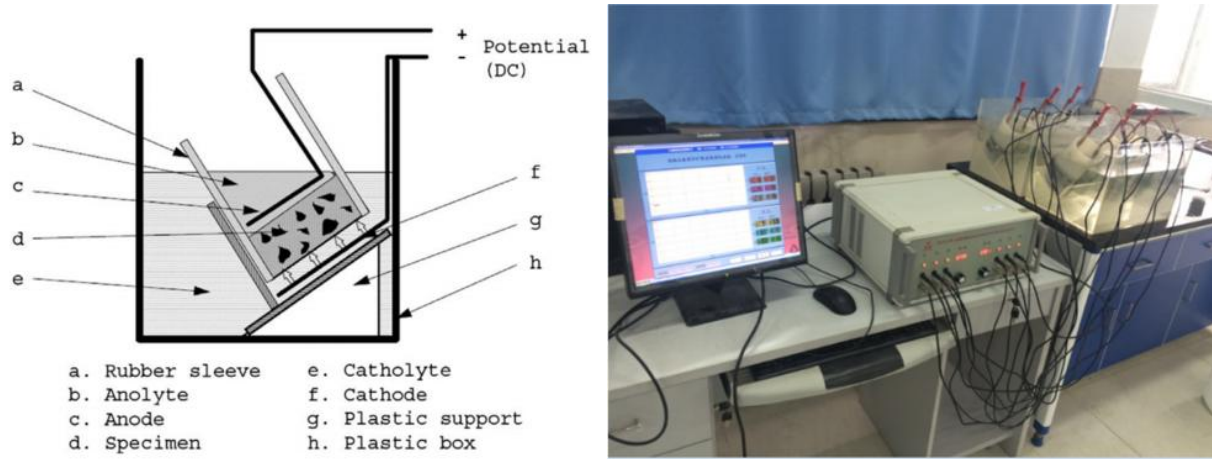
Fig.1. Preparation process of specimens with cracks

Table 2.Width and depth of crack

Crack width(mm)	Crack depth(mm)		
0.1	5	10	15
0.2	5	10	15
0.3	5	10	15

2.3. Test process

In this research, the Rapid Chloride Migration(RCM)test, standardized in NordTest NT-Build 492[8], is used for the the chloride penetration. As described in Section 2.2, cracking specimens with a diameter of 100mm and a height of 50mm were made before 7 days before the test. Afterwards the concrete specimens were placed back in the fog room until the testing date. One day before testing the specimens were vacuum saturated with a saturated $Ca(OH_2)$ solution. Afterwards an external electrical potential (30V)was applied across the specimen for a limited duration, forcing the chloride ions from the 10% NaCl solution (catholyte) to migrate into the specimens. For all the specimens, the test duration was 24 h, as prescribed by NT BUILD 492 .



(a) Experiment principle (b) Test instrument

Fig.2. Test method

After the test, the specimens were axially split, perpendicular to the notch (Fig. 8 and 9). On each of the freshly split sections, a 0.1M $AgNO_3$ solution was sprayed and the chloride penetration depth is measured at 15 points, visually checking the presence of white silver chloride precipitation(Fig. 10). This colorimetric method is described in NT BUILD 492. For each concrete specimen , two penetration profiles were obtained. In this way, 6 penetration profiles are obtained for each composition and notch configuration. The penetration profile for crack width 0.3mm and crack depth 20 mm is given in Fig. 10.

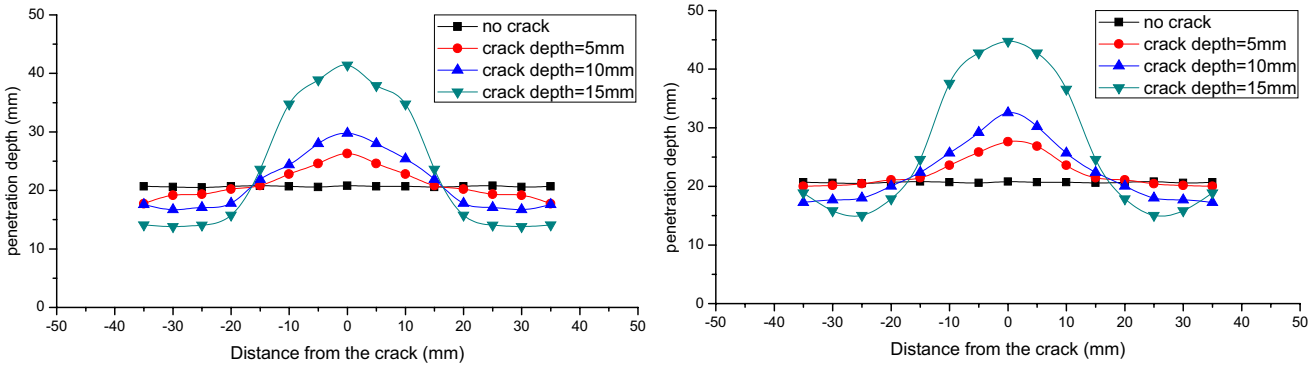
(a) Splitting specimen in half (b)Specimens in half (c)Penetration profiles

Fig.3. Experimental procedure

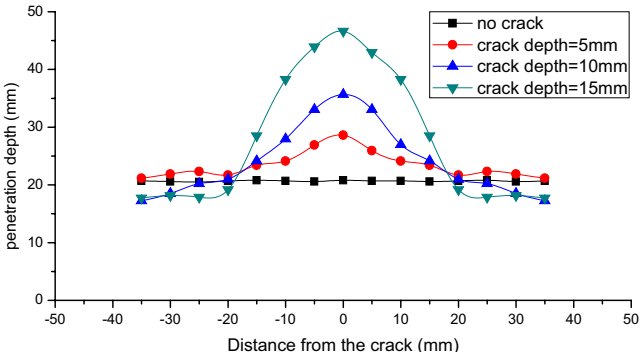
3. Discussion of test results

From the chloride penetration profiles,two influencing factors will be studied :crack with and crack depth.

3.1. Influence of crack depth



(a)Crack width is 0.1mm (b)Crack width is 0.2mm

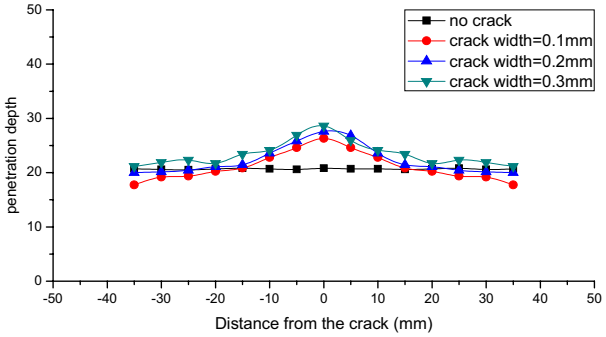


(c)Crack width is 0.3mm

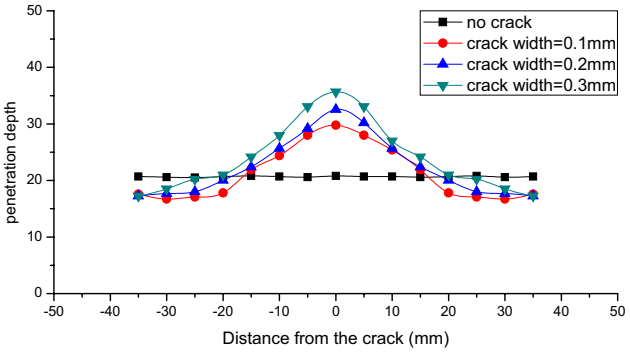
Fig.4. Chloride penetration depth(Same width)

In order to analyse the influence of crack depth from the chloride penetration profiles, the information of profiles are described with 3 figures according to the crack width in Fig.4. It shows that the penetration depth increases with the increase of crack depth around -20mm to 20mm and that the penetration depths have different values in different regions of the cracked concrete. The maximum penetration depth of the crack zone is calculated as the value of the penetration depths around the notch. Around -10 mm to 10 mm of distance from the crack, the penetration depths were significantly higher than in the other part of the concrete. On the other hand, the effect of crack depth on penetration depth is sharp increasing around -20 to -10 mm and 10 mm to 20 mm. Besides the penetration depths around -30 to -20 mm and 20 mm to 30 mm are quite close, and the penetration depth of concrete is significantly lower than that of other regions.

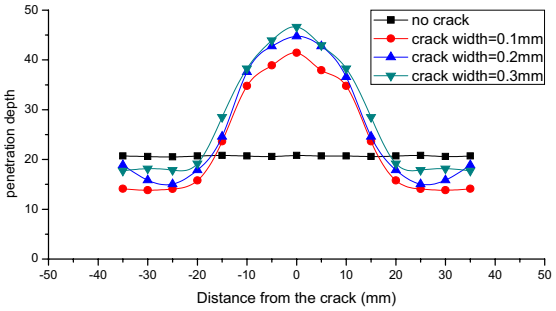
3.2. Influence of crack width



(a) Crack depth is 5mm



(b) Crack depth is10mm



(c) Crack depth is 15mm

Fig.5. Chloride penetration depth(same depth)

Figure.5 shows the influence of crack width on chloride penetration. It shows that the penetration depth of chloride ion increases slowly with the increase of crack width. The penetration depth of chloride ion in concrete is quite different in different regions. Around -10 mm to 10 mm of distance from the crack, the penetration depths were significantly higher than in the other part of the concrete. On the other hand, the effect of crack depth on penetration depth is sharp increasing around -20 to -10 mm and 10 mm to 20 mm. Besides the penetration depths around -30 to -20 mm and 20 mm to 30 mm are quite close, and the penetration depth of concrete is significantly lower than that of other regions.

4.Significance and importance analysis of crack width and depth

The maximum penetration depth of chloride ions of concrete specimens are shown according to the different width and depth of cracks in Table 3.

Table3. Maximum penetration depth and diffusion coefficient of chloride ion

Crack width/mm	Crack depth(mm)	Maximum penetration depth(mm)	扩散系数($10^{-12}m^2/s$)
0	0	22.1	9.4
0.1	5	26.3	9.6
	10	29.8	9.7
	15	41.4	10.4
0.2	5	27.6	10.3
	10	32.6	10.3
	15	44.8	12.4
0.3	5	28.6	10.8
	10	35.7	11.1
	15	46.7	13.3

4.1 Significance analysis

In order to analyze the the influence of crack width and crack depth on the maximum penetration depth and diffusion coefficient of chloride ion, multivariate analysis of variance is applied by SPSS software(the confidence level is 95%). The analysis result is given in Table 4. As can be seen from table 4, the influence of crack width on the maximum penetration depth and diffusion coefficient of chloride ion is significant,because the significance is between 0.01 and 0.05. The influence of crack depth on the maximum penetration depth of chloride ion is very significant on account of the significance less than 0.01 and on diffusion coefficient of chloride ion is significant because of the significance between 0.01 and 0.05.

Table3. Result of multivariate analysis of variance

Source	sig.（Dependent variable: maximum penetration depth of chloride ion）	sig.（Dependent variable: chloride diffusion coefficient）
Crack width	.013	.027
Crack depth	.000	.021

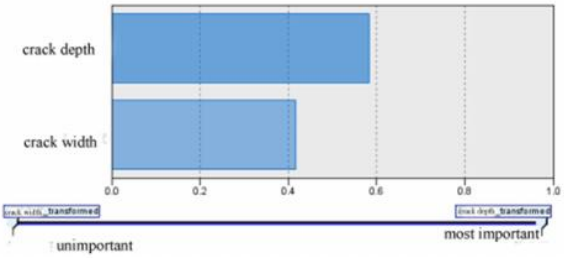
influence is not significant. If it is between 0.01~0.05, the effect is significant, and if less than 0.01, the effect is very significant

4.2 Importance analysis

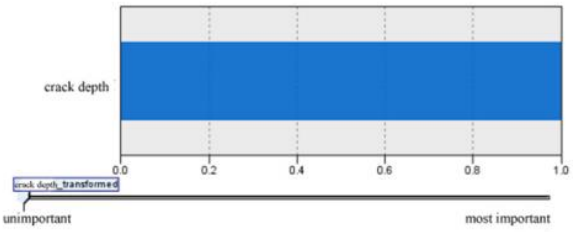
According to the linear regression function of SPSS software, the importance of the crack width

and depth on the maximum penetration depth and diffusion coefficient of chloride ion is predicted, as described in Fig.6.

Can be seen from Figure 6(a), the importance coefficient of crack depth is about 0.58, the importance coefficient of crack width is 0.42, the importance of the crack depth is greater than the crack width, crack depth is the main factor affecting the diffusion coefficient of chloride ion.From Figure 6(b), the importance coefficient of crack depth is about 1 and crack width is 0. Obviously, the importance of the crack depth is far greater than the crack width. In short, the crack depth is the main factor affecting the maximum penetration depth of chloride ion.



(a) permeability coefficient of chloride ion



(b) maximum penetration depth of chloride ion

Fig.6.Importance prediction

5. Conclusion

(1) The highest of penetration depth was obtained around the crack, Away from crack leads to smaller penetration depth.

(2)The influence of crack depth on the maximum penetration depth and diffusion coefficient of chloride ion was more significant than that of crack width.

(3)The main factor that affects the maximum penetration depth and diffusion of chloride ion is the crack depth.

Acknowledgment

Notes：If the significant value is greater than 0.05, the
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STUDY ON SETTLEMENT AND DISPLACEMENT OF ADJACENT BUILDINGS OF EXCAVATIONS TO CORNER EFFECT

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Key words: conrner effect, three-dimensional settlements, numerical simulation, parameter analysis

Abstract: The corner effect has significant influence on the displacement of buidings near the excavation. Based on the abnormity excavation project that has a rhomb section in coastal soft soil area, a couping model of the foundation pit and the buildings outside the pit is established. The displacement characteristics of the buildings near the corner are studied by using the method of Midas/GTS. The calculated results show that the settlement of the buildings near the corner is characterized by 3D, which performance in both vertica and parallel directions of the excavation. The size of the pit angle will affect the significant degree of the 3D settlement. Under certain pit angle, the distance between the building and the foundation pit, the depth of the foundation will affect the deformation of the building. In addition, the stress and strain of the wall and floor of the building also has a certain distribution law.

Conference topic : 2. Civil engineering or Sustainable engineering structure

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0 Introduction

For deep foundation pit engineering, the first is to ensure the safety and stability of the foundation pit itself, but in recent years, the movement of surrounding strata and the influence on surrounding buildings in the process of foundation pit excavation become the popular and difficult point. In some areas of our country, the soil character is good, such as plasticity clay and sandy soil with medium density, the displacement of surrounding strata caused by excavation is small, as long as the realistic protection measures can be taken to protect the surrounding structures. But in some poor formations ares,such as Tianjin, Shanghai, Guangzhou, etc, most of the soil in the areas is soft clay, the excavation of the foundation pit is bound to cause a larger displacement of the surrounding strata,and then have a negative impact on the surrounding buildings.

With the rapid development of foundation pit engineering, the surrounding environment of foundation pit is becoming more and more complex, and the method of theoretical experience can not meet the requirements of Engineering complexity. Th-e emergence of numerical methods provides a new way to solve this kind of problem. Although the time of this method is not very long, it is obviously the most popular method in engineering field.

1 Description of influence of corner effect on the buildings nearby

The foundation pit is a kind of three dimensional space that has the characteristics of long, wide and deep. Its stress deformation characteristics is closely related to the properties of the soil, the geometric size and the type of support. The influence of pit angle on the analysis of foundation pit is an important aspect to consider the space effect. At present, it is often done by analyzing the cross section along the side of the foundation pit on the foundation pit problem, at the same time, the theory of elastic foundation beam is studied. The analysis method, which ignores the influence of pit angle, is often conservative and uneconomical, and the analysis result is not accurate.

Because of the existence of pit angle, the displacement of the soil outside the pit is different. In the area close to the pit, the displacement is smaller, but the displacement is larger far from the corner. It can be inferred that due to the existence of the pit angle the existing buildings around the foundation pit will have uneven settlement and flexural deformation. The difference of soil displacement make the force and deformation of the surrounding buildings very complex.

In this paper, the shape of the foundation pit is diamond. So in this case, the force deformation of the building nearby is more complex.

At present, the influence of excavation on adjacent buildings is often ignored. In the analysis of this effect, the empirical method often be used in which the foundation pit and building are discussed separately. Firstly, the ground settlement is estimated without the building outside the pit. Then the deformation of the building is estimated according to the deformation. Obviously this method is not accurate. It can not consider the coupling between the foundation pit and the existing building, and the result strongly dependents upon engineering experience.

2 Engineering background

The Jianxin building foundation pit is located in the eastern coastal areas of southern Huanghua, Nan Dagang. The foundation is surrounded by four roads, the East is the Jianshe street, west of Haifeng street, south of Xueyuan Road, north of Bohai Road. Neither of these four roads is from the east to the west, nor from the south to the North. The shape of the foundation pit is restricted, shaping diamond section. The length of the pit is 60 meters. The two adjacent angles is 60 degrees and 120 degrees respectively. Adjacent to the north side of the foundation pit is the primary school. The west side of the foundation pit is a newly built large office building. The South and east of the foundation pit are residential areas. Especially the residential buildings, built in the last century in 90s, disrepair. Therefore, it is required to have the least influence on these residential areas. The foundation pit is supported by underground continuous wall and steel support. Underground continuous wall thickness is 0.8 meters. Support system uses two layers of steel pipe. Each layer has 10 roots. Diameter of the steel pipe is 609 millimeter. The wall thickness of the steel pipe is 16 millimeter. During the excavation of the foundation pit, the supporting are loaded in time. The first layer is supported at one meter away from the ground, and the second layer is supported at a distance of 6 meters from the ground. The site where geomorphic units belong to Bohai underwater slope. Followed by artificial filling, backfill to the current elevation. Soil is rich in organic matter, soft soil. Because of the short history of soil formation and the flocculation of sea water, the degree of consolidation is poor. According to its genesis, lithology and physical and mechanical properties, the soil is divided into 12 layers. The lithology and distribution of the soil layers in the site are shown in table 2-1. The values of material parameters are shown in table 2-2.

Table 2-1 lithologic distribution of soil layers

Layer number	Historical reasons	Rock name	Bottom elevation/m	Lithologic characteristics
①	Q_4^{2ml}	miscellaneous fill	-0.14~-1.43	Brown gray, with silt clay, silt silty clay is given priority to, contain decay to plant quality, soft plastic ~ plasticity, for recent filling
②		muddy clay	-1.29~-3.6	Brown gray, with rotten vegetation, thin layer of silt, soft plastic, no vibration response, gloss, dry strength and toughness
③		silt	-3.84~-6.92	Gray, containing mica, shell, medium dense, very wet, shake reaction medium, dull reaction, low dry strength, low toughness
④	Q_4^{m}	silty clay	-5.95~-14.22	Gray, with rotten plant, shell, silt clay layer, soft plastic to plastic, no vibration response, slightly shiny, dry strength medium, medium toughness
⑤		silty clay with silt	-12.92~-18.32	Gray, with rotten plant, shell, soft plastic to plastic, no vibration response, slightly shiny, dry strength medium, moderate toughness, thin layer of silt
⑥	Q_4^{al}	silt	-21.67~-23.85	Brown yellow, containing mica, iron oxide, medium dense, very wet, shake quickly, dull reaction, low dry strength, low toughness.
⑦	Q_3^{mc}	silty clay	-32.67~-41.35	Tyrosinance ~ brown, containing iron oxide, plastic, no shaking,

			slightly shiny, dry strength medium, medium toughness, with thin silt
⑧	silt	-44.13~-46.57	Pale yellow, with mica, iron oxide, dense, wet, shaking fast reaction, dull reaction, dry low strength and low toughness. The bottom of a thin layer of silty clay, hard plastic

Table 2-2 material parameters

material types	bulk density $\gamma/(\mathrm{kN\cdot m^{-3}})$	Elastic modulus E / MPa	Poisson ratio v	internal friction angle φ	cohesive c / kPa
miscellaneous fill(1 m)	18	5	0.4	17.6	23
muddy clay(2 m)	19.2	3	0.39	11.3	10
silt(3 m)	19.1	10	0.38	19.3	39
Silty clay(8 m)	19.1	12	0.38	25	34
silty clay with silt(6 m)	19.2	14	0.39	17.8	32
silt(2 m)	19.4	14	0.38	18.6	24
silty clay(8 m)	19.0	20	0.37	20	21
silt(10 m)	19.4	20	0.39	22	30
reinforced concrete	24	25000	0.17	—	—
steel support	78	210000	0.2	—	—

3 Building parameters

Based on the study of the 3 story concrete buildings near the foundation pit, the building models are established and simplified. The model of the building is 16 meters long, the width is about 8 meters, and the long side direction is parallel to the edge of the foundation pit. The height of the floor is 3 meters, and the thickness of each floor is about 0.3 meters. The whole building is simulated by plate element. Elastic constitutive relation is used. Taking into account the factors such as the disrepair of the building, the elastic modulus of the wall is smaller than that of the floor slab. The elastic modulus of the wall is 220MPa, and the elastic modulus of the floor is 30GPa. Figure 3-1 show the settlement under the action of deadweight. The maximum differential settlement is 5.2millimeter . The relative deflection value is 0.325×10^{-3}, satisfying to the allowable range of the literature. It can be seen that the parameter selection of building model is reasonable. Because the building is existing building, this deformation under the action of self weight should be cleared before excavation.

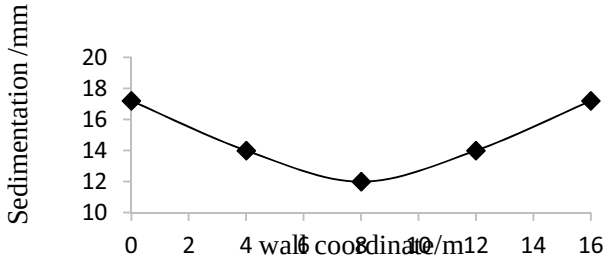
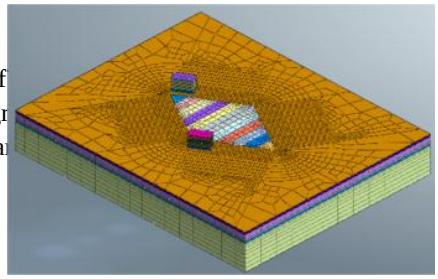


Figure 3-1 Settlement under the action of deadweight

4 Building settlement analysis

4.1 Analysis of building settlement under different pit angle

In order to study the effect of pit angle on the deformation of under different angle of the pit, the buildings are placed at 60 degr degrees, 45 degrees, 135 degrees, 30 degrees, 150 degrees of pit a shown in Figure 4-1.



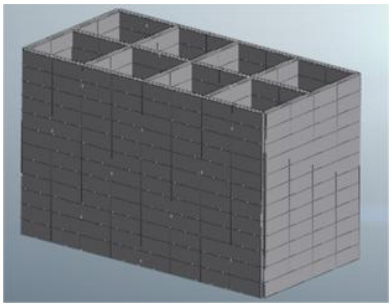


Figure 4-1 Coupling model of building and foundation pit

Due to the existence of the corner effect, it will inevitably have a negative impact on the buildings near the corner. This effect is different from that of the building near the center of the pit. In order to study the effect of corner effect on existing buildings, Firstly, the influence of different angles on the outside of the building is considered. The buildings are placed in six different angles near the corner respectively, while the buildings placed in the center of the long side of the rectangular foundation pit and the former six cases contrast. At the same time, a control group is established, that place the building outerside the center of the long side of the rectangular foundation pit. The long side of the building is parallel to the edge of the pit, with a distance of 4 m. The observation points are set up around the building to observe the settlement of the building. After the excavation of the foundation pit, the settlement of the buildings near the six different angles of the pit is shown in figure 4-2. The settlement of the control group is shown in figure 4-3

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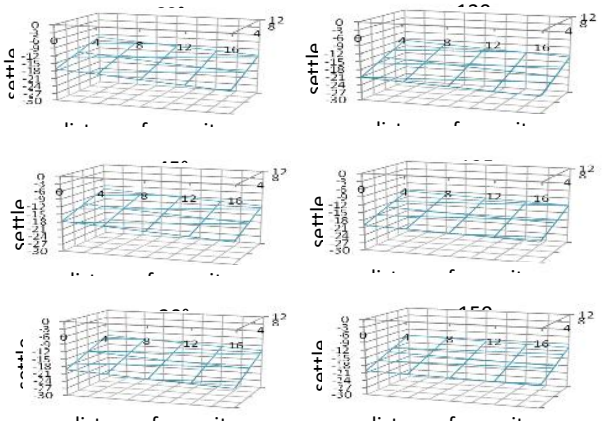


Figure 4-2 Settlement of building foundation near six different corner

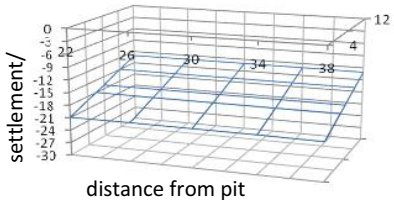


Figure 4-3 The settlement of the building located in the center of the long side of the rectangular pit

In the six case, the uneven settlement occurred, and all were obvious. The building at the 60 degree corner, the maximum settlement is 21.23 mm. In the settlement of the four corners of the building, the maximum differential settlement occurs on the diagonal, which is 10.87 mm. The building at the 120 degree corner, the maximum settlement is 21.23 mm, and the maximum differential settlement is 10.87 mm. The building at the 45 degree corner, the maximum settlement is 21.30 mm, and the maximum differential settlement is 10.82 mm. The building at the 135 degree corner, the maximum settlement is 22.68 mm, and the maximum differential settlement is 9.2 mm. The building at the 30 degree corner, the maximum settlement is 22.3 mm, and the maximum differential settlement is 11.32 mm. The building at the 150 degree corner, the maximum settlement is 21.35 mm, and the maximum differential settlement is 9.54 mm. These figures can be used to express the influence of corner effect on building settlement. The common point of the six cases is that the buildings are shown bidirectional rotation. In this case, any wall of the building, including the floor of each floor, may result in cracks. As can be seen from the figure, when the building is located near the center of the long side of the rectangular foundation pit, the building will only produce a single direction of rotation, which is inclined to the pit. In this case, the cracks usually appear in the vertical wall of the foundation pit. The edge of the wall and the floor will not appear cracks.

4.2 Influence of different foundation depths on settlement

We know that the excavation will have a certain impact on the surrounding soil, so the depth of the foundation of the building will become an important factor affecting the building settlement. Next, the buried depth of the foundation pit is studied by taking the diamond pit with 60 degrees and the adjacent angle of 120 degrees as an example. In the preceding article, the settlement curve that buried depth of 2m has been obtained. Next, the bottom of the foundation are buried in the ground below 4m and 6m, the settlement curve as shown in figure 4-4.

Under the circumstances that 60 degree angle and the depth of 4 m, the maximum settlement is 17.8 mm and the maximum differential settlement is 8.4 mm. Under the circumstances that 120 degree angle and the depth of 4 m, the maximum settlement is 14.5 mm and the maximum differential settlement is 6.4 mm. Under the circumstances that 60 degree angle and the depth of 6m ,the maximum settlement is 8.5 mm and the maximum differential settlement is 3.4 mm. Under the circumstances that 120 degree angle and the depth of 6 m ,the maximum settlement is 8.2 mm and the maximum differential settlement is 2.6 mm.

Based on the analysis, the deeper the foundation, the more obvious the effect of settlement control. When the base depth is 6 m, only a very small settlement and uneven settlement occurs. It shows that the influence of the excavation on surrounding soil is about 6 m.

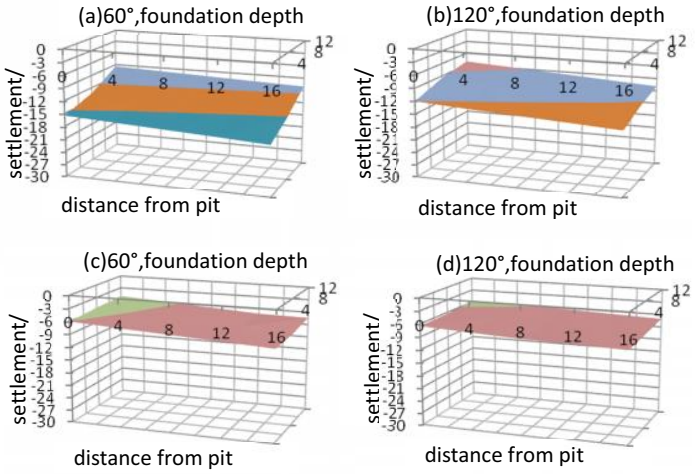
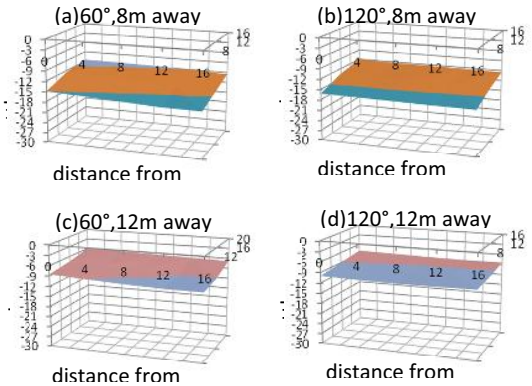


Figure 4-4 Foundation settlement under different buried depths

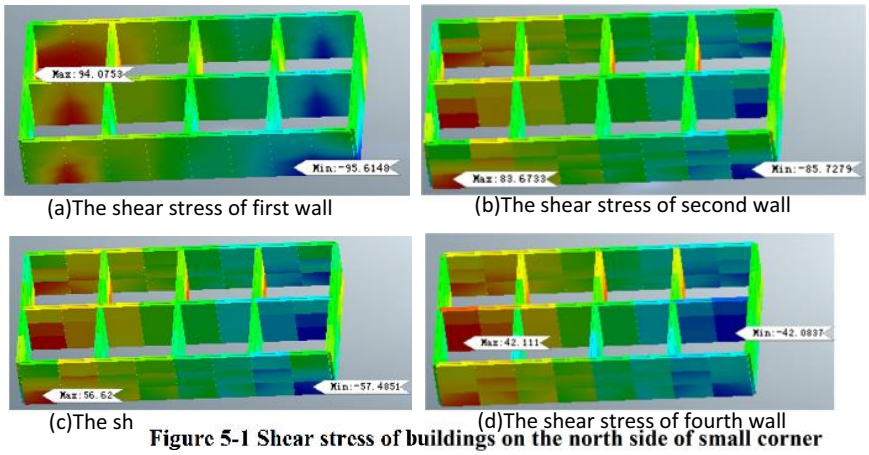
4.3 Settlement of buildings in different locations



According to the former analysis, the settlement shape of the vertical wall direction after the wall is "spoon type". The distance between the building and the foundation pit is different, and the settlement will be different. Next, we discuss the influence of the distance between the edge of the foundation pit and the length of the building. The following analysis is done by taking the diamond pit with 60 degrees and the adjacent angle of 120 degrees as an example. The buildings are placed at 8 m and 12 m from the edge of the pit, and the final settlement of the building is shown in figure 4-5. At a distance of 8m from the 60° pit angle, the maximum settlement is 16.5mm and the maximum differential settlement is 4.0 mm. Compared with the distance of 4m, reducing 20.5% and 63.2% respectively.

5 Shear stress analysis of the wall

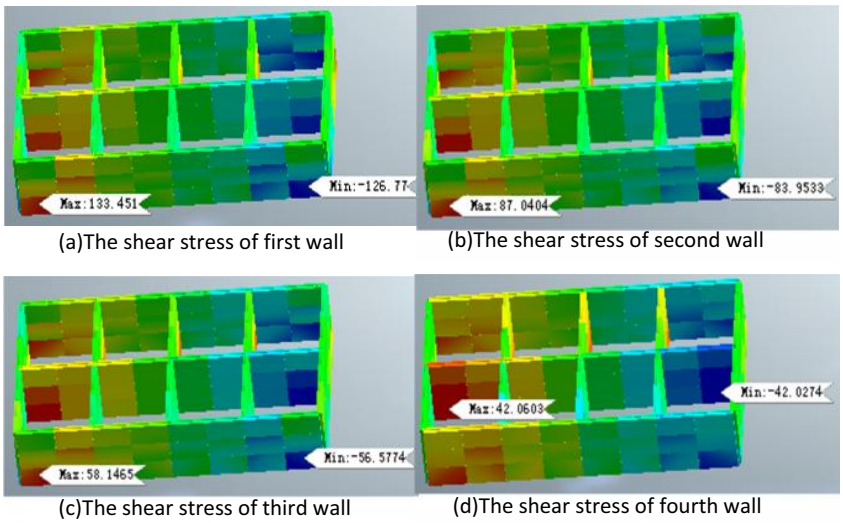
The shear stress of the building located at the north small corner when the excavation is completed is shown in figure 5-1. The maximum value of shear stress on the first wall of the building is 95 kPa. The maximum value on the second wall is 85 kPa. The maximum value on the third wall is 57 kPa. The maximum value on the fourth wall is 42 kPa. The maximum shear stress of each layer wall appears on the longitudinal wall as well as near the transverse wall. The maximum shear stress decreases from the first layer to the fourth layer.



(a)The shear stress of first wall (b)The shear stress of second wall

(c)The sh (d)The shear stress of fourth wall

Figure 5-1 Shear stress of buildings on the north side of small corner



(a)The shear stress of first wall (b)The shear stress of second wall

(c)The shear stress of third wall (d)The shear stress of fourth wall

The shear stress of the building located at the south large corner when the excavation is completed is shown in figure 5-2. The maximum value of shear stress on the first wall of the building is 133 kPa. The maximum value on the second wall is 85 kPa. The maximum value on the third wall is 58 kPa. The maximum value on the fourth wall is 42 kPa. The maximum shear stress of each layer wall also appears on the longitudinal wall as well as near the transverse wall. The maximum shear stress decreases from the first layer to the fourth layer.

Conclusion

The size of the pit angle will affect the significant degree of the 3D settlement. Under certain pit angle, the distance between the building and the foundation pit, the depth of the foundation will affect the deformation of the building. In addition, the stress and strain of the wall and floor of the building also has a certain distribution law.

Acknowledgment

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EXPERIMENTALE STUDY ON MECHANICAL BEHAVIOR OF NEW TYPE STEEL-TIMBER COMPOSITE BEAM

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Key words: *STC beam; mechanical behavior; damage form*

Abstract: As a traditional building structure type, the environmental features of timberwork are attached importance to by engineers. Improve the bearing capacity of wood structure is becoming more and more important. In this paper, the new type steel-timber composite (STC) beam refers to it is composed of two wooden boards and one steel plate, which is designed in order to Improve the timber bearing capacity. According to the loading capacity of the testing machine, the maximum size of the reduced scale specimen was calculated. Each scale specimen's steel and timber member was connected with two bolts which diameter is 7mm on the member side. Based on the same length and height, 18 kinds of scale specimens were made and tested. Experimental results show that the bending strength of STC beam is 1.1-1.3 times compared with pure wooden beam. When member cross section is fixed, the ultimate load of specimen is decided by the thickness of steel plate. During beams' bending deformation process, other conditions being equal, as the distance of the two bolts increase, bearing capacity of the STC beam don't change significantly. The damage of wood member belongs to brittle failure, which leads to the STC beam's sudden loss of bearing capacity. Through the above research, as a new structure, the STC beam has obvious advantages, and it can be applied to practical engineering.

1. Introduction

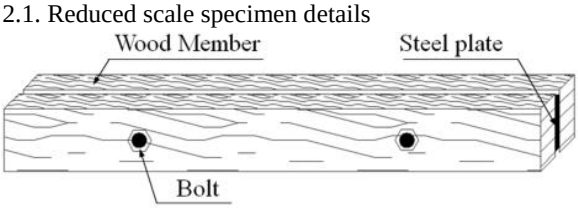
As a kind of environmentally friendly structural material, wood has been paid more and more attention by engineers. Great deals of research results show that the energy consumption and carbon emission of wood structure is less than that of concrete structure and steel structure [1]. As well as, wood has good decorative performance. But wood also has many disadvantages such as easy deformation, low strength and so on. In order to optimize the mechanical properties of wood structure, steel served to makes up the shortcomings of wood [2-3].

Two types of steel-timber composite (STC) structures are concerned by researchers, and one is integrated and the other is separated. Separated STC structure has been widely applied in Japan, and its structural performance studies have been systematized[4]. Study on the integrated steel-timber composite structure is in the initial stage. For STC structure, there are two main ways of integration: bonding and bolting [5]. The use of adhesive bonding method can only be used in space, in where temperature and humidity can be controlled well; otherwise the connection is not reliable. The application of bolt connection is more extensive than that of adhesive bonding. The bearing capacity of bolted STC connections was studied by Weng, which demonstrated that the failure modes, bearing capacity and ductility of connections are relative to the thickness of wood side members [6].The mechanical properties of a cantilever STC beam was investigated by Pan, through which load-strain curve and load displacement curve were got [7]. And the experimental results were compared with the results of finite element analysis, which shows that STC beam has better anti-instability capability than steel structure in the case of dense bolts [8].

But dense bolts leads to the difficulty of construction increased and have a bad effect on the decoration. Accordingly, a new type beam was designed in the paper which only has two bolts, and mechanical performance of these beams was researched by three point bending test.

2. Experimental program

2.1. Reduced scale specimen details



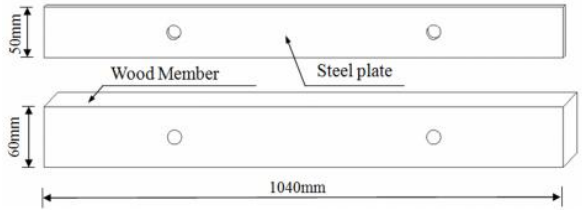


Fig.1. Configuration of the STC specimen and some fixed size

Table 1.Size and configuration of specimens (mm)

wood member thickness	steel plate thickness	pitch of bolts
20	0.72 (S1)	500
25	1.55 (S2)	600
	2.56 (S3)	700

Each reduced scale specimen was made up of a steel plate and two wood members, and the steel plate was placed in the middle of the two planks to make up a specimen that was connected by two bolts (Fig.1). In total, eighteen different combinations whose geometry, detail and setup are shown in Fig.1 and Table 1 were tested. Considering the decoration effect of the members in the actual engineering, the height of the steel plate is smaller than that of the wood unit 10mm in the specimens which is about 1/5 to 1/10 of the actual members' size. In order to make the bolts easy to penetrate, the aperture of the wood member and steel plate is 8mm, which is larger than the diameter of the bolt 1mm. After the integration of specimen, the screw out of the nut is truncated. For ease of recording, the specimens were numbered, such as T25S2D600 which stands for this specimen was made up of 25mm thickness timbers, 1.55mm thickness steel plate and 500mm spacing between two bolts (Table 1).

2.2. Material properties

In this study, Pinus Sylvester was selected as wood member, which was the origin of Russia. All of wood came from the same batch, thus ensuring the uniformity of these specimens. Along the length of the beam, each plank was made parallel to the grain, and the surface of it was leveled by plane. Six wood samples were randomly selected for mechanical testing. Wood members of the STC beams were directly used to measure bending strength (σ_{bW}) and bending modulus of elasticity (E_W). Test result shown that (1) the Maximum of E_W was 21.6 MPa, the minimum was 15.3 MPa and the average was 19.29 MPa; (2) The Maximum of σ_{bW} was 81.6 MPa, the minimum was 56.8 MPa and the average was 69.8 MPa. Three thickness steel plates applied to the experiment, which was shown in Table 1. For each thickness of steel plate, three samples were taken for tensile strength test, and the average strength was shown in the Table 2.

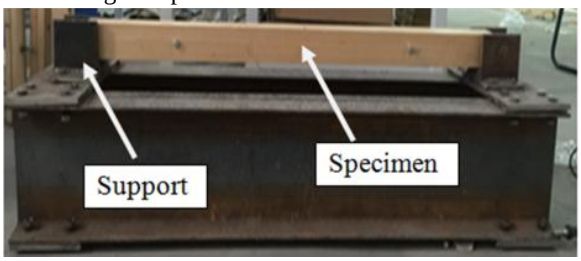
Table 2.Tensile strength of steel plates of different thickness (MPa)

Tensile strength	S1	S2	S3
Yield strength	556	283	384
Ultimate strength	618	454	483

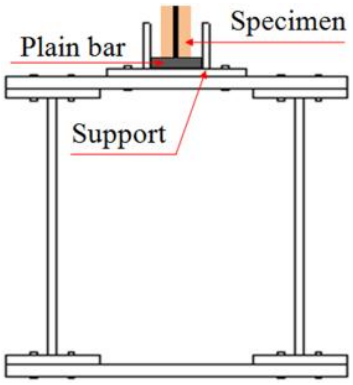
Steel and wood was integrated by two 4.8 grade (Chinese national standard: GB/T 3098.1-2010) bolts which could apply to connect steel structure. In order to ensure that the bolts were not damaged during the experiment, diameter of bolt is big relative to STC beam, because the main research object is wood and steel plates.

2.3. Test setup and instrumentation

In order to use the universal testing machine to carry out the three point bending test, a steel loading frame was designed and manufactured [Fig. 2a]. In order to ensure the safety of the experiment process, the supports were welded by three steel plates and a section of plain bar with a diameter of 20mm [Fig. 2b], which was bolted together with the loading frame, and the two vertical plates on which could effectively prevent the sudden sliding of the specimen. Cylindrical pressing head of the machine and the plain bar of the loading frame enable the force to be evenly distributed with the contact area along the direction of specimen width, without considering the specimen size error.



(a) Loading frame front photo



(b) Transverse sketch map

Fig.2. Loading frame and specimen placement

In order to monitor the deformation of each specimen in the course of the test, two strain gages were pasted on the bottom of the middle position of the wood members on the specimen's both sides, and a strain gage was placed

at the lower edge of the plate. A strain gage was placed on each wood member's surface under the bolt, which could reflect the effect of bolts on wood relatively. On some specimens, strain gages were placed on the wood surface which was located on two flanks of the bolt, which could apply to observe the interaction of wood and bolt. The location and the number of the strain gages were shown in Fig.3.

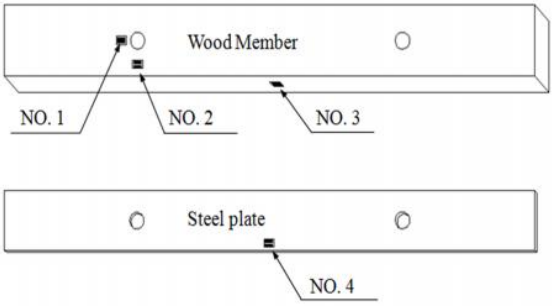


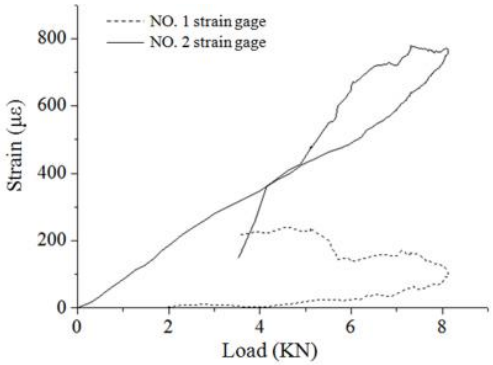
Fig.3. the location and the number of the strain gages

Before the formal test, the specimens were pre loaded so as to reach the displacement of 5mm, in order to make the wood feel force ahead of test. The force acting on the center of the specimen is controlled by displacement, which was as follows: before the deflection of specimen reaches 7mm, loading speed of 10mm/min was given by the universal testing machine; when the deflection reaches 7mm, loading speed changed to 5mm/min until the specimen breaks.

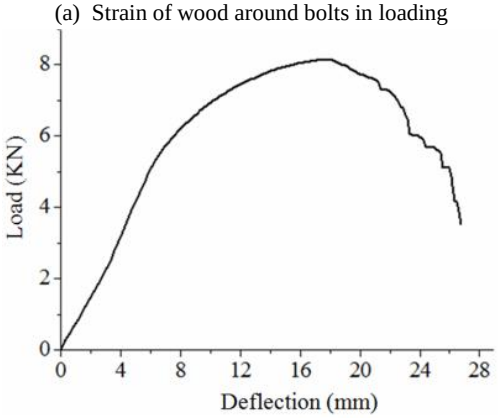
3. Discussion of test results

3.1. Force situation of wood around bolt

As shown in Fig.4, before mid-span deflection to 6mm as well as load to 5kN, these phenomena can be seen, such as: (1) the specimen is in the elastic phase; (2) there is no obvious change in strain of NO.1 gage; and (3) with the loading, the strain under the bolt exhibits approximately linear growth. When load exceeds 5kN to ultimate load, the specimen enters fortified phase, and the lateral force which screw bear increases obviously according to the change of NO.1 strain. When the specimen goes into the destruction stage, neutral axis has disappeared, so the strain of NO.1 lateral force increases rapidly.



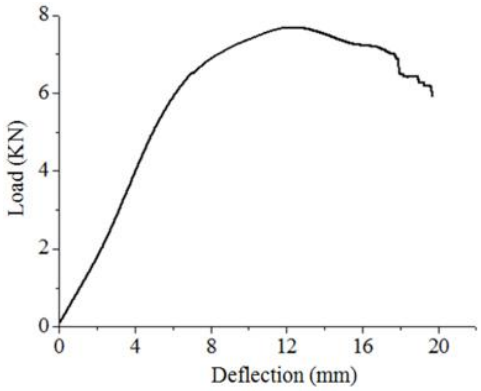
(a) Strain of wood around bolts in loading



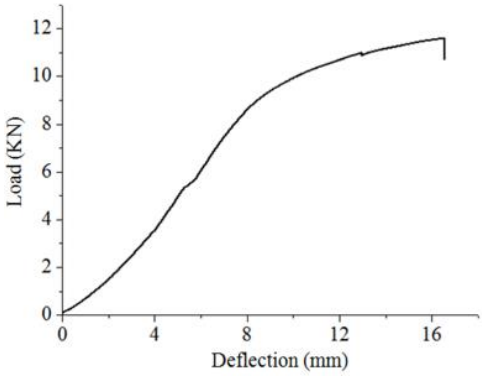
(b) Load-deflection curve

Fig.4. Experimental results of mechanical properties of T20S2D500

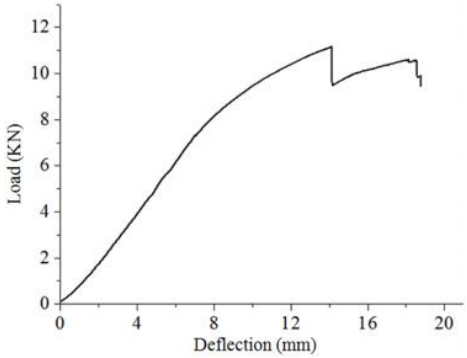
3.2. Bearing capacity of specimen



(a) T20S1D600



(b) T25S3D600



(c) T25S2D600

Fig.5. Load-deflection curves of three specimens

By observing a large number of experimental results such as Fig. 5, the yield point can be obtained, which is the load value corresponding to the deflection of 7mm or so. Therefore, we regard the load value corresponding to the deflection of 7mm as the yield point of the specimen, which was shown in table 3, table 4 and table 5.

Table 3.Yield load of specimens with 20mm thickness of timbers (kN)

Thickness of steel plates	Yield load (D500)	Yield load(D600)	Yield load(D700)
S1	5.95	6.35	5.85
S2	5.70	6.23	5.52
S3	6.80	6.87	7.10

Table 4.Ultimate load of specimens with 20mm thickness of timbers (kN)

Thickness of steel plates	Ultimate load(D500)	Ultimate load(D600)	Ultimate load(D700)
S1	7.34	7.35	7.41
S2	8.43	8.37	8.42
S3	9.29	8.88	9.51

In contrast to the pure wood beams' ultimate bearing capacity which is 7.00kN for T20, S1 specimens' bearing capacities (Table 4) are not significantly improved. As shown in Table 4, the ultimate bearing capacity of the S2 and S3 specimens is obviously higher than that of the pure wood beams, and the range of improvement is 20% to 36%. In contrast to the pure wood beams' yield load which is 5.28kN for T20, the yield load of the S1 and S3 specimens is higher from 11% to 34%, As shown in Table 3. But the yield load of the S2 specimen has not improved significantly, because yield of S2 strength is relatively low which is shown in Table 2.

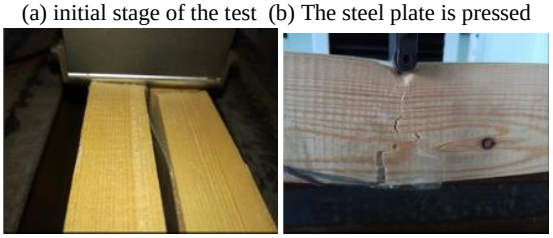
Table 5.Ultimate load of specimens with 25mm thickness of timbers (kN)

Thickness of steel plates	Ultimate load(D500)	Ultimate load(D600)	Ultimate load(D700)
S1	9.50	9.99	9.66
S2	10.10	10.69	10.57
S3	10.45	10.96	10.74

Because the yield loads of the T25 specimens have similar analysis results with T20 specimens, therefore no longer to say more about it. In contrast to the pure wood beams' ultimate bearing capacity which is 8.60kN for T25, the ultimate bearing capacity of the specimens is higher from 10% to 27%, as shown in Table 5.

3.3. Analysis of the damage process

(a) initial stage of the test (b) The steel plate is pressed



(c) Instability of steel plate (d) Destruction of wood member

Fig.6. Sample deformation during the test

At the initial stage of the test, the specimen shows linear elasticity and this time steel plate has not been pressed yet, as shown in Fig. 6(a) and Fig. 5. As the load increases, the deflection becomes larger and the steel plate begins to load as shown in Fig. 6(b). In this stage, the specimen stiffness increased than before, this can be seen in Fig. 5. The load continues to increase and the gap between the two wood members expands (Fig. 6(c)), which means that the steel plate began to lose lateral stability and the specimen goes into the strengthening stage (Fig. 5). Finally, the specimen is damaged, because of the fracture of wood member (Fig. 6(d)). Because spacing between two bolts is large, which lead to steel plate instability is easy to occur, so there was no significant correlation between Bearing capacity and spacing of bolt.

4. Conclusion

In this paper, bearing capacities of STC beams with two kinds of thickness timber, three kinds of thickness steel plate, three kinds of bolts spacing were investigated by Three point bending test. The following conclusions can be drawn: the new type STC beam has higher bearing capacity than the pure wood; Steel plate lateral instability results in that it is failure to increase the bearing capacity; The stress of bolt increases sharply and the risk of shear failure exists when the beam goes into the yielding stage; and Further studies are needed to determine the reasonable proportion of steel to wood.

Acknowledgment

This work is supported by the Beijing Key Technology Co-innovation Center of Energy Conservation and Emission Reduction, the Projcet from the teaching research project of Beijing Jiaotong University Haibin College (HBJY15007, HBJS14005), the Projcet from the National Natural Science Foundation of High-speed Rail Joint fund Projects (U1434211), the National Natural

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STRUCTURAL DESIGN AND ANALYSIS OF D BUSINESS AREA OF EVERGRANDE INTERNATIONAL CENTER

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Keywords: frame - shear wall; performance design; weak link; thermal stress

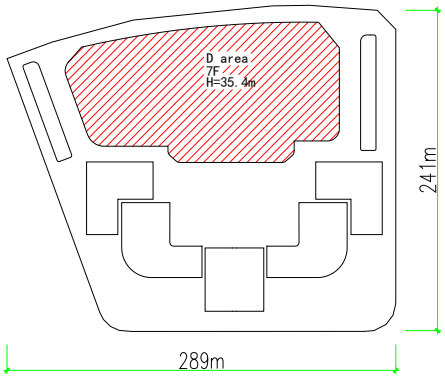
Abstract

D area of EVERGRANDE International Center with the height of 35.4m and 7-storey on the grand uses the frame - shear wall structural system and the plane dimension of 203m×104m. In this paper, firstly, the structural system selection of the main structure, combined architecture scheme and economic analysis, is explained. Then, we introduce the calculation measures and the design measures about the plane irregularity, the vertical irregularity and the other irregularity of the structure systematically. At the same time, we state the performance design method of the structure particularly. Finally, treatment measures of super-long structure are presented emphatically. Research and design results show that, by using reasonable computational measures and design measures, the irregularity of the original structure has been effectively controlled, and the structure calculation index can better meet the requirements of the standards. The structure is safe, reliable and the structural design is feasible.

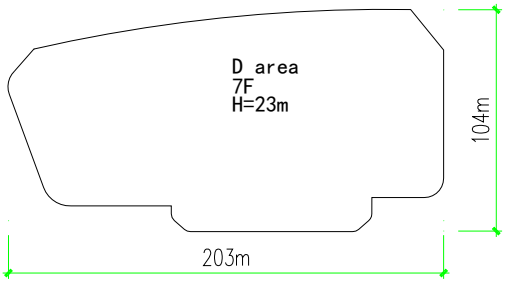
1 Introduction

EVERYGRAND International Center is located in Nanning City, Guangxi Province, Wuxiang New District headquarters, with a total construction area of about 620,000 ㎡, divided into A, B, C, D, E, F, G, H multiple structural monomer, and it is a typical urban complex buildings. Among them, D area in the entire project specific location and ground plane size as shown below.

D area of EVERGRANDE International Center with the height of 35.4m and 7-storey on the grand uses the frame - shear wall structural system and the plane dimension of 203m×104m. The sixth floor of the building has set-back model, and the D area is for commercial use. The Single-layer construction area is about 15,000 ㎡, and the total floor area of the ground is 22,000 square meters. The main body structure on the ground does not set the anti-seepage joints or expansion joints, and it is a single structural monomer. The ground structure corresponds to a 4-storey basement.



Picture1 Position indication of D area



Picture2 Ground plane size of D area

The main design conditions for District D are described as follows:

(1) Seismic fortification intensity is 6 degrees (0.05g). According to the advice of Earthquake Safety Assessment Report, the maximum value of the earthquake impact coefficient α_{max} which calculated by the small earthquake can be chosen according to the equivalent 7 degrees (0.10g), and the medium and large earthquakes and structural seismic measures are selected according to the specification.(determine the selection plan based on the discussion of the transfinite review group)

(2) Design seismic grouping is the first group; Construction site category is Ⅱ; Characteristic period of site ground T_g is 0.35s; the grade of the seismic design standard is C.

Based on the relevant geological conditions, the basic form of the D area business is chosen as the raft foundation with variable thickness.

2 Structure selection

Combined with the construction of the project and the economic analysis of the structure, the structure of the project selection and analysis are as follows.

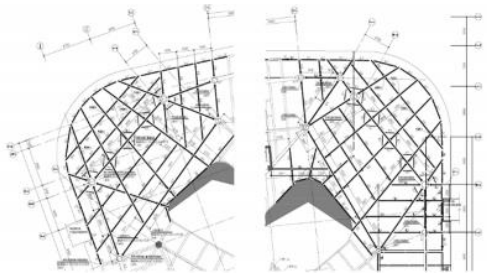
(1) The height of commercial buildings in D area is 35.4m, which belongs to high rise buildings. Combined with the arrangement of building stairs and elevators, there are more abundant space layout structure shear wall (shear wall spacing, uniform degree, etc.). The calculated ground motion parameters of small earthquakes are equivalent to 7 degrees (0.1g) (based on seismic safety assessment report). Considering the height of the main body, the layout of the building and the ground motion parameters of the small earthquake, the frame- shear wall structural system is adopted in the commercial building in D district.

(2) In the D Business District of the northwest corner and northeast corner two commercial main entrance, due to the requirements of building plane, elevation effects, in the 3~5 floor must be set large cantilever structure, maximum overhang length has more than 11m, and the 5 layer cantilever roof for practice, planting soil under high load. Considering the cantilever length and loading condition, building plane and vertical layout and building use function, to pick beam span 3~5 layer local settings, the steel structure system is used in the relevant area, and the steel column is used to support the large cantilever steel beam. Local large cantilevered steel structure is as shown below.

(3)

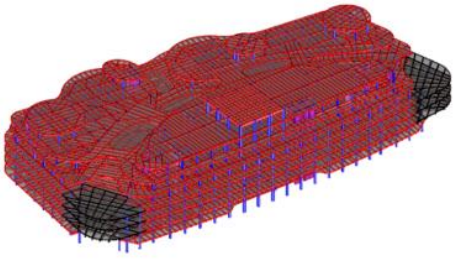
Picture3 Local large cantilevered steel structure

(4) Based on the requirements of the building layout, there are many large openings (span approach or exceed 20m) in the middle of the floor of each floor of the commercial area of the D district. Due to the

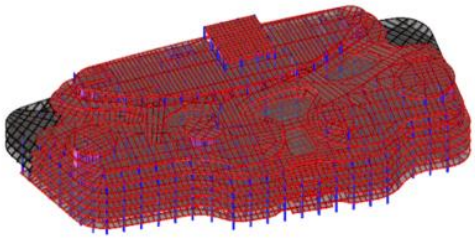


lack of vertical members in the adjacent areas of the north and south sides of the cave, a large number of large cantilevered (cantilever length of more than 5m, maximum 8m) areas are formed. In order to meet the requirements of the normal serviceability limit state, the large span concrete members are used to meet the requirements of the limit state of the structure.

The calculation model of this project is shown in the following figure (main program YJK).



a. Spatial computing model (northeast perspective)



b. Spatial computing model (northwest perspective)

Picture4 Space calculation model of D area

3 Structural transfinite (irregular) discrimination

The main structural transfinite (irregular) discrimination of the project is as follows. (Based on the design of the project time node, the basis for the identification is the “review of technical points for seismic fortification of super high rise buildings").

(1) In the 6 degree zone, a high degree of reinforced concrete frame-shear wall high rise building is the maximum applicable height is 130m, D District commercial building main structure height is 35.4m, and the height is not overrun.

(2) The maximum torsional displacement ratio of accidental eccentricity of the project is calculated to be over 1.24, which exceeds the standard limit of 1.20. Therefore, the project turned out to be torsional irregular.

(3) In the middle of the floor of each floor of the project, there is a dense and large opening area of east-west direction, and the effective width of the floor is less than 50%. Therefore, this project belongs to the floor discontinuous irregular.

(4) This project has set-back model in the 6 floor, and the indent size is greater than 25%, belonging to the elevation of the sudden change in the size of irregular.

(5) There are some irregular items such as column supported by beam in order to supporting superstructure in the atrium on both sides of the large span, large cantilevered area, and so on.

(6) There are other irregular items such as long column (through layer), etc.

(7) After review, special irregular items do not exist in this project.

So, the height of this project is not overrun. There are four irregular items, including torsion irregularity, floor discontinuity, abrupt change of elevation, individual component conversion and long column (through layer). So this project belongs to overrun high-rise building

Based on the above irregular items, the system adopts the calculation measures and design measures to control the system.

4 The calculation measures of structural overrun control

On the basis of the results of the third section of the structure, for the effective control of irregular structure, improve the seismic safety performance

of the structure, the selection of the project control measures include the following 9 aspects.

(1) Description of calculation model: Considering the seismic action of two principal axes and considering the influence of the earthquake under the action of the 5% accidental eccentric earthquake, the mode decomposition response spectrum method and elastic time history method are used to analyze the seismic response. The whole calculation model of this project takes the basement roof as the fixed end. At the same time, for the first floor of the middle of the East and the west of the continuous large openings, adding a layer of the underground as a fixed end of the calculation model, as a large opening and adjacent areas of the vertical component envelope design model.

(2) The elastic response spectrum analysis of small earthquakes: under the action of small earthquake, the structure is in the elastic state, and the bearing capacity and deformation of the structure meet the requirements of the code. The main calculation program is YJK1.5.3, which used to control the main design index of the structure (the displacement ratio of the main body, the ratio of shear to weight, the ratio of overturning moment, the ratio of lateral stiffness, the ratio of torsion to translational cycle, etc.) and calculate the strength of the member. The supplement calculation program is MIDAS BUILDING, which used for the design of the main structural indicators review, performance-based design and strength envelope of key parts.

(3) The elastic time history analysis of small earthquakes: according to the 5.1.2 code for seismic design of buildings, the commercial area of D should be analyzed by elastic time history analysis method. According to the elastic time history analysis of multi event earthquake which is based on the selection of the corresponding seismic wave to the structure, the shear force at the bottom of the structure calculated by each time history curve is not less than 65% of the bottom shear force calculated by the vibration mode decomposition response spectrum method, and the average value of the shear force at the bottom of the structure calculated by the 7 time history curve is not less than that of the 80% bottom shear force obtained by the modal decomposition response spectrum method. Then compared with the calculation results of time history analysis and modal decomposition response spectrum method, the mean value of the calculated time history curve of seismic action and the larger value of the calculated results of the mode decomposition response spectrum method are used in the design. As the standard plane of the C tower is a typical L shaped plane, the elastic time history analysis of 2 groups of small earthquakes is carried out according to the orthogonal direction of the axis network and the direction of the most unfavorable earthquake input.

(4) Performance-based design. According to the performance objectives (see section sixth), in order to ensure that the relevant components can meet the design requirements of the performance objectives, we have carried out the calculation of the large earthquake in the quasi elastic medium.

(5) Elastic-plastic static analysis. According to the requirements of the code, the elastic-plastic deformation of the weak layer under the action of the rare earthquake should be checked, and the elastic-plastic deformation of the commercial area in the D area under the condition of rare earthquake is checked by using the elastic-plastic static analysis method. In view of the D business district for the frame shear wall structure system, the number is not high (Level 7), large plane size (203mx104m), the lateral rigidity of the structure is great. So the static analysis of D Business District of plastic, in addition to checking under the rare earthquake elastic-plastic deformation (far less than the specification limits), as the weak parts of the structure of the large earthquake, and to strengthen the effective basis of design in the construction phase.

(6) Weak connection correlation.

A. The calculation model of the North South sub block model and the zero stiffness plate of the North South sub tower and the weak connection zone is added as the envelope model of the strength design of various structural members.

B. The stress analysis of the slab finite element model is carried out under the action of small earthquakes and moderate earthquakes, which is based on the model that the slab is not cracked and the reinforcement of the floor slab is not taken as the envelope model.

(7) The basement roof as the fixed end checking; adding a layer of underground embedded computing model, on the first floor D District Central East-West continuous structure with big openings and adjacent region component according to the underground embedded supplement calculation results and underground embedded related structural measures envelope design.

(8) Supplementary calculation of MIDAS BUILDING (review of the whole structure calculation index).

(9) Checking the comfort of large span and large cantilevered steel structure at the entrance of the East and west.

5 Overrun control design measures

On the basis of the results of the third section of the structure, for the effective control of irregular structure, improve the seismic safety performance of the structure, the design of the project overrun control measures include the following 10 aspects.

(1) The calculation results of the multiple software (YJK, MIDAS BUILDING, etc.) is used to control the whole structure's index.

(2) According to the seismic parameters and specification requirements provided by the safety assessment report, A_{max} provided by the safety assessment report are used for the small earthquake calculation. The remaining parameters are in accordance with the standard. The medium and large earthquakes and seismic measures are carried out according to the standard (6 degrees, 0.05g).

(3) Using the performance-based design method, the overall performance target of the structure is controlled according to the standard C level. The specific seismic performance of the key parts and key components is adjusted according to the structural importance degree.

(4) For the large-span and large cantilever girder of the local steel structure at the entrance to the east and the steel frame of the concrete-filled steel tubular frame, the seismic grade is higher than the specification level, that is, from the four to three.

(5) Review of large span and large cantilever parts of the steel structure to meet the specifications for the normal use of the limit state (deflection-based) and comfort check the relevant requirements.

(6) For the D area of the commercial structure of the reverse irregularities such as irregular items, combined with the construction of stairs, the elevator arranged evenly and reasonably set the shear wall, the effective control structure to reverse, control structure torsional displacement ratio of not more than 1.30.

(7) For the non - continuous irregularities of commercial floor in area D, the following measures are used to strengthen the local weak connection area.

A. For the local floor of the weak connection area to take the ribs and the local floor thickening measures (according to the specific situation thickened to 150) to be strengthened.

B. According to the results of the finite element analysis of the seismic stress of the floor, the envelope design of the floor reinforcement is carried out according to the principle that the small earthquake floor does not crack and the center of the middle plate does not yield.

C. According to the north and south sub-tower and weak connection area zero stiffness plate supplement calculation model results, all kinds of structural components are designed by the principle of strength envelope.

(8) For the D area of commercial structure facade size mutation, individual component conversion and other vertical irregular terms, provide reasonable performance objectives for relevant large span, large overhangs and conversion class-related components to ensure that they meet the relevant requirements for specification and performance design.

(9) Support the east and west entrance of the large span, large cantilever beam frame column are made of concrete filled steel tubular columns. This can effectively improve the carrying capacity and ductility, to ensure that it can meet the specifications and performance-related design requirements.

(10) The following measures were used to strengthen the pillars, which mainly caused by the floor of the hole caused by large holes in

the commercial area of D.

A. The performance-based design method requires the bearing capacity of the pillars to meet the seismic design requirements.

B. The control of the through-column reinforcement is not less than that of the adjacent non-perforated column (the steel structure column has a control cross-section and the stress ratio is not less than the adjacent non-perforated column).

6 Performance design

In order to effectively cope with the structural overrun condition, the performance selection method of the project is selected. The overall performance target of the structure is controlled according to the standard C level. The specific seismic performance target of the key parts and key components is adjusted according to the structural importance degree. The specific content of the performance-oriented design can be divided into the following three parts.

(1) Performance design of common horizontal components and common vertical components. The specific control forms are shown in Table 1.

(2) In line with the actual situation of this project, the definition of "key component" is set up for the effective control of the component, the improvement of the performance and the goal of strengthening the performance. The so-called "key component" refers to a component that does not belong to "critical construction" but is improved according to the degree of importance of the

(3) project and the improvement of seismic fortification performance standard. The specific control forms of the key components are shown in Table 2.

(4) Key construction of the performance design. The specific control forms are shown in Table 3.

According to the structural calculation and design results, through the appropriate calculation and design measures, the performance of the project design goals can be achieved. Through the selection of performance-based design methods, the structure of the seismic safety performance to achieve a comprehensive upgrade, focusing on strengthening the design goals.

7 Conclusion

(1) The project is a high degree of non-limit, with four general irregular, no special irregular high-rise buildings.

(2) This project combined with the architectural scheme and structural economy requirements, comprehensive selection, selected the framework - shear wall structure system. Local large suspension area using a steel structure system.

(3) The effective control measures and design measures are adopted in the structural design, which can reduce the adverse effect caused by the overrun, which makes the structure have good seismic performance, and the calculation results meet the requirements of the existing norms and procedures.

(4) In this project, a performance-based design method is adopted in the structural design, which has achieved a comprehensive improvement, focusing on strengthening the design goal, and the comprehensive seismic performance of the structure has been greatly improved.

(5) The project is safe, reliable and structural design is feasible.

yield load is great, but the influence on ultimate bearing capacity is not obvious.

Performance target 1 Table 1

Fortification level	Frequent Earthquake	Fortification intensity	Rare earthquakes
Performance description	Intact	Repairable	Not collapsed
Allow interlayer displacement	1/800	-	1/100
Frame beam	Flexibility to meet the specification	Shear capacity to meet the non-yield design requirements	Meet the specification
Ordinary vertical members (bottom reinforcement frame, shear wall strict control)	Flexibility to meet the specification	Positive cross section bearing capacity to meet the non-yield design requirements; Shear bearing capacity to meet the elastic design requirements	Meet the requirements of JGJ3-2010 formula 3.11.3-4

Performance target 2 Table 2

Fortification level		Frequent Earthquake	Fortification intensity	Rare earthquakes
Performance description		Intact	Repairable	Not collapsed
Allow interlayer displacement		1/800	-	1/100
Important component	Long column (through layer)	Flexibility to meet the specification	Normal bearing capacity and shear bearing capacity to meet the elastic design requirements	Meet the requirements of JGJ3-2010 formula 3.11.3-4
	the large span beam(above 18m)	Flexibility to meet the specification	Positive cross section bearing capacity to meet the non-yield design requirements; Shear capacity to meet the elastic design requirements (consider the vertical earthquake)	
	Large cantilever beam(above 5m)	Flexibility to meet the specification	Positive cross section bearing capacity to meet the non-yield design requirements; Shear capacity to meet the elastic design requirements (consider the vertical earthquake)	
	A frame column that supports large span beams and large cantilever beams	Flexibility to meet the specification	Positive cross section bearing capacity to meet the non-yield design requirements; Shear bearing capacity to meet the elastic design requirements	

Performance target 3 Table 3

Fortification level		Frequent Earthquake	Fortification intensity	Rare earthquakes
Performance description		Intact	Repairable	Not collapsed
Allow interlayer displacement		1/800	-	1/100
Key components	Large span, large cantilever beam at the east and west entrance	Flexibility to meet the specification	Bearing capacity to meet the elastic design requirements (consider the vertical earthquake)	Meet the requirements of JGJ3-2010 formula 3.11.3-4
	Concrete Filled Steel Tubular Columns with Large Span and Large Cantilever Steel Beams	Flexibility to meet the specification	Bearing capacity to meet the elastic design requirements	
	Local pillar conversion beam and its support frame	Flexibility to meet the specification	Normal bearing capacity and shear bearing capacity to meet the elastic design requirements	

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Application of Low Yield Point Steel in Structural Seismic Design

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Key words: Low yield point steel; Seismic design; Dampers.

Abstract: With the development of steel structure and the improvement of seismic requirements, energy dissipation design has become a development direction of building seismic. Low yield point steel has good plastic toughness and narrow intensity fluctuation range, and easy maintenance and replacement, low cost, so in the seismic damping component has a significant advantage. In this paper, the performance requirements and application status of low yield point steel in seismic structure are introduced. The application of low yield point steel in elastic energy dissipation structure is proposed, which provides the basis for the application of low yield point steel.

1. Introduction of low yield point steel

Low yield point steel is the yield strength under 235MPa (ultra low yielding point steel finger 100MPa below), yielding strength within a narrow range (ys ± 20MPa), without apparent yield point, and ductility hysteresis performance is good (LYP100: elongation exceeds 50%; LYP235: Energy dissipation material with elongation exceeding 40%.

Low yield point steel has the characteristics of low yield strength, stable strength and strong deformation ability. When the earthquake and the wind perk, the damping device with low yielding point steel is used to enter the yielding, the yield strength is lower and relatively stable before the deformation of the main structure, and it has sufficient plastic deformation ability to absorb large quantities of seismic energy.

1.1 Analysis of mechanical properties of low yield point steels

Low yield point steels are used as energy dissipation components in aseismic structures. The energy dissipation component needs to be surrendered before the other components, and the effect of absorbing energy dissipation by repeated plastic deformation. At present, foreign iron and steel Company by continuously reducing the carbon content and alloying element content of steel, the development of many different specifications of products, the yield strength is generally 100~225N/mm2, and the damping steel is different from the traditional structural steels which work mainly in the elastic range, requiring plasticity under very small strain and sufficient bearing capacity and large plastic deformation capacity, low yield strength deviation, and good lower cycle fatigue performance. In response to the above-mentioned requirements, Nippon Steel Company first developed the yield strength below 100MPa in the 1989 extremely low yielding point of the steels [1-2], its main mechanical properties, as shown in table 1-1, have taken the lead by 1998 to make different types of dampers and applied to high-rise building structures with the yield strength of steel 100MPa and 225MPa. In 1998 Japan's Kawasaki Iron has also developed similar steel. After the founding of the Japanese Steel Alliance (JFE), the low yield point steel was incorporated into the construction steel catalogue. The grades are LY100, LY160, LY225, and their main mechanical properties are shown in table 1-2[3], the stress-strain curves of the low yielding point steel with the traditional soft steel (mild steel) are shown in Fig. 1-1[4]. It can be seen that, whether from the yield strength or ductility, low yield point steel is far superior to traditional steels, is the ideal material for the manufacture of dampers. Good fatigue performance is necessary for aseismic low yield point steels, based on this requirement, Baosteel researcher Fengming Song [5-6], etc., has studied the fatigue performance of low yielding point steels with a large number of tests. Anti-seismic design has such a requirement that if the material meets the requirements of low-cycle fatigue, it must have more than 100 cycles under the cyclic loading of different amplitude. According to the research results of Fengming Song and other scholars, LYP100 and LYP160 meet the requirements of seismic design, i. e., satisfying fatigue performance. According to the relevant test, the steel used to make damper has the following characteristics in its mechanical properties:

(1) Yielding point is very low, and has strong deformation ability;

(2) The yield is relatively small, the carrying capacity of repeated loads is obviously improved;

(3) Low-cycle fatigue performance of steels with very low yield points is basically identical to that of ordinary mild steels;(4) The effect of strain velocity on

Table 1-1 Mechanical properties of low-yield steels produced by NSC

Grades	Thickness/mm	Yield strength/Mpa	Tensile strength/Mpa	Elongation rate/%
BLY-100	6~50	80~120	200~300	≥50
BLY-225	6~50	215~245	300~400	≥40

Table 1-2 Mechanical properties of low-yield steels produced by JFE

Grades	Yield strength /MPa	Tensile strength /MPa	yield ratio/%	Elongation/%	impact property Temperature/℃	impact property A_{kv}/J	Thickness/mm
LY100	100±20	200~300	≤60	≥50	0	≥27	6~50
LY160	160±20	220~320	≤80	≥45	0	≥27	6~50
LY225	225±20	300~400	≤80	≥40	0	≥27	6~50

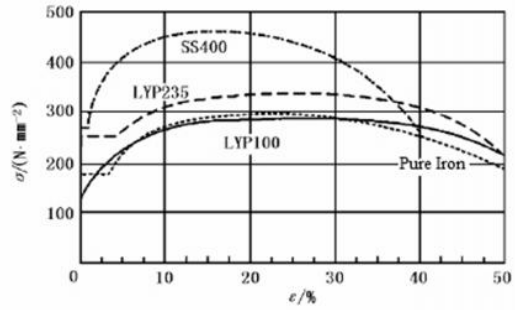

Fig. 1-1 Comparison of stress-strain relationship between low yield point steel and mild steel

2. Research and application of low yield point steels

Low yield point steel in structural energy dissipation design is mainly used for the production of soft steel dampers and non-bonding support, steel shear walls and other energy dissipation elements, the corresponding technology in the new building energy dissipation and existing building seismic performance improvement has been practical application.

2.1. Development and application of soft-steel damper

Soft-steel damper is a kind of passive energy dissipation device in structure control, its damping mechanism is: in the earthquake or wind vibration, the structure of plastic deformation, the soft steel damper first into the yield, the use of the metal itself hysteresis characteristics dissipation energy, so as to achieve the purpose of damping. Low yield point steel is the main material for making soft steel dampers. At present, there are mainly X-type soft-steel dampers, triangular soft-steel dampers, open-hole soft-steel dampers, ring soft steel dampers, etc., compared with other damping devices, soft steel damping apparatus has the advantages of simple structure, economical durability, convenient replacement and strong reliability after earthquake.

The passive energy dissipation measures can improve the aseismic performance of the structure, and the shear plate damper as a passive energy dissipation measure is a device that uses the shear elastic plastic deformation generated by the steel plate in the plane to dissipate the energy. It has the following advantages: Hysteretic performance stability, initial stiffness, fatigue performance is superior, easy to draw, precisely because of its advantages, domestic and foreign scholars have done a lot of research, and the use of such energy-consuming equipment to some practical engineering practice. The shear plate damper consists of two parts: the core plate and the flange plate, wherein the core plate is made of ordinary mild steel or low yielding point steel, flange plate is made of ordinary strength steel, as shown in Fig. 2-1 as the basic tectonic form and deformation mode of shear plate damper.

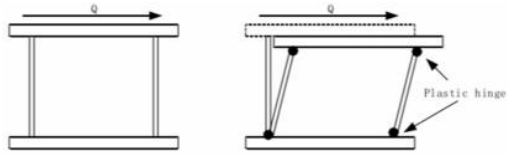

Fig. 2-1 Deformation mode of shear panel dampers

The height of the shear plate damper is generally 1/3~1/10, the shear deformation of the damper is slightly smaller than the interlayer displacement, but the shear strain is significantly larger than the interlayer displacement angle. Fig. 2-2 illustrates several mounting forms of the shear plate damper in the structure. The shear plate damper and the surrounding frame are connected by a short column or support with a large stiffness, and a short column connecting the damper to the main frame is called an interstitial column. The damper and the structure frame are arranged through the supporting connection, as shown in Fig. 2-2 (a), which can be mounted on the bottom of the interlayer, the middle of the interlayer and the top of the interlayer respectively. The damper and the structure frame are connected through the column as shown in Fig. 2-2 (b), the damper is generally installed in the middle of the interlayer, and the damper can be spliced to form a composite damper as needed, so the dosage of the damper can be adjusted according to the actual situation.

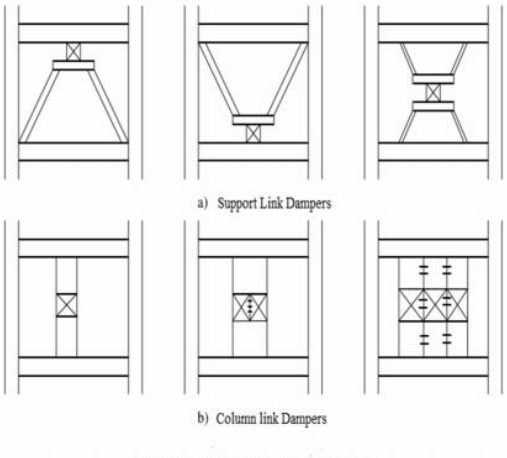

Fig. 2-2 Installment style of panel dampers

3. Conclusion

The traditional structural aseismic system is the damage of the main structural components, the disaster consequence is serious and the structural repair is difficult. The development of low yield point steel materials and application research provides a new energy dissipation system for the actual engineering structure, and the energy dissipation damper produced by low yielding point steel absorbs seismic energies through hysteresis deformation, which can effectively protect the safety of the structural subjects. As a new steel grade for aseismic steels, low yielding point steels will have broad application prospects.

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THE INFLUENCE OF CRD METHOD ON THE EXISTING STATION

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Key word：new tunnel, the existing station, CRD method, vault, major structure.

Abstract: Analyzing the new subway tunnel through existing station,get the rule that new subway tunnel affects upper existing station. FLAC3D 4 finite difference software is used for 3D numerical simulation,The settlement caused by CRD method for existing upper station is studied.The maximum settlement value and the maximum settlement rate of the existing station are obtained.This conclusion has guiding significance to engineering construction.

1 Introduction

The subway is the main transportation forces in Beijing, take traffic most of the area, according to the "Beijing city rapid rail transit construction planning", it is expected to reach thousands of kilometers to the Beijing area in 2020 the total mileage of subway.See the next subway plan we can, the future city rail transit network will be realized, it will be more of the intersection and transfer station, this will be a large number of new subway tunnel crossing the existing subway station. So in the process of tunnel excavation, the soil around the tunnel will appear plastic failure caused by soil subsidence and in the affected soil near the deformation of adjacent buildings may be deformed, in this case, with what kind of construction, to the existing structure deformation control in the specified range, so a large number of domestic and foreign scholars and experts to do research in this area. Gu.Koo and Huwang of South Korea for a new tunnel under the high-speed railway tunnel by two-dimensional finite element analysis of [1].Adachi, Kojioa by using the finite element method of tunnel excavation in soil surrounding stress and displacement, stress of lining structure, the construction step of the interaction [2]. Bai Haiwei (2014) of the new tunnel Undercrossing the existing tunnel, he first divided the wear type, then according to the existing tunnel theory analysis this kind of problem in the longitudinal settlement, stress mathematical expressions, the general calculation method and the way of resolving this problem in theory, comparison of monitoring data at last with the actual project, verify the correctness of the [3]. theory combined with the domestic and foreign scholars on the study of new tunnel under the existing building effects, using CRD three-dimensional numerical method in tunnel construction. The influence of the construction of the new tunnel on the vertical deformation of the existing tunnel excavation is studied.

1 Project overview

1.1 Engineering background

According to the Beijing metro line 14 a new tunnel under the station as the engineering background, this section of the street is located in a busy traffic in the Beijing area, next to many large office buildings and high-rise residential, on both the road and the building near the station, the settlement control is very strict. The construction of this section the single hole single line spacing, 15m~18m, wear along a highway overpass, the existing railway station. The design of interval mileage: left K12 015.200~ left K12 946.800 (short chain 0.173m), right K12 015.200~ K12 946.800 right left right, length 931.427m, length 931.600m. The existing station is from north to south, the main part is 182m, the rail surface elevation intermediate position of the station to 16.761. station is divided into three main parts, South and North three layer excavation of frame structure, both ends of the length 118.2m, width 18.9m; the middle part is excavated, structure form, single column double arch length 61.2m, wide 18.3m, single Zhuchang 1m, width 0.7m, column spacing is 6m.

1.2 Engineering geological survey

1.2.1 Formation condition

The region is located in the eastern plain area of Beijing City, landforms and ancient gold river Oxbow River on both sides of the platform, the East West High low. The alluvial layer in late the maximum depth of the project reconnaissance formation is 60m, the survey area of the formation is divided into thear tificial accumulation layer, quaternary Holocene alluvial layer in late Pleistocene alluvial the three layer layer, with its row from top to bottom points shown in the table below.

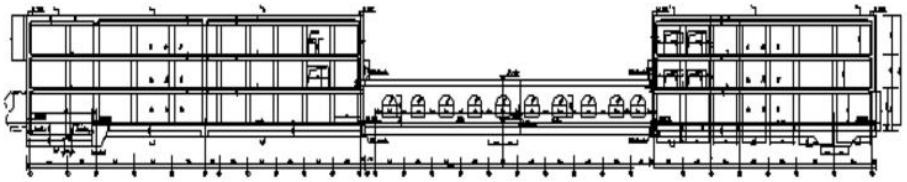


Fig1-1 Main structure of existing station

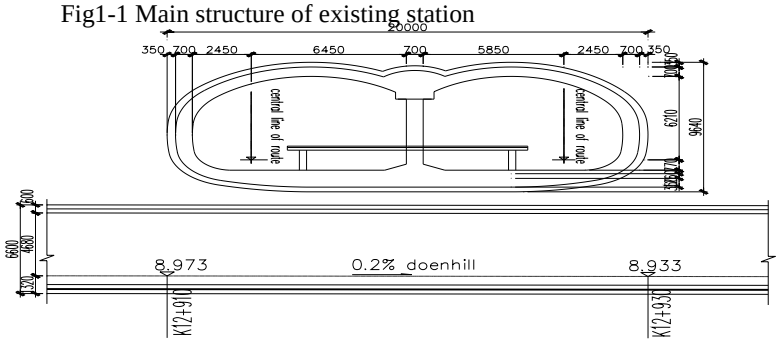


Fig.1-2 Longitudinal profile of the interval and the existing line

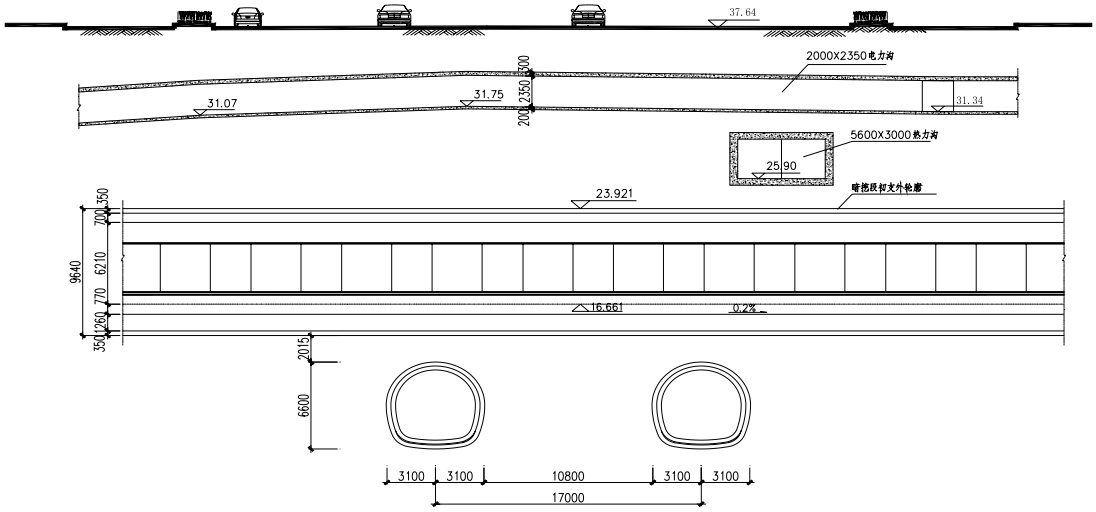


Fig.1-3 Cross section of the interval and the existing line

Table 1-1 Stratigraphic detail table

Artificial fill layer	Silt fill 、miscellaneous fill $_1$
Quaternary Holocene alluvial deposit	Silt soil 、Silty clay $_1$、The fine sand $_3$、Medium coarse sand $_4$
Late Quaternary alluvial deposit	Round grave 、Medium coarse sand $_1$、The silty clay 、The 1 clay $_1$、The silt $_2$、medium coarse sand $_1$、Fine sand $_2$、Silty clay $_4$、The silty clay 、The clay $_1$、The silt $_2$、Fine sand $_3$

1.2.2 Hydrological profile

According to the survey data in this section, there are three layers of groundwater in this section, respectively, phreatic water (two), confined water (three) and confined water (four). We connot find perched water in the survey , but because of rain, individual pipeline leaks and other force majeure factors, does not exclude the existence of local perched along the possible details in the table below.

Table 1-2 Groud water level

Groundwater properties	Water level/head buried deep (m)	Water level/head elevation (m)	Aquifer and characteristics
Phreatic water (two)	11.51~12.09 12.40	21.20~21.65 18.68	fine sand $_3$、medium coarse sand $_4$、gravellayer 、medium coarse sand layer $_1$
Confined water (three)	16.32~17.41 19.00	25.21~26.94 25.28	medium coarse sand layer $_1$、fine sand $_2$、fine medium sand $_3$
Confined water (four)	21.63~22.34 26.24	29.52~30.68 32.46	silt $_2$、silt $_2$、clay 1、silty clay 、clay $_1$

Table 2-1 Physics mechanical parameters of stratum

Formation Number	Formation Name	Elastic Modulus(MPa)	Cohesion(KPa)	Friction angle(°)	Poisson ratio	Density (KM/m^3)
1	Miscellaneous fill	-	0	12	-	1700
2	Silt	19.5	20	26	0.3	1900
3	Silty clay	66	25	18	0.3	2000
4	Fine sand	111	0	38	-	2050
5	Medium coarse sand	138	0	35	-	2050
6	Round gravel	150	0	40	0.23	2100
7	Clay	105	35	20	-	2000
8	Medium fine sand	120	0	35	-	2100

2.3 Model establishment and result analysis

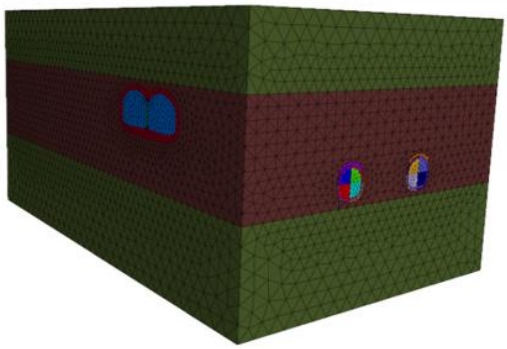


Fig.2-1 The numerical calculation model

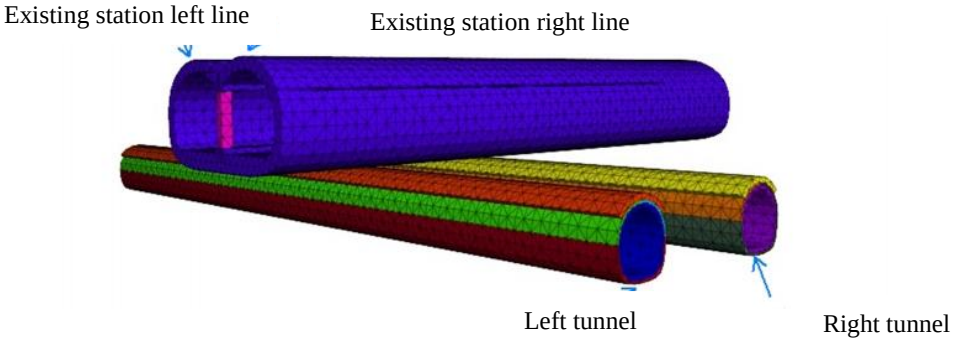


Fig.2-2 The position relation of the existing ewly built tunnel

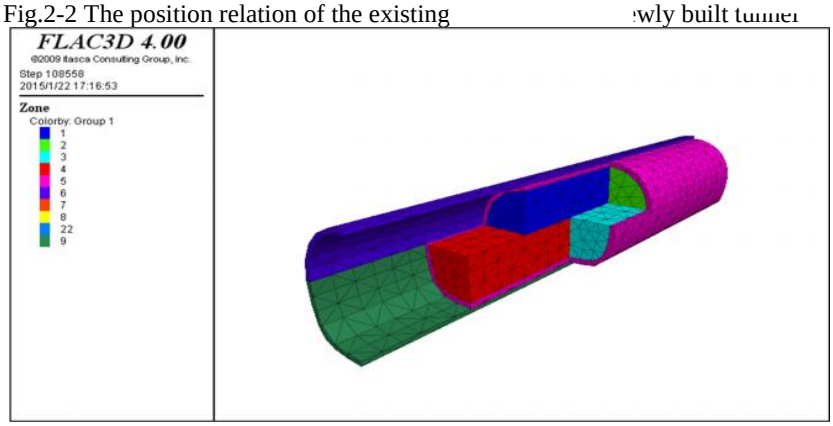


Fig.2-3 The schematic diagram of the numerical simulation of tunnel excavated by CRD Method

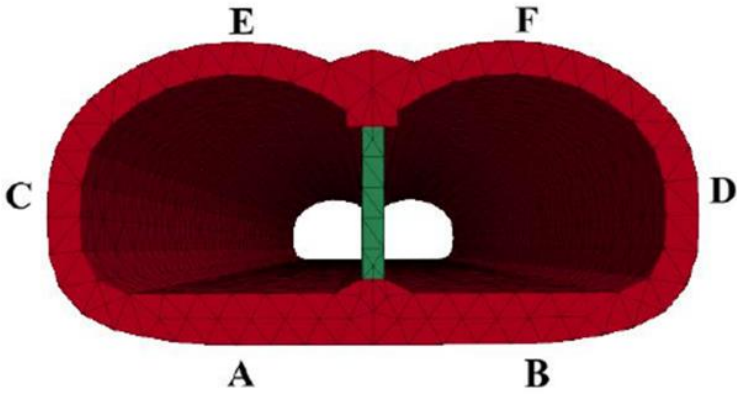
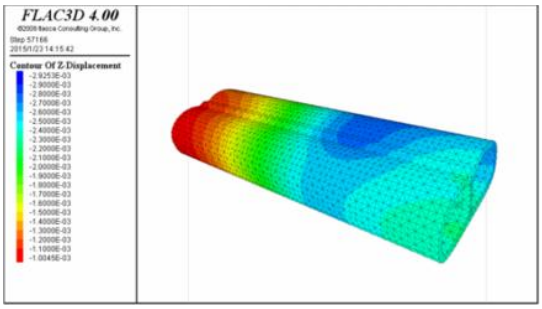
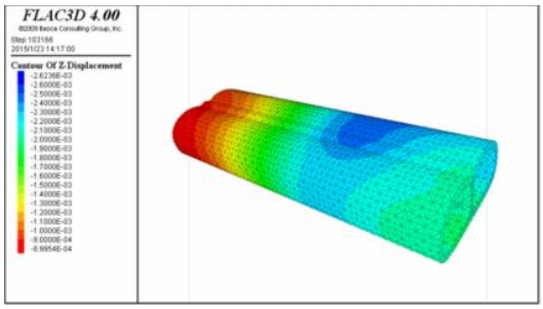


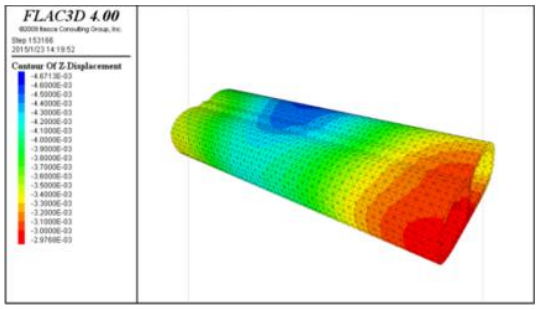
Fig.2-4 The monitoring points location of the station structure



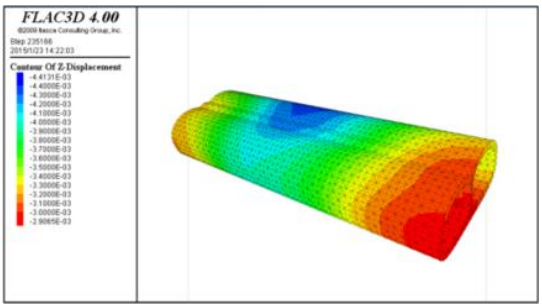
a) The right tunnel excavation completed (Secondary lining is not installed)



b) The right tunnel excavation completed (Secondary lining is installed)



c) The left tunnel excavation completed (Secondary lining is not installed)



d) The left tunnel excavation completed (Secondary lining is installed)

Fig.2-5 Settlement nephogram of the existing structures at each stage of tunnel excavated by CRD Method

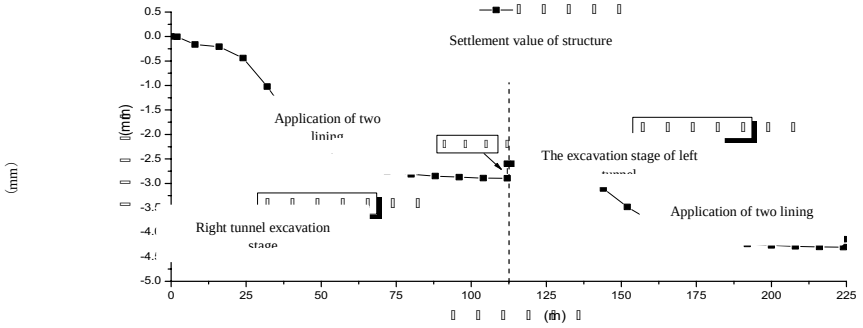


Fig.2-6 The settlement cu the right tunnel vault corresponding position

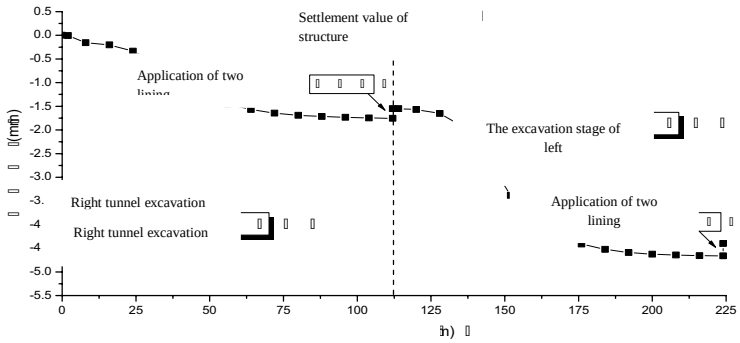


Fig.2-7 The settlement curve of left tunnel vault corresponding position

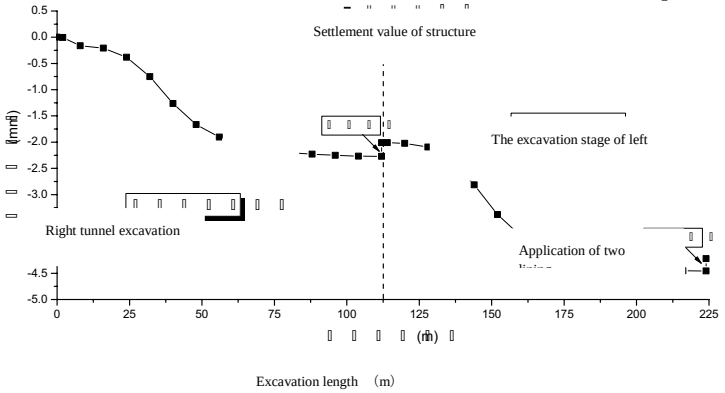
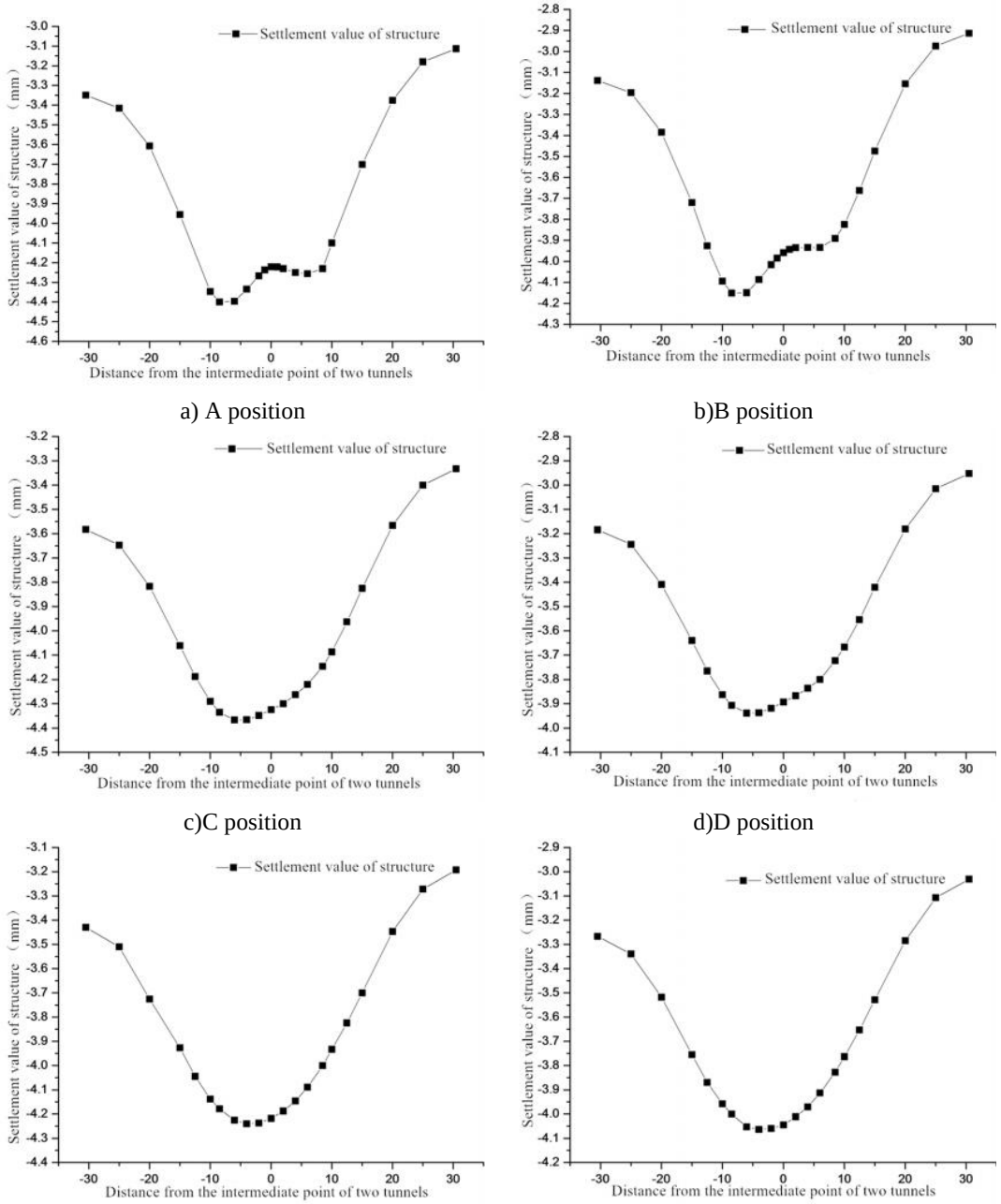


Fig.2-8 The settlement curve of A monitoring of the Intermediate point of two tunnels corresponding position



a) A position b)B position

c)C position d)D position

e)E position f)F position

Fig.2-9 The vertical deformation curves of the each monitoring position after the construction of both tunnels to be finished

As can be seen from the above three diagrams, with the excavation of the new tunnel, will cause settlement of A point, and with the excavation, the overall look is growing, but also can see when the excavation to close to the existing station is under the moment, the subsidence rate of a point is the largest, along with the continuation of excavation, the subsidence of A point tends to stabilize. After the excavation of the tunnel after the second lining, A position of subsidence displacement decreased, this full description of the two lining of the maintenance of the cavity has played a positive role, and the structure has a certain uplift effect.

From the above illustration, we can see that the excavation of the tunnel will illuminate the monitoring points of 6 locations, and the greater the settlement of the intermediate position of the existing station, 6 positions at the maximum settlement of the points are in the vicinity of the -5m, the difference is that a, b position of the point of the middle of the settlement than the two sedimentation trough. The subsidence curves of the

sides of the settlement of the lower and minus 5m, sedimentation curve at the bottom of the present "W" shape is the double peaks

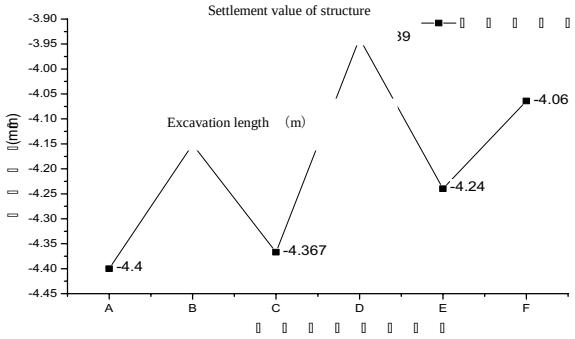


Fig.2-10 The maximum vertical deformation of the each monitoring position of the station

From the above, we can see that the rear floor, rear vault, rear arch waist of the maximum settlement is greater than the front of the bottom plate, the front vault, the former arch waist, so that the existing station in the excavation after the completion of the backward tilt situation.

3 Conclusion

The following conclusions can be obtained by simulating analysis of tunnel excavation by CRD method：

(1) The current tunnel excavation to the existing station, the existing station of the maximum subsidence rate, with the continuation of the tunnel excavation slowly away from the existing station, the settlement rate gradually decreased, and ultimately achieve stability. After the excavation of the tunnel is made of two lining, the structure of the station has a certain lifting effect.

(2) The influence of the excavation of the right tunnel on the existing structure of the right tunnel vault is larger than the corresponding position of the left tunnel vault, and the corresponding position of the left tunnel to the left tunnel vault is larger than the corresponding position of the right tunnel vault. The influence of the excavation of the left and right tunnels on the intermediate position of two tunnels is basically consistent.

(3) The maximum settlement of the existing station is basically located near the middle part, and the settlement of the rear floor, vault and arch waist is larger than that of the former backplane, vault and arch waist.

Through the use of FLAC3D 4.0 software to the tunnel under the construction method of simulation analysis results, the actual project has certain knowledge and reference significance. However, because in the process of simulation of soil and surrounding rocks and so on have some simplified treatment, and the actual results may be different, in the actual project should be arranged monitoring points to always monitor the existing station subsidence situation, by combining on-site actual monitoring and software analysis to guide the actual project.

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COLLISION INSPECTION ANALYSIS OF ASSEMBLED BUILDING BASED ON BIM TECHNOLOGY

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Keywords: Assembled structure; BIM（uilding Information Model）; Collision check; Collision optimization

Abstract: For reducing engineering accidents and improving construction efficiency. Based on BIM technology, this paper carries out collision checking and structural optimization of assembled structure, utilizes different kinds of collision checking, discusses the optimal collision inspection scheme, and carries on the in-depth study on the classification principle of collision inspection and the implementation steps of collision inspection. It provided theoretical basis for solving architectural problems in advance and improving construction efficiency.

1. Introduction

National Thirteen-Five Plan clearly stipulates: To promote green city, green building, green building materials development, we must vigorously promote the assembly building and BIM technology collaborative development. The development of assembling building is the only way to achieve green construction and can reduce construction waste and pollution of buildings. However, the development of assembled architecture is inseparable from the support of BIM technology, and BIM technology can share and transmit the data information in the whole lifecycle process of project planning, operation and maintenance, and the project units can make the correct decision on various architectural information, thereby improving the construction efficiency and reducing the project loss. Therefore, the combination of assembled architecture and BIM technology must be a technological revolution in the construction industry.

Fig.1 Status of BIM

2. The Introduction of Collision Inspection

Collision inspection is the collision contact problem that may arise in the professional or between different professionals during the construction process by using BIM technology (for example, the Clash Detective function of Navisworks) before the project construction. Collision checking according to the contact type is divided into two kinds of "hard collision" and "soft collision". "Hard collisions (shown in Figure 2)" mainly refers to the overlap between the entities, "soft collision (Figure 2)" also known as "gap collision" mainly because the actual spacing between entities is less than the prescribed minimum spacing to preventing the installation of the construction site.

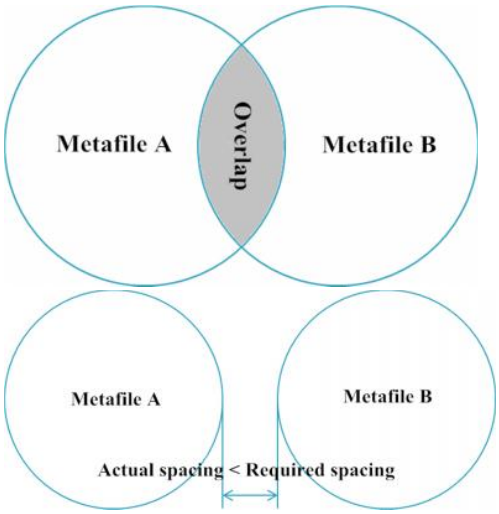

Fig.2 “Hard clash” and “Soft clash”

Collision checking is divided into two kinds of "static collision" and "dynamic collision" according to the time relation of collision. Static collision refers to the collision which is usually said in the situation, and in the actual projects, dynamic collision is the most common in the actual construction. Dynamic collisions (Fig. 3) are the components（for example, prefabricated laminated plates and ladder segments）that may collide with each professions during the hoisting process [2].

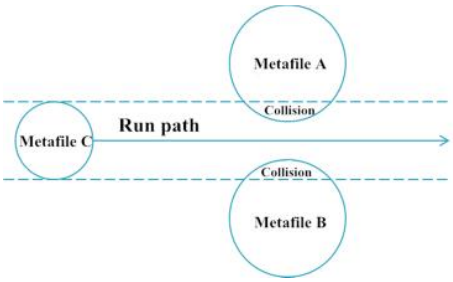

Fig.3 Kinetic clash

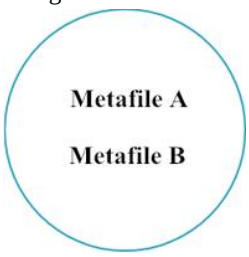

Fig.4 Overlapped clash

3. Classification principle of collision inspection

The reason for classifying collision inspection is to improve the accuracy of model collision checking and save the analysis time and avoid excessive avoidance collision point to affect the results of the analysis. If take all of the collision check one-time for model, it will produce a lot of collision points, then there is no rule to follow and seriously affect the efficiency and accuracy of post-processing, so the classification principle of collision research is very important.

The general principle of classifying collision inspection is the crossover combination of different regions. Under the basis of the principle of collision analysis, it can effectively improve the accuracy and efficiency of collision inspection, reduce the workload of post-processing, and the classification should be moderate, coarse classification can reduce accuracy, and meticulous classification will increase the workload (Figure 5), so the appropriate classification is essential.

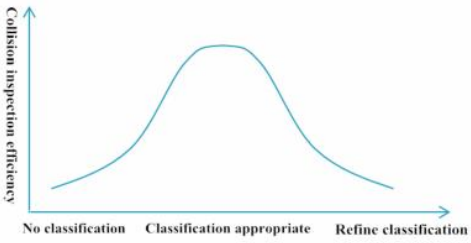

Fig.5 Efficiency curve of clash detective

Professional classification is according to different professional models to classify, such as architecture, structure, water supply, drainage, heating ventilating, and so on. The classification is conducive to processing and accountability. The regional classification is classified according to the construction layer, the component category and the material kind. For example, in the assembling structure, the most intuitive is by the layer classification and then by prefabricated beams, prefabricated columns, laminated plates and so on.

In collision detection, the profession and the region are mutually overlapping in the actual project. A professional classification includes multiple regions types, as well as one area classification includes multiple professional types. Therefore, before modeling, the principle of classification must be clarified, and the priority order is determined. Therefore, before modeling, the principle of classification and the priority order must be clarified.

Based on the above discussion, we draw the following classification pattern for the assembly structure:

(1) Collision inspection between different professions. First of all, the software conduct preliminary collision checks for two majors. Then according to the number of collision point,we adopt different treatments. If the number is few, it can be optimized by subsequent processing. If the number of many, we need to refine the classification: according to floor, the software conduct collision checks for two majors, then conduct collision checks for two majors between different floors, as figure 6.

(2) Collision inspection of the same profession. Firstly, the software conduct preliminary collision checks for two kinds of components of the same profession. Then according to the number of collision point, we adopt different treatments. If the number is few, it can be optimized by subsequent processing. If the number of many, we need to refine the classification: according to floor, the software conduct collision checks for two components, then conduct collision checks for two components between different floors, as figure 6.

(3) The collision inspection of the component itself. First, the software selects two parts of the component, for example, the embedded parts and the embedded steel bar of precast beam, and then the component carries on its own collision inspection according to the component model, as figure 6.

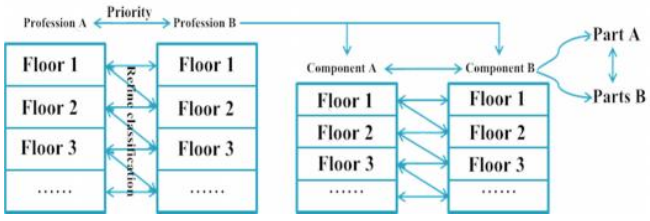

Fig.6 Classification schema of clash detective

4. The implementation steps of collision inspection

Based on the classification principle of collision checking, the collision inspection of assembled structure can be divided into following steps:

(1) The transformation and assembly of BIM model. In order to improve the coordination and compatibility of the model, we always use common system to model, so different majors should use the auxiliary software of each specialty to transform the core model. In Prophase, the BIM core model is created by using the sub-professional modeling model. Different majors extract and upload files in the central file, each specialty can only edit their own professional model, and other professional models can only be viewed, so when the model is completed, only the model will be assembled ,we can get the overall model.

(2) overlap collision inspection. Due to the probability of this collision detection is little, the whole model can checked in advance.

(3) Collision inspection between different professions. There are a lot of research on the type of collision inspection in China and mainly focuses on collision checking between the specialty of plumbing and the structure or the architectural specialty.

(4) The collision inspection within the profession. In the assembling structure, the collision inspection in the structural specialty is the heavy important, the collision inspection is mainly concentrated between the precast elements, including the precast concrete and the embedded parts.

(5) Dynamic collision inspection. Dynamic collision improves a dimension than static collision, and is the most important collision inspection form. Not only to test the three-dimensional model in collision inspection, but also to detect the moving entities in the process of time. In the assembling structure, the dynamic collision inspection is mainly for the hoisting of precast units and the reasonable arrangement of construction machinery.

(6) Optimization grouping of collision points. Collision point is the result of collision inspection, although it has been classified according to certain principles before collision analysis, the output result still has invalid collision points, so it needs to extract effective collision points according to a certain principle and make the grouping mark. The effective collision points is the collision points that really affects the efficiency and quality of the actual construction process and the hoisting process during the construction of the assembling structure, for some collision points which are checked out, but the effect on the actual project is less ,then they can be ignored.

(7) Propose modifications and feedback to the BIM core model. Through the above-mentioned process, the engineers propose views for different types of collision points and feedback them to relevant majors, and then modifies the BIM core model.

The above steps are a cyclic dynamic process, and we think that the model is the final model when we do not discover the effective collision points after several adjustments to the model. This model can be used to deep process for later models.

5. Conclusion

The collision analysis of assembling structure based on BIM technology improves the construction efficiency and quality and saves the cost. Through analyzing the whole process of the collision inspection of assembled structure, this research improves the analysis efficiency and the analytic precision of the BIM collision analysis software and provides the way to optimize the assembling structure and guarantees the fine refinement management and the benefit maximization of the whole life cycle of the assembled building.

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The relationship between the fracture toughness and loading rate of rock-like materials

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Abstract：Based on the mechanical properties of rock-like quasi-brittle materials and analytical solution of stress field at the crack tip, the shape and size of the fracture process zone (FPZ) at the crack tip are calculated by using the Mohr Strength Criterion. The relationship between the FPZ and the crack propagation velocity of crack is also determined. And then the relationship between the initiation toughness of the rock and loading rate is deduced by the size effect formula. The results show that the area of the FPZ at the crack tip increases with the increase of the crack propagation velocity, and the fracture toughness of the rock increases with the increase of the loading rate. The dependence of the fracture toughness on the loading rate calculated in this paper is in agreement with the experimental data, so the correctness of the theoretical model is confirmed. There is a discrepancy between the calculation and experimental results, the authors think that the reason for the discrepancy between the calculated and experimental results is that the model has not considered the viscosity effect of deformation and fracture. The viscous effects will be considered in future studies.

Key words：Fracture process zone (FPZ), Size effect, Crack propagation velocity, Loading rate.

Introduction

There are two kinds of size effect[1] of the rock-like quasi-brittle materials, like rocks, concrete. One kind of the size effect is a statistical size effect[2,3,4], which can be described by Weibull's random local strength theory; the other size effect is the energy size effect. The energy size effect of the quasi-brittle material can be divided into two types: type I size effect[5-10], which describes the failure of the specimen due to the occurrence and development of the crack without the initial crack, and the type II size effect, which describes the presence of deep incision or stress-free deep cracks before the damage is present in the specimen[5,11-14].

For type II size effects, microcracks in quasi-brittle materials such as rocks are induced by the strain localization and developed near the tip of the crack to produce a so-called fracture process zone (FPZ). The characterization of this area is extremely important for the development of non-linear fracture mechanics of quasi-brittle materials such as rocks. The effect of three kinds of brittle failure strength criterion and material parameters on the size of FPZ is discussed by Hu and Huang[15] using Westergaade solution of crack tip stress field, and the expression of contour and characteristic size of microcrack zone at crack tip are deduced.

Hu and Duan[13.14] think that the factors controlling the quasi-brittle fracture behavior of concrete are the interaction between the cracked process zone and the nearest specimen boundary. They call this effect a boundary effect, and accordingly propose a size effect law containing the size of the

fracture process zone (FPZ). According to this model, the strength of concrete depends on the relative size of the failure process zone with respect to the specimen size. The relative size of the damage process zone is greater, the strength of the specimen is greater.

For study of the stress field at the front of the moving crack, Yoffe[16] proposed a special motion crack model that a Griffith crack moves at a constant speed. Freund[17] found the displacement field and stress field of the crack at constant speed and variable speed.

Chen, Xia and Gao[18-20] performed experimental research on the dynamic fracture properties of granite with separate Hopkinson pressure bar to determine the initiation toughness, propagation toughness and propagation speed of a crack, and found that the initiation toughness and propagation toughness of granite increase with increasing loading rate.

At present, the relationship between the loading rate and the fracture toughness of the rock is determined on the experimental data and its physical mechanism has not yet been reasonably explained. Recently, for the type I size effect of specimen without initial crack, Qi et al[21] demonstrated the close relationship between the type I size effect and the strain rate effect. So what is the intrinsic relationship between the type II size effect and the strain rate effect of specimen with the initial crack? Based on the mechanical properties of the quasi-brittle materials and the analytical solutions of the dynamic stress field of the crack tip, the size of the fracture process zone of the dynamic crack tip is determined by using the Mohr Strength Criterion. Combined with the size effect equation, the relationship between the type II size effect and the loading rate effect of quasi-brittle material is explored.

2. The relationship between the size of the dynamic fracture process zone and the velocity of crack propagation

In 1951 Yoffe[16] published the first mathematical solution of the moving crack based on a special crack model that a Griffith crack moves at a constant speed *v* in an infinite solid. After performing Fourier transform and solving dual integral equations, the following solution was obtained $K_I(v) = \lim_{x=0} \sigma\sqrt{2\pi a}$. In the crack model proposed by Yoffee, the dynamic stress intensity factor is independent of time and the velocity of the crack.

Freund[17] solved the problems of crack propagation with constant speed and variable speed. According to the model, the stress field near crack tip can be expresses as follows:

$$\begin{cases} \sigma_{xx} = \frac{K_I}{\sqrt{2\pi}} \frac{(1+\alpha_2^2)}{4\alpha_1\alpha_2 - (1+\alpha_2^2)^2} \left\{ (1+2\alpha_1^2-\alpha_2^2) \frac{\cos\frac{\theta_1}{2}}{\sqrt{r_1}} - \frac{4\alpha_1\alpha_2}{1+\alpha_2^2} \frac{\cos\frac{\theta_2}{2}}{\sqrt{r_2}} \right\} \\ \sigma_{yy} = \frac{K_I}{\sqrt{2\pi}} \frac{(1+\alpha_2^2)}{4\alpha_1\alpha_2 - (1+\alpha_2^2)^2} \left\{ -(1+\alpha_2^2) \frac{\cos\frac{\theta_1}{2}}{\sqrt{r_1}} + \frac{4\alpha_1\alpha_2}{1+\alpha_2^2} \frac{\cos\frac{\theta_2}{2}}{\sqrt{r_2}} \right\} \\ \sigma_{xy} = \frac{K_I}{\sqrt{2\pi}} \frac{(1+\alpha_2^2)}{4\alpha_1\alpha_2 - (1+\alpha_2^2)^2} \left\{ 2\alpha_1 \left(\frac{\sin\frac{\theta_1}{2}}{\sqrt{r_1}} - \frac{\sin\frac{\theta_2}{2}}{\sqrt{r_2}} \right) \right\} \end{cases}$$

where,

$$\bar{y} = \frac{r_1}{\alpha_1}\sin\theta_1 = \frac{r_2}{\alpha_2}\sin\theta_2, \bar{x} = x_1 - \dot{a}t = r_1\cos\theta_1 = r_2\cos\theta_2$$

$$r_1 = \sqrt{\bar{x}^2 + \alpha_1^2\bar{y}^2}, \theta_1 = arctan(\alpha_1\frac{\bar{x}}{\bar{y}})$$

$$r_2 = \sqrt{\bar{x}^2 + \alpha_2^2\bar{y}^2}, \theta_2 = arctan\left(\alpha_2\frac{\bar{x}}{\bar{y}}\right)$$

$$\alpha_1 = \sqrt{1-\frac{v^2}{c_1^2}}, \alpha_2 = \sqrt{1-\frac{v^2}{c_2^2}} \qquad (1)$$

where v is the speed of crack propagation ; c_1 is longitudinal wave velocity ; c_2 is the shear wave velocity ; K_I is the stress intensity factor. The coordinates of the crack tip are shown in Fig 1.

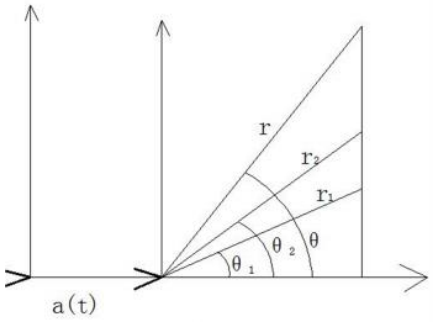

Fig 1.The coordinates of the crack tip.

By using the following transformation:

$$r_2 = \sqrt{r_1^2\cos^2\theta_1 + \frac{r_1^2\alpha_2^2\sin^2\theta_1}{\alpha_1^2}}$$

$$\theta_2 = tan\frac{\alpha_2\tan\theta_1}{\alpha_1} \qquad (2)$$

we can get the principal stresses near the type I crack tip:

$$\sigma_{1,3} = \frac{\sigma_{xx}+\sigma_{yy}}{2} \pm \sqrt{(\frac{\sigma_{xx}-\sigma_{yy}}{2})^2 + \sigma_{xy}^2} \qquad (3)$$

For brittle failure of rock materials, the Mohr Strength Criterion has the following form[15]:

$$\sigma_1 - \beta\sigma_3 = \sigma_u,\ \frac{[\sigma_t]}{[\sigma_c]} = \beta \qquad (4)$$

where σ_u is the ultimate tensile strength of rock ; σ_t is the uniaxial tensile strength of rock ; σ_c is the uniaxial compressive strength of rock.

The polar coordinate function $r(v, \theta)$ can be deduced from the equations (1), (2), (3) and (4), and $r(v, \theta)$ are the polar coordinate functions of the crack propagation velocity v.

The experimental data[18] are used to verify the correctness of the calculation. The specimen is a semicircle with a diameter of 0.04 m and the thickness of 0.016 m. The sample material is Stanstead granite, the longitudinal wave velocity is c_1=5000m/s, the shear wave velocity is c_2=3000m/s , σ_t =5Mpa, the crack length is a=0.008m. For different crack propagation speeds, the corresponding fracture process zone shape is shown in Figure 2, the corresponding interpretation of the results is presented in Table 1.

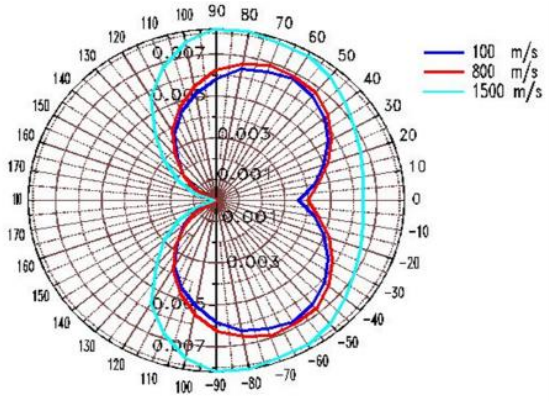

Fig.2 The size of the fracture process zone at different crack speeds.

Table1 The area of the fracture process zone at different crack speeds.

Crack velocity (m/s)	FPZ area(m²)
100	6.80E-05
400	6.99E-05
700	7.43E-05
1000	8.20E-05
1200	9.37E-05
1500	1.20E-04

It can be seen from Tab.1 that the fracture process zone area of the crack tip increases with the increase of the crack velocity.

The relationship between the area of the fracture zone and the crack velocity can be expressed as follows:

$$A_{fpz} = \frac{1}{16856 - 5.47v} \qquad (5)$$

The corresponding plot is shown in Figure 3.

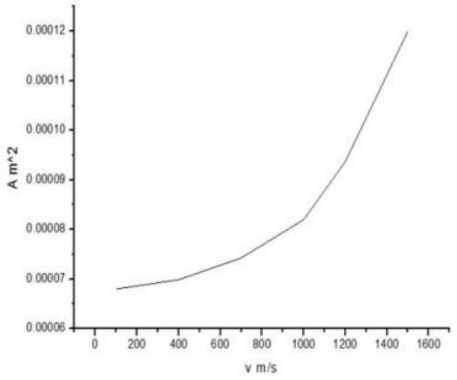

Fig.3 The area of the fracture process zone at different crack speeds.

3. The relationship between the strain rate effect on the initiation toughness and the size effect

The ability of quasi-brittle materials to resist crack propagation is the fracture toughness, and the fracture toughness corresponding to crack initiation is called initiation toughness. In order to explore the relationship between the loading rate effect on fracture toughness and the size effect and reveal the physical mechanism of the loading rate effect on fracture toughness, we start with the size effect model of Hu and Duan[9, 10].

Hu and Duan's size effect law has the following form[9,10]:

$$\sigma_n = \frac{f_t}{\sqrt{1 + \frac{a}{a_{FPZ}}}} \qquad (6)$$

where σ_n is the nominal stress; a_{FPZ} is the characteristic size of the fracture process zone; f_t is the tensile strength; a is the crack size.

The relationship between the size of the fracture process zone a_{FPZ} and the area of the fracture process zone A_{FPZ} can be expressed as :

$$a_{FPZ} = \sqrt{A_{FPZ}}$$

From which we can get the nominal stress of the rock under the size effect:

$$\sigma_n = \frac{f_t}{\sqrt{1+\frac{a}{\sqrt{A_{FPZ}(v)}}}} \quad (7)$$

There is a stable statistical relationship between the opening of the crack and the crack size[22]:

$$\Delta=\frac{\delta}{a}=\Theta\times10^{-2} \quad (8)$$

where Θ is a coefficient in the range of 1/2－2.

The relationship between crack length and crack velocity:

$$\dot{a}=v,\ \dot{\varepsilon}=\frac{\dot{\delta}}{d}=\frac{\Delta\dot{a}}{d}=\frac{\Theta\times10^{-2}\times v}{d} \quad (9)$$

where *d* is the specimen size.

The relationship between nominal stress and strain rate of rock：

$$\sigma_n(\dot{\varepsilon}) = \frac{f_t}{\sqrt{1+\frac{a}{\sqrt{A_{FPZ}(\dot{\varepsilon})}}}} \quad (10)$$

The relationship between initiation toughness and strain rate is,

$$K_{IC}(\dot{\varepsilon}) = \lim_{x=0}\sigma_n(\dot{\varepsilon})\sqrt{2\pi a} \quad (11)$$

where a is the crack *initial* length.

Under the experimental conditions, the fracture toughness can be calculated from (5), (9), (10), (11), and is shown in table 2.

Crack velocity m/s	Loading rate Gpa$\sqrt{m}s^{-1}$	Initiation toughness Mpa$\sqrt{m}$
100	50	3.1
200	100	3.3
300	150	3.4
400	200	3.6
500	250	3.8

Table 2 The initiation toughness at different strain rates.

The theoretical calculation data are compared with the experimental data as shown in Fig 4:

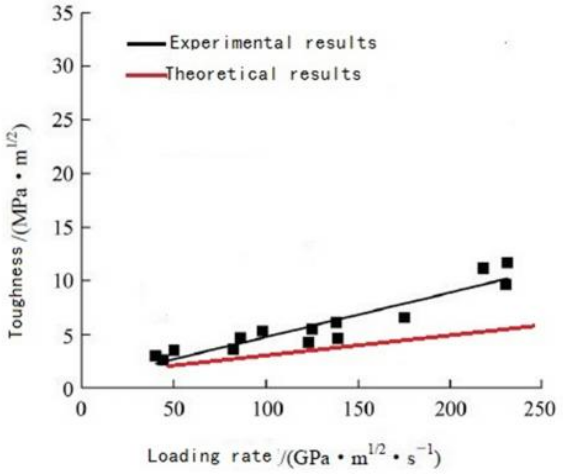

Fig 4 Theoretical calculation data compared with experimental data.

Theoretical results show that initiation toughness of rock-like quasi-brittle materials increases with the increase of loading rate, which is in agreement with the experimental data. With the increase of loading rate, the theoretical results and the experimental data have discrepancy. The authors think that the main reason is the neglecting of the cohesive force and viscosity in the fracture process zone of the crack, which causes the energy dissipation. The present analytical model does not consider the viscosity and cohesion. In the future the authors will study the relationship between the strain rate effect and the size effect of the fracture toughness of the rock with the consideration of the viscosity of the rock.

4.Conclusion

Based on the mechanical properties of the quasi-brittle materials and the analytical solutions of the stress field near the dynamic crack tip, the size of the fracture process zone of the dynamic crack tip is determined by using the Mohr Strength Criterion. Combined with the size effect equation, the relationship between the type II size effect and the loading rate effect on fracture toughness of quasi-brittle material is explored. The following conclusions can be drawn:

1. The area of the fracture process zone at the dynamic crack tip of the rock-like quasi-brittle

material increases with the increase of the crack propagation velocity.

2. The initiation fracture toughness of rock-like quasi-brittle materials increases with increasing loading rate.

3. In the calculation of the fracture process zone in this paper, as the loading rate increases, there is a discrepancy between the theoretical and experimental values of the fracture toughness of the quasi-brittle materials. The authors think that the reason is the neglecting of the cohesive force and viscosity in the fracture process zone of the crack, which causes the energy dissipation. The present theoretical model does not take into account the cohesion and the viscous effect of deformation and failure. The authors will consider them in studying the relationship between the strain rate effect and the size effect on the fracture toughness of the rock.

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DYNAMIC ANALYSIS OF TUNNEL CROSSING SOIL-ROCK BOUNDARY UNDER DIFFERENT WATER PRESSURE

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Keywords: Submarine shield tunnel, High water pr essure, FLAC3D, Geological fault, Dynamic response.

Abstract
In this paper, by inputting Kobe seismic acceleration at t he bottom, seismic response of undersea tunnel crossing soil-rock boundary under different water pressure is studi ed. Numerical simulation shows that, the fracture zone h as a significant effect on the dynamic response of the stru cture. Especially in the fault zone, the dynamic response of the structure is more intensive, which is greatly affecte d by the seismic wave fluctuation. With the increase of w ater pressure, the displacement and stress of the structure increase, especially the change of stress. The peak of the principal stress of the structure occurs at the junction of r ock and soil along the axis of the tunnel. Due to the diffe rence of the displacement and deformation between soft soil and rock, the shear deformation and large stress occu rred in the structure, especially in the shoulder and feet o f the tunnel structure, which should be protected.

1. Introduction
China is a country with a vast territory, the geological co nditions in Tunnel Engineering are complicated and chan geable. In view of the response characteristics of the tunn el crossing the fault zone in the earthquake, the domestic researchers have done a lot of research works. Hechuan a nalyzed the dynamic response of tunnel in the fault zone by means of seismic test and numerical simulation analys is [1]. Shen Yusheng used numerical simulation to study the transversal force and displacement analysis of tunnel crossings at different inclinations [2]. Liang Jianwen stud ied the force of pipeline structure through three different soil layers, and established the vibration equation of pipe line [3]. Gao Xinjun used the dynamic simulation metho d to study the dynamic performance of the tunnel crossin g soil-rock boundary under the conditions of compressiv e wave oblique incidence [4]. In summary, the current re search on the tunnel through different media is mainly co ncentrated on the land, but the submarine geological situ ation is more complex and changeable, the tunnel will wi thstand greater water pressure. It is of great significance t o study the dynamic response of the tunnel crossing the s oil-rock boundary under different water pressure.

2. Introduction to the calculation model
In order to study the influence of the change of water pre ssure on the dynamic response of shield tunnel crossing s oil-rock boundary, in this paper a three-dimensional mod el is established by FLAC3D software. The bedrock is m icro-weathered tuffaceous sandstone and its mechanical behaviour is simulated by Mohr-Coulomb constitutive m odel. The soil layer is silty clay, and the Duncan-Chang' s constitutive model is used to simulate the behaviour of soil [5]. The lining of the tunnel is mainly composed of c oncrete, and the amount of steel bars is relatively small . After considering the effect of the pipe joint on the over all stiffness, the lining structure is homogenized as homo geneous ring, and the elastic model is used to simulate it (Material parameters are shown in Table 1-1 and Table 1 -2). In order to save the computation time, this model use s the artificial boundary condition (free boundary) to red uce the boundary seismic wave reflection to simulate the semi-infinite interval calculation. At the same time, Rayl eigh damping is used to simulate the damping of seismic wave propagation. Due to the lack of seismic records at t he site of the subsea tunnel, in this paper, a 15s portion o f Kobe wave with adjusted peak acceleration 0.6g is sele cted. In the calculation by using the FLAC3D software, t he mesh size is determined by the wavelength correspon ding to the maximum frequency of the input waveform. I n order to control the size and quantity of the meshes and

save computing time, it is necessary to filter out the high frequency components of the input wave. In addition, for each full waveform, there will be a residual velocity and displacement at the bottom at the end of the calculation, so we need to correct the baseline of seismic waves [6]. In this paper, we use SeismoSignal software to intercept, filter and adjust the Kobe seismic wave. And the adjusted seismic wave is shown in Fig. 1-1. At the bottom of the model, a part of the foundation is soft soil foundation. Therefore, it is necessary to convert the acceleration wave into a velocity wave (Fig. 1-2). Equation 1-1 and Equation 1-2 are transformation relations, and the final seismic wave is applied in the form of stress at bottom of the model.

$$\sigma_n = -2(\rho C_p)v_n \quad (1\text{-}1)$$

$$\sigma_s = -2(\rho C_s)v_s \quad (1\text{-}2)$$

Where: v_n is the forward velocity of the boundary, v_s is the tangential velocity of the boundary, ρ is the medium density, C_p is the propagation velocity of the P wave in the medium, and C_s is the propagation velocity of the S wave in the medium.

Table 1-1

Physical parameters of soil Table 1-2 Physical parameters of bedrock and lining

	c（Pa）	φ(。)	Rf	K	n	Kb	m	Kur
silty clay	3.70E+04	28	0.633	127.93	0.55	260.44	1.03	319.825

	c（Pa）	φ(。)	σ_b /Pa	ρ （kg/m3）	ν	E/Mpa
Micro - weathered tuffaceous sandstone	1.00E+06	35	1.00E+05	2645	0.2	8.20E+09
Shield segment	1.00E+06	50	2.65E+06	2550	0.18	3.45E+10

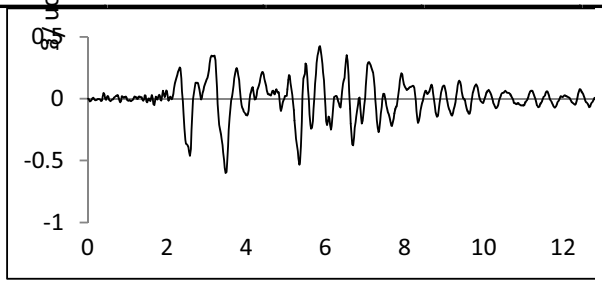
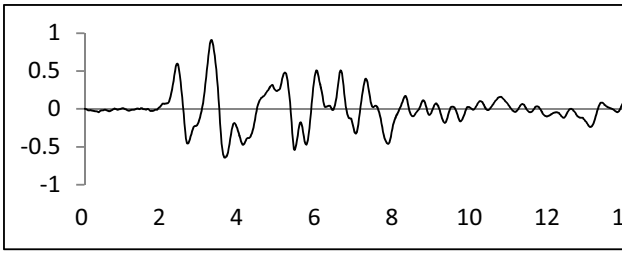


Figure 1-1 Adjusted seismic wave acceleration time history



The main monitoring points are set as shown in Fig. 1-3. In order to compare the dynamic response of the tunnel i n the soft soil and in the surrounding rock, the monitorin g points are set every five meters along the axial directio n of the tunnel, and within 30 meters on both sides of th e junction monitoring points are set every 2.5 meters. The damage of the tunnel under the action of earthquake is mainly shear failure, this paper will focuse on monitori ng the first principal stress and the structural displaceme nt in different parts of the structure.

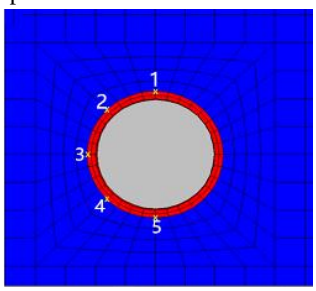


Figure 1-3 Monitoring points layout

The longitudinal direction of the model is 200 meters lon g along the axial direction of the tunnel, where the bound ary of the rock and soil is located at the middle of the mo del and perpendicular to the axial direction of the tunnel . The transverse direction of the model is 80 meters on b oth sides of the tunnel center and 40 meters above the up per part of the tunnel. The bottom of the model is 60 met ers high and the tunnel is 11.4 meters in diameter. The li ning is 0.7meters thick and 1.5meters wide. The model i s shown in Figure 1-4. The depth of the tunnel is 30 metr es below seabed and the influence of the hydrodynamic p ressure on the seismic response of the tunnel structure is relatively small[7], and can be neglected in the calculatio n. The simulation of water pressure in the way of hydrost atic pressure applied at the top of the model, water pressu re selection 200 meters (2.0MPa), 100 (1.0MPa) an d 0 m (0.0MPa) three conditions were compared.

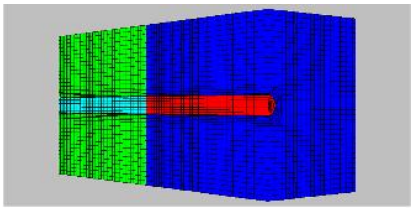


Figure 1-4 Computational model mesh

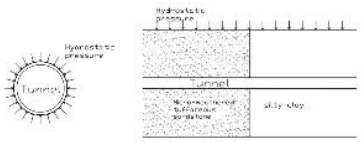


Fig. 1-5 Schematic diagram of calculation model

3 Calculation result analysis

3.1 Displacement

Figure 1-6 shows the horizontal displacement time-history of the vault at the junction of soil and stone (+10 m) under the water pressure of 2.0 MPa. Figure 1-7 to 1-9 are the peak displacements along the axis of the tunnel at the vault, arch waist and arch bottom.

Figures 1-7 to 1-9 show the horizontal displacement of e ach monitoring point, from which the following conclusi ons can be drawn:

(1) The displacement of the tunnel is greatly affected by t he seismic wave, the maximum displacement peak at eac h monitoring point occurs in the most intensive period of seismic vibration, and the displacement fluctuation beco mes smooth with the slowdown of the seismic wave. Co mpared with the rock area, the displacement of the tunne l in the soft soil is more intensive, which indicates that th e soil is more sensitive to the seismic wave.

(2) The maximum displacement of the structure occurs i n the soft soil, and the displacement peak is about 4 time s larger than that in the rock. Under the action of shear s eismic waves, the displacement of the tunnel in the soft s oil is much larger than that in the rock. As can be seen fr om Fig. 1-7, the displacement peak curve varies along th e axial direction of the tunnel, especially when the water pressure is 2.0MPa, at about 30 meters away from the ju nction of rock and soil the peak displacement began to in crease, and then stabilized, which is consistent with the t heoretical derivation of the literature [8].

(3) The effect of water pressure on the displacement pea k of tunnel is significant. On one hand, the peak displace ment of the tunnel in the rock is different, and the peak v alue of the structure is increased at the junction of the roc k and soil. On the other hand, the water pressure increase s the influence range of the medium mutation. As can b e seen from Fig. 1-7, the influence range of the mediu m mutation increases with the increase of the water press ure.

3.2 Stress Analysis

Figures 1-10 to Figures 1-11 show the variation of t he first principal stress peak along the tunnel axis at the s pandrel and foot of the tunnel under different water press ures.

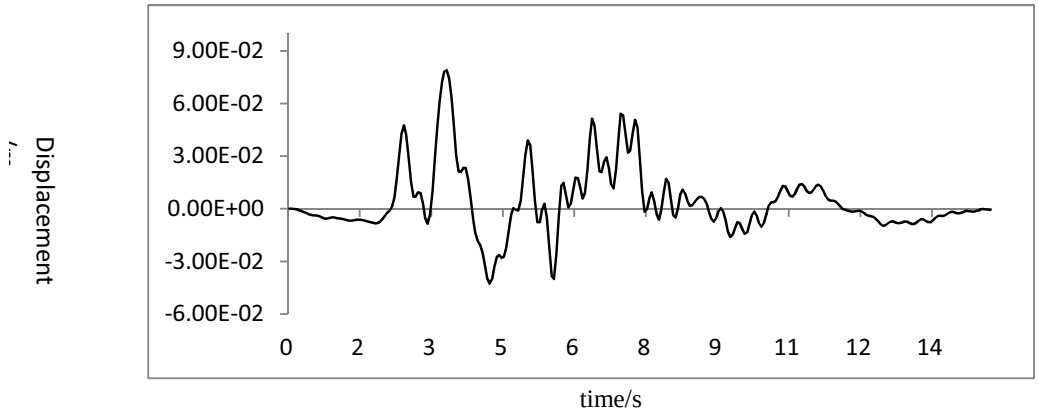


Fig. 1-6 Time history of horizontal displacement of tunnel structure under water pressure of 2.0MPa

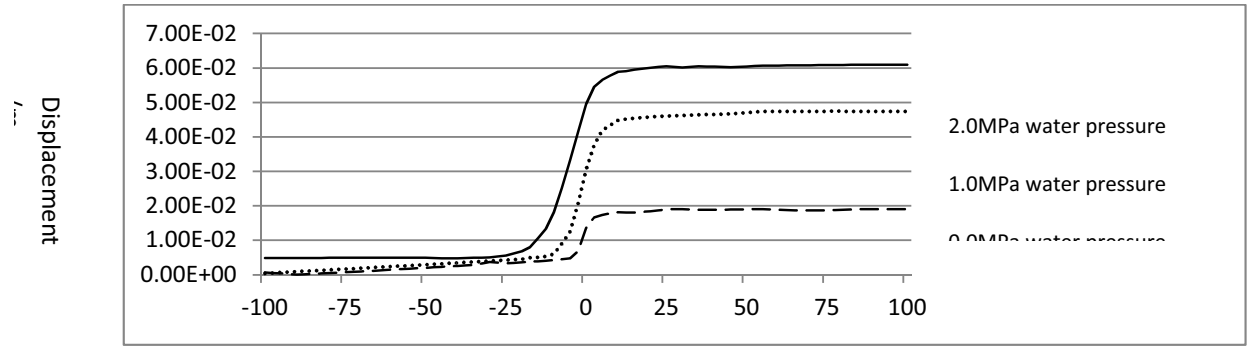


Fig. 1-7 Horizontal displacement of vault under different water pressure

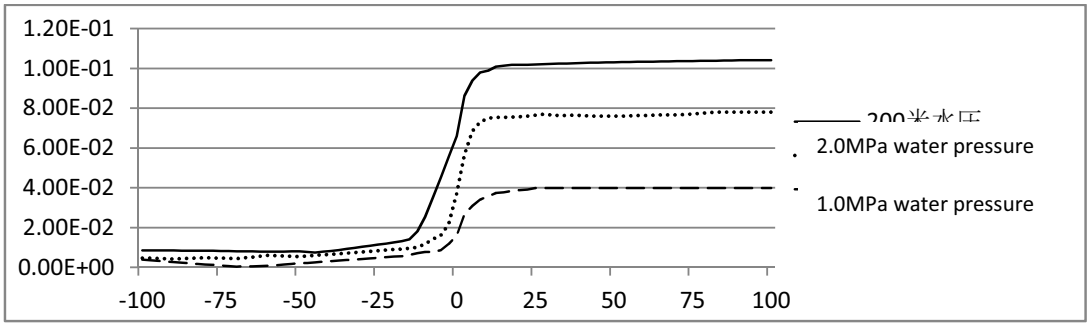


Fig. 1-8 Horizontal displacement of archwaist under different water pressure

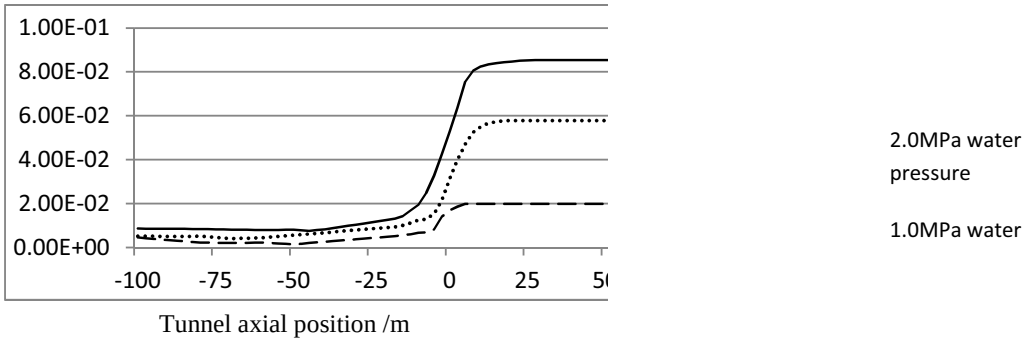


Fig.1-9 Horizontal displacement of arch bottom under different water pressure

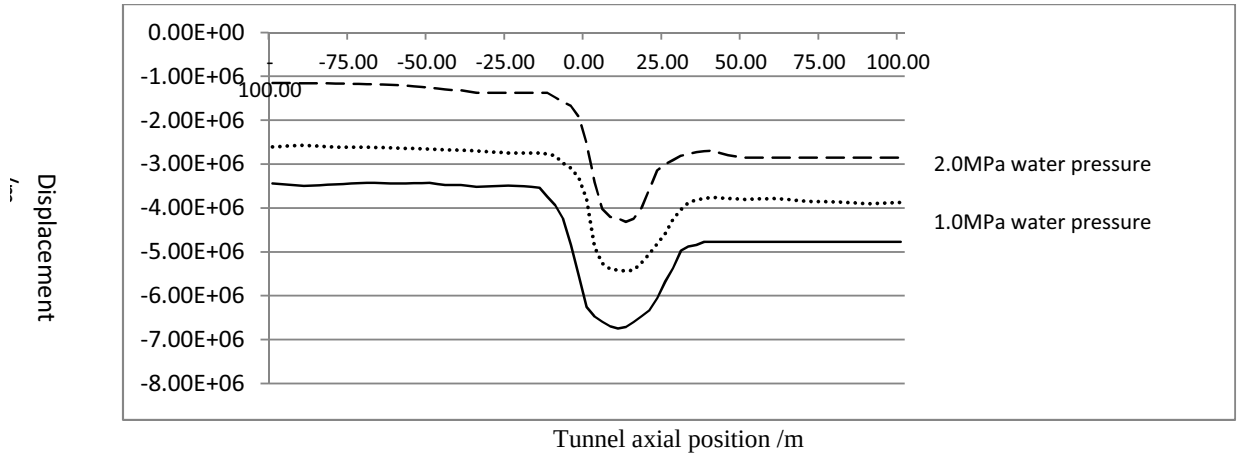


Fig.1-10 The variation of the first principal stress of arch shoulder with water pressure

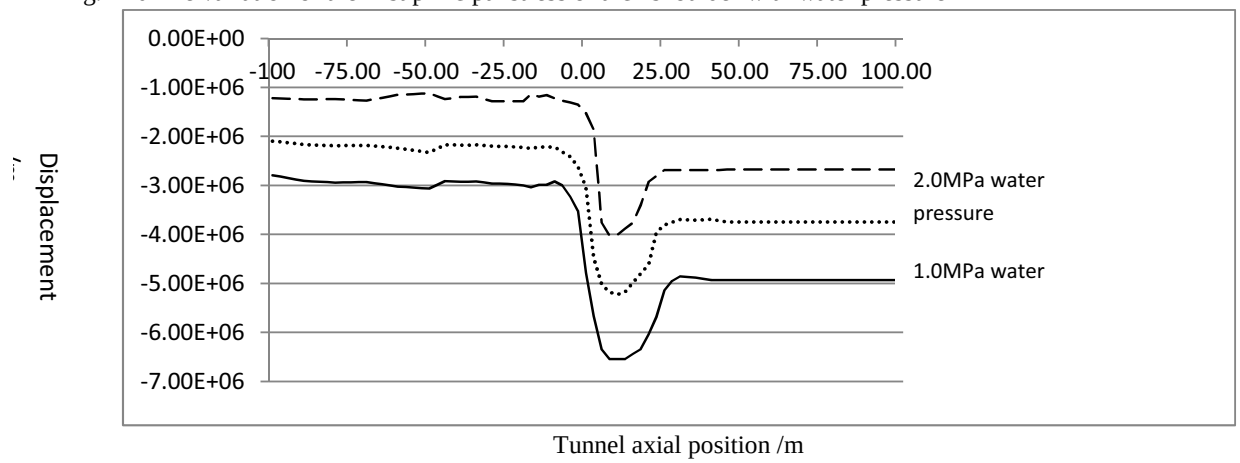


Fig.1-11 The variation of the first principal stress of arch foot with water pressure

We can draw the following conclusions from Figure 1-10 to Figure 1-11

(1) The effect of seismic action on the structural force is much more intensive near the soil and rock boundary, and in rock the influence of the earthquake is much smaller.

(2) Under the seismic action, the stresses of the shoulder and the lower corner of the tunnel is much larger than that in other parts of the tunnel, which is in accordance with the results of [9]. On the other hand, the maximum stresses occurs at the junction of the soil and rock. This is mainly due to the difference of displacement and deformation of the rock and soft soil under the action of seismic wave, so that the structure produces a greater stresses.

(3) When the water pressure changes, the first principal stress at the monitoring points of the tunnel structure is less affected, and the stress is affected by the change of the water pressure at the boundary of the soil and rock. From Figure 1-10 to Figure 1-11 it can be issued firstly, the impact of water pressure is mainly reflected in the stress peak and the scope of the impact of media changes. At the junction of the soil and rock, high water pressure can be found to increase the peak value of the first principal stress in the structure. On the other hand, the water pressure increases the influence range of the medium mutation. As can be seen from Fig. 1-10, the water pressure increases from 0.0MPa to 2.0MPa when the first principal stress began to increase about 15 times. Therefore, the presence of high water pressure can increase the stress peak of the tunnel structure in the soil and rock, and increase the influence range of the medium mutation

.

4. Conclusion

In this paper, by inputting Kobe seismic accelerogram at t he bottom of the calculation model seismic response of the undersea tunnel crossing soil-rock boundary under differe nt water pressure is studied. The numerical simulation sho ws that,

(1) The effect of water pressure on the peak response of tu nnel in soft soil area is remarkable. On one hand, the pea k displacement of the tunnel in rock and soil is different, a nd the peak displacement of the structure increases with th e transition from rock to soil. On the other hand, the wate r pressure increases the influence range of the medium mu tation. As can be seen from Fig. 1-7, the influence range o f the medium mutation increases with the increase of th e water pressure.

(2) The effect of seismic action on the structural internal f orces is more intensive in soil and the soil and rock bound ary. In rock the influence of seismic action is much smalle r. Under seismic action, the stresses at the shoulder and th e lower corner of the tunnel is much larger than that in oth er parts of the tunnel. On the other hand, the maximum str ess occurs at the junction of the soil and rock. This is main ly due to the difference of displacement and deformation o f the rock and soft soil under the action of seismic wave.

(3) When water pressure changes, the first principal stres s at the monitoring points of the tunnel structure is less aff ected, but is seriously affected by the change of the wate r pressure at the boundary between soil and rock. From Fi gure 1-10 to Figure 1-11 it can be seen that, firstly, the im pact of water pressure is mainly reflected in the stress pea k and the scope of the impact of media changes. At the jun ction of the soil and rock, high water pressure can be foun d to increase the peak value of the first principal stress of t he structure. On the other hand, the increase of water press ure increases the influence range of the medium mutation . As can be seen from Fig. 1-10, the water pressure increas es from 0.0MPa to 2.0MPa when the first principal stres s began to increase about 15 times.Therefore, the presenc e of high water pressure can increase thedisplacement, stre ss peak of the tunnel structure in soil and rock, and increas e the influence range of the medium mutation.

Acknowledgement

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CRETE TOWER CONSTRUCTION OF LARGE CABLE STAYED BRIDGE

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Keywords: large cantilever construction, hydraulic climbing formwork, accurate measurement

Abstract

The large cable-stayed bridge main tower for the diamond type tower above the deck is an inverted Y type tower for vertical single tower; the middle tower is split into two out of the tower in the tower beam at the towers at together, under the tower for rigid whole wall section, under the beam and the lower pylon binding site adopt arc transition, under the tower tower at 7.15 meters and the cantilever bending point on the lower beam 8.8 meters of vertical cantilever two big cantilever force of the key position. Under the tower pylon by formwork construction, on the pylon by hydraulic climbing formwork construction method, with accurate measurement and control technology, the construction of large cantilever concrete pylon.

1 Introduction

The large cable-stayed bridge using PK composite box girder 1250m. High beam 3.5m. main tower with diamond type tower, using C50 concrete, the main tower 217m. tower for vertical single tower, the middle tower is split into two out of the tower in the tower beam at the towers at together, under the the whole wall section. The main pier fortification ship collision force is large, the main tower of up to 4800t, to meet the design force of ship collision, the traditional tower tower under the double limb together increase the stiffness of the tower, resulting in the construction process under the tower at 7.15 meters in the tower bending cantilever; in the lower beam of 8.8 meters, is also a vertical cantilever pylon, as in the inflection point, the main tower structure as shown in figure 1.

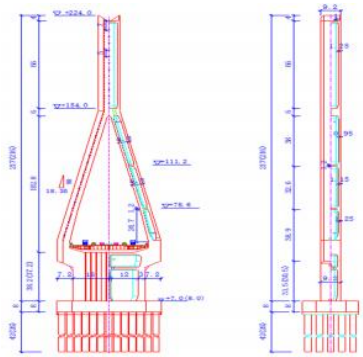


Fig.1 Schematic diagram of main tower

2 Bridge tower construction scheme

The pylon by step pouring construction method. Under the pylon by bracket turnover form construction, beam and tower crown with support formwork construction, the remaining segments are used hydraulic climbing formwork construction technology. The tower is divided into 51 sections (not including the 80 cm initial pouring together with the pile cap construction, under the tower section) the 12 section, in the tower sections of 22 segments, 17 segments on the tower section.

3 Construction Emphasis

1) The appearance quality of concrete tower directly reflects the overall level of construction of the bridge, is also an important factor affecting the durability of the structure. Therefore, the appearance quality of concrete tower, tower construction is the key.

2) The tower beam under the force of special structure, for the construction of large volume concrete, and prestressed pipe is numerous, is the key of construction control.

3) The high tower construction crane, elevator layout and running, climbing safety hydraulic climbing formwork construction system, is a key and difficult point of construction.

4) The tower construction precision control requirements are very high, the construction control to put forward the high requirements.

4 Key construction technology

4.1 Preview Under the tower construction technology

4.1.1 Setting the stiffness matrix

In order to ensure accurate positioning the location of steel tower construction, but also to prevent at high altitude, tilted steel pylon deformation, internal need to set the skeleton. According to the structural characteristics of tower and beam, reinforced the construction of stiff skeleton design for plane space truss column and the horizontal direction of the temporary connection part two truss. When installing, using two line hammer two vertical section of frame column the precise positioning, and then draw the main reinforcement skeleton in the above position, assist in positioning binding main bars.

4.1.2 Temperature control technology of mass concrete pouring of lower beam

In mass concrete construction, often by laying cooling water to cool down, according to the beam structure of this project under the bearing characteristic and prestressed pipe dense, such as the layout of cooling pipe, will further reduce the cross-sectional area, the structure stress and prestressing effect. Through research on the beam structure, prestressed pipe dense can be used as a cooling pipe.

In the process of construction, the main use of water cooling pipe. The prestressed pipe pressure for plastic corrugated pipe, corrugated pipe pressure water to the air compressor, accelerate the bellows internal air circulation, bring out the concrete internal temperature, reaches a temperature effect. The beam of prestressed pipe layout is as follows.

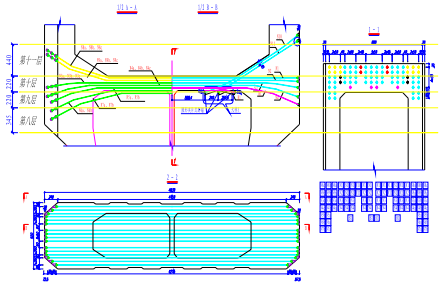


Fig.2 Prestressed beam layout of the lower pylon

The air compressor pipe access and ventilation pipe, adjust the water content, the water enters the air compressor, high pressure air and water in the mix, the water in the high wind pressure, oscillating granular form a fog effect, then the air containing mist is pressed into the corrugated pipe, the pipe heat by water absorption at the same time through the fog, wind speed high pressure air circulation pipeline, temperature control is achieved.

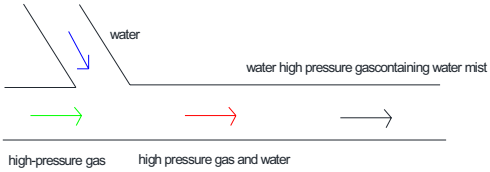


Fig.3 Schematic diagram of working principle

MIDAS FEA is used to analyze the whole model of the solid element, and the influence of the discontinuous and continuous construction of each layer is fully considered.

Fig.4 Simulation model

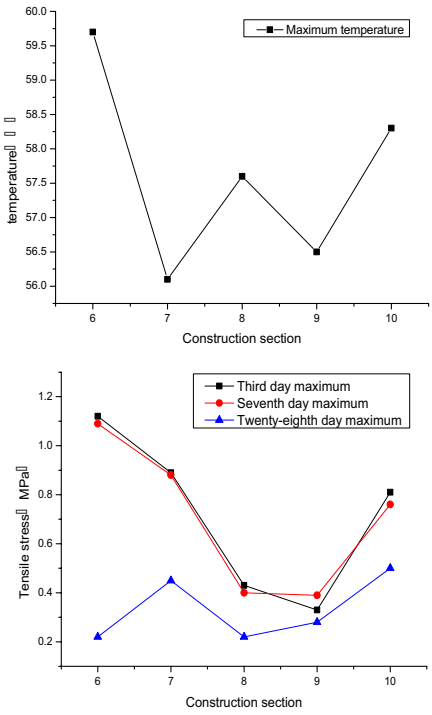


Fig.5 Model results

The model calculates the maximum temperature generally occurs third or fourth days after concrete pouring, under the tower structure mutation at low temperature, the main tower pier under the concrete internal maximum temperature calculation value of 59.7 degrees.

Through the water after the corrugated pipe wall temperature can be reduced, and close to the outside air temperature, temperature control of mass concrete can meet the requirements, but in the water process, to fully consider the size of the pressure of the air compressor, ensure the ventilation effect of high pressure steam. The scene of implementation, field measurement, temperature peak time is about third days after pouring, the highest temperature is 52.3 degrees centigrade, the comparison of model, found the temperature control effect is obvious.

4.1.3 Prestressed construction of lower beam

The lower beam is arranged in the construction of prestressed beam, pre buried corrugated pipe, concrete pouring has been completed through the beam, tension, grouting, anchor seal construction. The tower beam of prestressed steel beam is longer, and the root number, take the current installed in the prestressed pipe to wear steel strands in concrete strength and elastic. To meet the requirements of die after prestressing. The prestressed steel strand tensioning order should be carried out in accordance with the tension after the first intermediate upper and lower left are symmetrical, 90%. tower prestressed prestressed concrete strength is not less than the design strength of the grouted by vacuum process, improve the plumpness, reduce the influence of bubbles.

4.1.4 Prestressed construction of lower beam

The tower and beam construction under the tower formwork construction method. In the template before installation, accurate measurement lofting plane position of pylon by total station polar coordinate method (including axis and edges) and tower bottom elevation, measured skeleton plane position. To be reinforced, template installation is completed, check again check with level template, the top surface elevation, using adjustable rod precise adjustment template to the correct position fixed. After each template, according to the design position accurately calculate the top opening of the 3D coordinates, using the same method of repeated check adjustment, the plane position, top elevation and verticality have reached the design requirements. After the baseline level setting out vertical distance measurement check the measured elevation, do double control checks.

4.2 The construction technology of middle pylon

4.2.1 Structure of hydraulic climbing formwork system

The main tower pier, pylon design of formwork system for hydraulic self climbing form, using the self climbing formwork system of LG-100 hydraulic. In the upper standard pouring height of 4.5 meters, the tower, 168 meters high, a total of 38 times the work cycle, the shrinkage section of tower body by cutting template to realize the combination of steel and wood template, template, climbing the lower frame body share 28 pieces of hydraulic, 30 specimens of upper body. In order to reduce the cost of the template, hydraulic climbing mold body adopts separate frame, hydraulic climbing formwork body is assembled into the advantages of standard construction can rely on the power system's climbing tower crane to hoist do not take, can greatly accelerate the construction progress, and a frame body with a perfected security maintenance system, ensure the safety of the construction.

Hydraulic self climbing formwork is the power source of the hydraulic lifting system itself, the hydraulic lifting system comprises a hydraulic cylinder and the upper and lower reversing box, commutating box can control the lifting guide rail and the lifting frame body, the hydraulic system can make up the formation of mutual support body and the guide template, so that the hydraulic self climbing form steadily upward climb, hydraulic self climbing formwork without other lifting equipment in the construction process, convenient operation, climbing speed, high safety coefficient. The hydraulic self climbing form is mainly divided into the template system, support system, embedded system, the four part of the hydraulic system.

4.2.2 The middle column of the active cross brace construction

In the tower to tilt to the inside, with the increase of construction period, need to set the active level of support, the unbalanced force generated by the balance tilted tower. The hydraulic climbing formwork in the construction of the tower, the first horizontal braces applied thrust force 334.6t, second horizontal braces applied thrust force 297.2t. third horizontal braces applying thrust force 269.4t. fourth horizontal braces applied thrust force 305.3t.

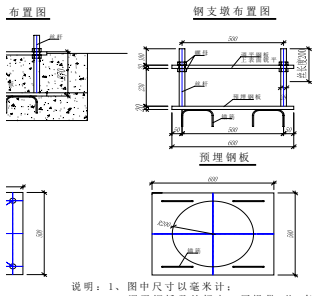


Fig.6 In the schematic diagram of horizontal braces

According to the tower structure and construction characteristics of the active cross brace of abrupt structure stress monitoring, stress measurement points of the section are: under the tower section (C-C), the upper side section and the main tower closure cap the maximum cantilever position (A-A, B-B). Off the mask position according to the monitoring calculation the results and practical engineering to determine the comprehensive consideration.

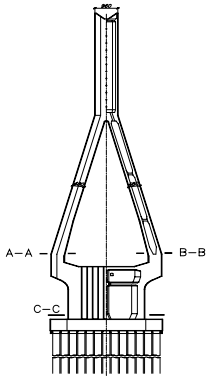
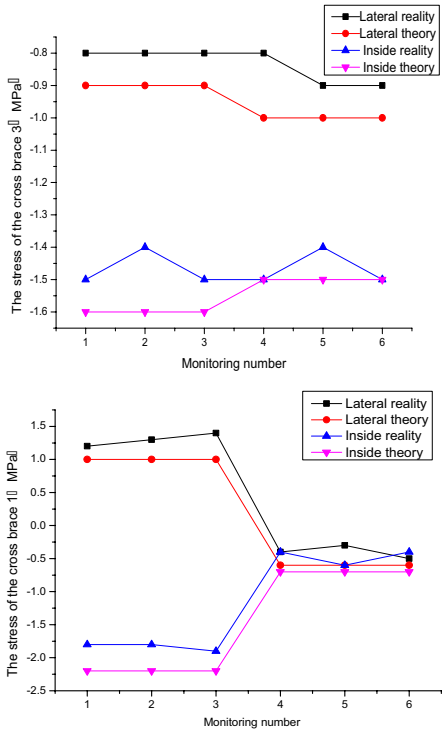


Fig.7 The sketch map of stress testing section

Using MIDAS finite element program of calculation and analysis, to establish the main tower model, the main tower is divided into 280 units and 310 nodes, in the process of construction, in the layout of the 4 tower root temporary active cross brace, the calculation model is shown in figure:

Before and after the four horizontal support, the stress of the measuring points was monitored, and the measured values were compared with the results of finite element analysis. The results of A-A section were compared, as shown in the figure:



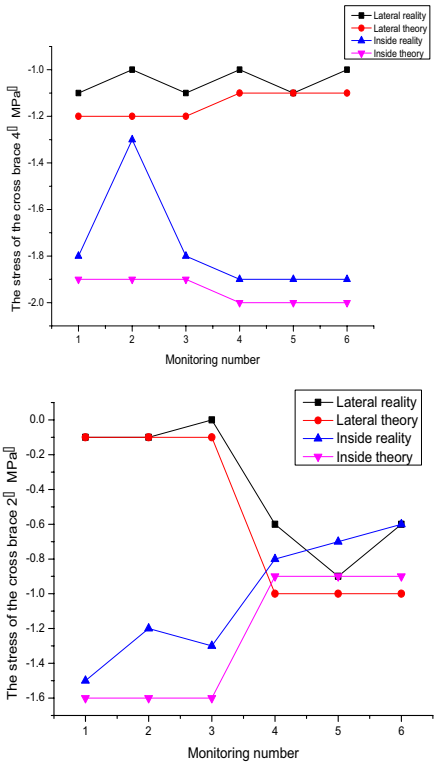


Fig.8 Comparison of internal and external stress measurement of section A-A

From the section above key construction stage on the tower stress test, cross brace 1,2,3 pushing measured before and after A-A and the measured stress is basically consistent with the theoretical value, the 4 cross brace pushing before and after A-A beta measured one point mutation value, but the value is smaller, less than the theoretical value. On the whole the tower, the stress theory and test are basically consistent, and consistent with the theoretical calculation, the structure is in the controllable state. All measuring points of the measured stresses are in good agreement.

5 Conclusion

The large cable stayed bridge pylon by formwork construction, on the pylon by hydraulic climbing formwork construction method.

1) Under the tower beam under the stress of special structure, for the construction of large volume concrete, and prestressed pipe is numerous, is the key of construction control. During the construction of cantilever beam under section, the tower section by molding method construction technology of construction, to ensure the safety of construction process is the key and difficult control from construction. Ordinary type brick tower, the tower is under the wall section, the section size is larger, belongs to the large volume of concrete pouring, pouring should be stratified and relatively reasonable, avoid disposable had hydration heat caused by internal and external temperature difference. The stress should be strictly monitoring the temperature of casting process,

prepared in advance and corresponding control measures and to avoid cracks.

2) In the tower to tilt to the inside of the pylon by hydraulic climbing formwork construction method in the construction of the tower in the gravity and the climbing frame under the action of additional moment will be produced and the unbalanced force, the inward tilt alignment will produce deviation, must set up the active cross brace. About the position and thrust force reasonable active cross brace to the tower tower after the final stress state. The simulation analysis can determine the number and location of the active cross brace, and can be determined for each active cross brace thrust force.

Acknowledgements

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The steel-concrete composite beam stress analysis of single pylon cable stayed bridge pylon pier consolidation area

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Key words: Composite beam；Cable-stayed Bridge；Pylon pier consolidation；Stress state；Improvement measures.

Abstract

In order to ensure the reliability of the steel-concrete composite beam cable-stayed bridge pylon pier consolidation area design, Using ANSYS to establish the local model of the consolidation area of Pylon Pier, on the premise of verifying the correctness of the model, the stress state of concrete and steel beam is analyzed: The tower pier consolidation area overall stress level meets the requirements, but there is stress concentration at the intersection of the beam end and the steel stringer and the beam. By increasing the size of anchor plate and increase the stress concentration area of common reinforced bar improvement measures, it can effectively alleviate the tower beam consolidation stress concentration region, this conclusion can provide reference for the similar engineering design and construction.

1 Introduction

The rigid frame bridge has the advantages of large stiffness and small deformation, which is especially suitable for single tower cable-stayed bridge [1]. Because it is not necessary to set the support, damper and seismic isolation facilities at the junction of the tower beam, which is convenient for bridge maintenance[2],there are potential risks in the design and operation due to the complex structure and the stress of the girder[3-5]. At present, there are many researches on concrete beams or steel box girders of the main girder, and there are few researches on the problem of the consolidation of the pier of the steel and concrete composite girder cable-stayed bridge, and the stress distribution is not clear.

This paper takes a steel-concrete composite beam for example single pylon cable-stayed bridge, establish a local entity model, analysis of tower pier consolidation area of mechanical characteristics and stress distribution, and makes a comparative study on the improvement measures, provide a reference for the similar engineering design and construction.

2 Establishment of finite element analysis model

2.1 Engineering background

Composite beam single pylon cable stayed bridge concrete steel bridge span is 130+150m, the steel girder with concrete bridge deck section form overall. The main tower is a concrete diamond type tower, tower height 99.4m, C50 concrete. Using steel girders cross into the beam under the tower to form a consolidation system, the bridge layout shown in Figure 1.

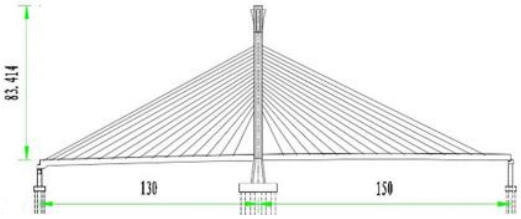


Fig. 1 Eneral layout of cable stayed bridge

2.2 Finite element model

First of all, using MIDAS/CIVIL2013 to establish a full bridge space bar model (Figure 2). Using the CAD-ANSYS combination technology of ANSYS [6], select the bridge beam center line, the lower 14m and the central line of the bridge on the left and right sides of each 9m main beam range to establish a local analysis model (Figure 3), the local mechanical properties of the tower beam are analyzed.

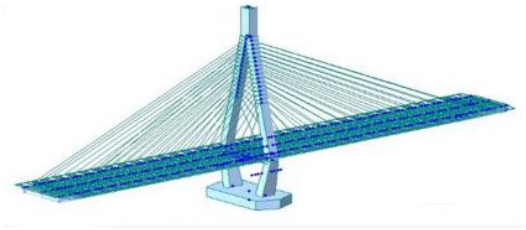


Fig. 2 MIDAS to establish the whole bridge space bar model

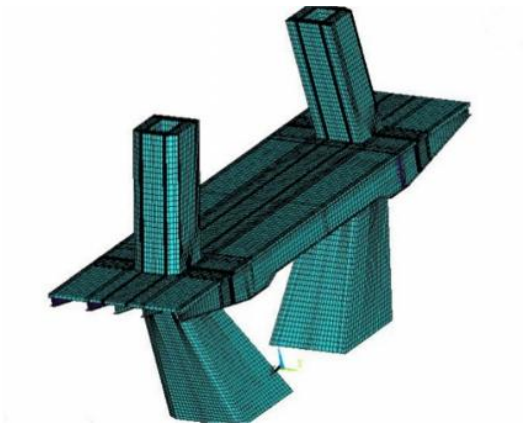


Fig. 3 ANSYS set up local computation model

2.2.1 Constraints and loading

The bottom of the bridge tower use the fixed end constraint, the middle node of the beam is symmetrically restrained, the boundary condition of force applied at the upper pylon truncations. Loading mode: loading in component (centroid) set the master node and the other stress points as nodes and the master node from the use of rigid coupling, axial force, the calculated main beam and tower end of shear and bending moment applied to the master node[7], as shown in Figure 4.

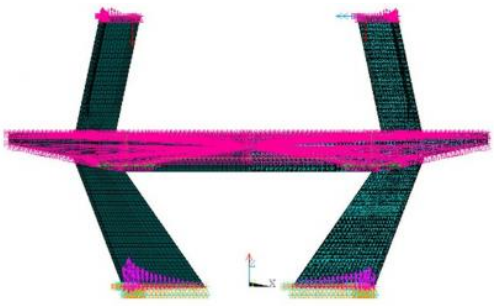


Fig. 4 Boundary condition model

2.2.2 Load condition

Working conditions one: the maximum cantilever construction stage; the conditions two: the use of stage load conditions; the conditions three: the maximum axial force of main girder under the action of constant live load; the conditions four: The maximum bending moment of the tower under the use of constant live load.

2.2.3 Computational model validation

Table 1 is the comparison of the calculation results of the internal force of the whole model and the local model of the main beam under the load condition of two. According to the data in Table 1, the results of the two models can satisfy the accuracy requirements, indicating that the local model boundary conditions are correct and the calculation model is effective.

Table 1 Comparison of beam section internal force

Section Internal Force /m	Components of the Internal Forces	Whole Model	Local Model	Ratio
2.4	Axial force /KN	-101202.0	-104884.7	1.03
	Shear force /KN	-1175.0	-1103.9	0.94
	Bending moment /KN·m	-30048.5	-30344.8	1.01
0	Axial force /KN	-103261.3	-98957.8	0.96
	Shear force /KN	-1554.6	-1624.7	1.04
	Bending moment /KN·m	68574.7	65364.6	0.95
-7.4	Axial force /KN	-98763.5	-95211.3	0.96
	Shear force /KN	581.1	568.7	0.97
	Bending moment /KN·m	-10002.7	-10172.1	1.01

3 Results and analysis

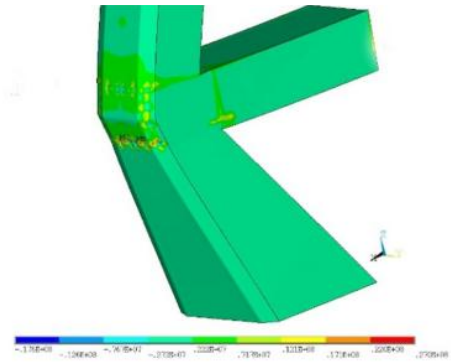
Through the analysis and comparison of the results of the four conditions, the working condition three is the most disadvantageous, and the following is the analysis of the structural stress state under the condition of three.

3.1 Principal stress analysis

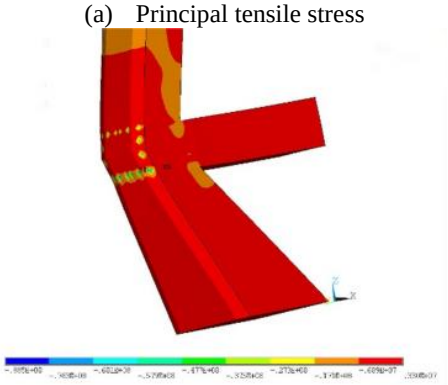
According to the symmetry of the structure, 1/2 extraction results of the model are selected. It can be seen from Figure 5, the principal tensile stress of overall local model mainly in the -2.72~2.22MPa, the maximum

principal tensile stress is 27.0MPa, the end of Anchorage Zone on the beam, but the range is very small, while stress concentration exists in the area of the beam passes through the beam and concrete contact; principal stress is mainly in the 5.58~16.8MPa, the maximum principal the compressive stress is 88.5MPa, which exists in the end part of the beam anchorage zone.

Because of the stress concentration at the intersection of the beam end and the intersection of the steel beam, the stress concentration can be improved by increasing the anchor plate, reinforcing ribs and spiral reinforcement and steel mesh. In addition to the transverse prestressing beam loading point, the stress distribution is uniform and the stress value is not large, which can be considered as the standard stress of the local model.



(a) Principal tensile stress



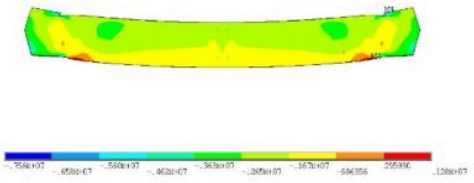
(b) Principal compressive stress

Fig. 5 Principal stress of consolidation area of Pylon Pier

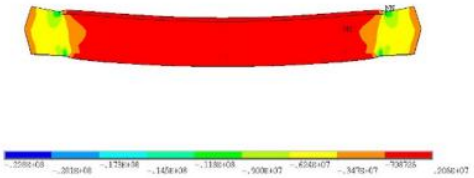
3.2Normal stress analysis

The normal stress distribution in three directions of the beam in the consolidation zone is shown in figure 6. The beam transverse and vertical to the bridge is the basically the compressive stress, and the stress value is small; the maximum value appeared in the area of intersection beam and pylon, the maximum compressive stress is -22.8MPa in the roof beam, the maximum tensile stress is 2.06MPa in the beam bottom, the stress value can meet the requirements of specification.

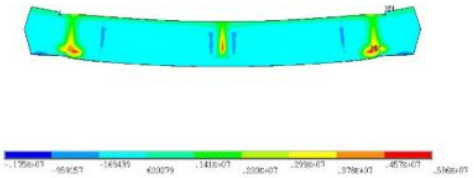
The transverse stress of the beam is approximately between -0.17 and 1.41MPa, and the maximum value is 5.36MPa when the steel beam passes through the beam, but decreases rapidly to -0.17MPa. The reason is that the normal stress is mainly caused by the axial force and vertical force of the main beam.



(a) Transverse normal stress



(b) Vertical normal stress



(c) Normal stress along bridge

Fig. 6 Normal stress of beam

3.3 Stress analysis of steel beam

As can be seen from Figure 7, the maximum Von-Mises stress of the steel beam is 153Mpa, which appears in the intersection of the vertical and horizontal beams in the consolidation zone, and the stress value is not exceeded in the very small range. The stress of the steel beam is small, the stress is mainly distributed in the lower flange of the longitudinal beam, and the stress value is between 17 and 85MPa.

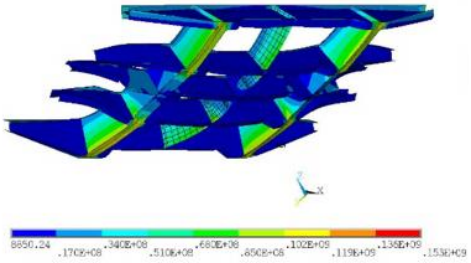


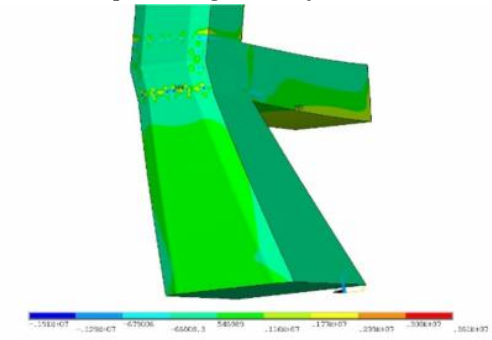
Fig. 7 Von-Mises stress of steel beam

Through the analysis of the Von-Mises stress of the steel beam, the maximum stress of the steel beam is almost in the vicinity of the consolidation of the beam, and the stress value can meet the requirements of the specification.

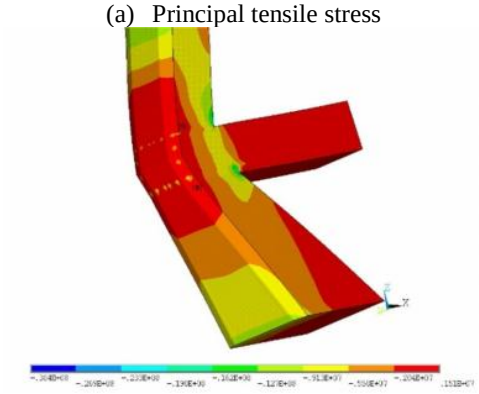
4 Improvement measures

4.1 Stress concentration improvement in consolidation zone

Structural improvement method: in the end of the beam to increase the ordinary steel, prestressed anchor area to increase the anchor plate; in the concrete and steel stringer contact area to add reinforced mesh. The improved stress distribution is shown in figure 8. It can be seen from the figure that the maximum principal tensile stress at the end of the beam is reduced from 27MPa to 3.61MPa, the maximum principal compressive stress is reduced from 88.5MPa to 30.4MPa, and the stress is improved significantly.



(a) Principal tensile stress



(b) Principal compressive stress

Fig. 8 Main stress in the consolidation zone of the improved column pier

5 Conclusions

(1) The steel girder into the tower beam formation consolidation system, forming the structure mode of high strength, high stiffness and good ductility, under the action of the design load, in addition to the local stress concentration, tower pier consolidation area overall stress level can meet the requirements of specification, the structural design is reasonable.

(2) At the end of the cross beam between the lower beam and the bridge tower, and the large steel beam passing through the beam area, there is a large stress. The longitudinal beam of the steel beam passes through the cross beam has a certain level of tensile stress.

(3) The maximum stress of the steel girder appears in the vicinity of the beam pier consolidation.

(4) By adding ordinary steel bar at the end of beam, adding Anchor anchor plate for prestressed anchorage zone and adding steel mesh in the steel beam through the beam area, the stress concentration can be reduced greatly, and the effect of stress improvement is very obvious.

Acknowledgements

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Study on post-fire seismic performance of SRC column-RC beam joints

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ABSTRACT: Pseudo-static tests and parametric analysis were developed to study the hysteretic behavior of SRC column-RC beam joints after fire. Numerical analysis were carried out by using ABAQUS software, and the influence of the parameters, such as heating time, axial compression ratio, concrete strength and steel ratio of profiled steel, on the hysteretic behavior of the joint after fire were investigated. The results show that the bearing capacity and stiffness of joints will degrade after fire. With the increase of axial compression ratio, concrete strength and profiled steel ratio of column, the bearing capacity and stiffness the of SRC column-RC beam joints after fire were improved.

KEYWORDS: joints, after fire, hysteretic behavior

1 INTRODUCTION

Building fire is not only a serious threat to people's lives and property, but also cause damage to the building itself. Sometimes, although there are no building collapses caused by fire, the structure performance must to be estimated in order to decide whether continue to use. Because of the superiority of high bearing capacity, good earthquake resistance and high stiffness, the steel reinforced concrete (SRC) structure are widely used in high rise building. At present, most of the research on the column-beam joints has mainly focus on the seismic behavior at ambient temperature, and little study has been concerned with the seismic behavior after fire.

In this study, three pseudo-static tests were conducted and the influence of heating time on the seismic behavior of the SRC column-RC beam joints after fire was achieved. And then, the influence of parameters on the bearing capacity and stiffness under cyclic loading was researched by numerical simulation.

2 EXPERIM ENTAL PROGRAMS

2.1 Test specimens

Three specimens were used in the test, which named as A, B and C. The main purpose of the test is to investigate the influence of heating time on the seismic behavior of the SRC column-RC beam joints, and three specimens are of the same. The joints 1960×300×300mm for columns and 2870×300×300mm for beams were designed based on the relate Chinese codes (JGJ. 1996, JGJ. 2001, YB. 2007). The H-shaped steel section encased in column is HW175×175 ×7×10, which fabricated from Grade Q345. The longitudinal reinforcement 18mm diameter bar and transverse reinforcement 10mm diameter bar were used for both the columns an beams. The yield strength of bars is 345 Mpa. And the concrete with grade C40 for the specimens was constructed in specimens.

2.2 *Test method*

In this study, two kinds of test were conduct, one is the fire test and the other is the pseudo-static test. Figure 1 shows the situation of fire furnace and setup of the pseudo-static test. The specimen A was the control one, and pseudo-static test was developed directly without fire condition. Specimen B and C were exposed to fire before the pseudo-static test. The specimen B and C were fire tested under ISO834 standard fire (GB/T. 2008), which heating time is 60 and 90 minutes respectively. The fire conditions were simulated in burning furnace with dimensions 3000×3000×1000mm in China Academy of Building Research.

After fire test, pseudo-static test was conducted, and three column-beam joint specimens were tested on a special steel girder frame for the pseudo-static test. Equal and opposite displacement were imposed at the beam ends by hydraulic jacks, and the top and bottom of the column were pinned against translation. An axial load of 0.35 column axial load capacity was applied on the top of columns. The test was displacement-controlled and

the specimens were subjected to an increasing cyclic displacement up to its failure.

Figure 1**.** The furnace and Test setup

2.3 *Hysteretic loops and envelopes*

Table 1 gives the details of the results in test. The yield load Py, maximum load Pmax and ultimate load Pu decreased with the prolongation of the heating time. The bearing capacities of J2 and J3 are 85.2%, 72.9% of that of J1. The ductility of the joints is 2.59, 2.53 and 2.0 respectively, which shows that the bearing capacities of the specimens reduced significantly when the heating time exceed 60 min. Hysteretic loops and envelopes of the specimens are shown in figure 2. Obvious pinch effect was noticed in the hysteretic loops, primarily due to the slip between the reinforced bars, profiled steel and concrete. And the area surrounded by the hysteresis loops of the specimen A is larger than that of specimen B and C, showing that when undergoing fire, the energy dissipation capacity would decreased. From the envelopes, the stiffness of the joint also decreased as the prolongation of heating time.

Table 1. The results of test

Specimen	Py	Δy	Pmax	Δmax	Pu	Δu
	kN	mm	kN	mm	kN	mm
A	67.7	14.8	100.1	33.4	84.6	38.3
B	55.9	16.3	85.2	36	72.3	41.2
C	53.9	18.5	72.9	32	64.5	37.1

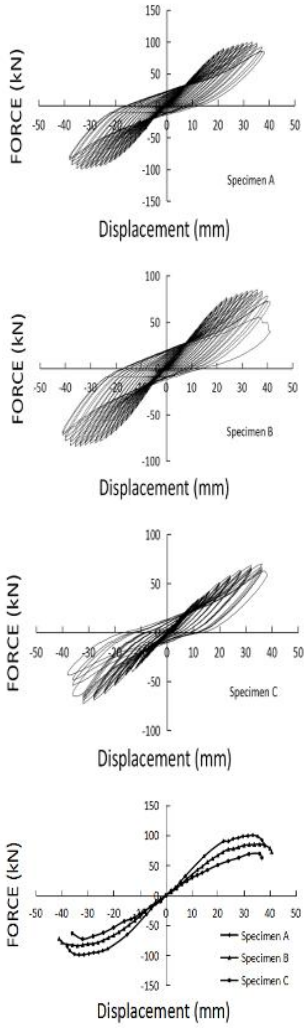

Figure 2. Load-displacement hysteretic loops and envelopes

3 FINITE ELEMENT ANALYSES

Based on the pseudo-static test after fire, parameter analysis was carried out to research the influence of parameters, which are heating time, axial compression ratio, concrete strength and steel ratio, on the hysteretic behavior of joint after fire. Ten finite element models of SRC column-RC beam joint were established by using the general finite element package ABAQUS/Standard v6.10, as shown in figure 3. The parameters are shown in Table 2. For validation, the Specimen J1, J2 and J3 are test piece and simulated by finite element method.

Thermo-structural analysis was developed to simulate the behavior of the SRC column-RC beam joints after fire. For thermal analysis, concrete and profiled steel were modeled with the element DC3D8, which was replaced by the element C3D8R in structural analysis. Element DC1D2 was used to model the reinforcement bar and replaced by the element T3D2

while structural analysis. ISO834 standard heating curve is used as the heating curve and the cooling process is considered to be a straight line. The heat radiation and heat convection are considered in heat analysis. Because of the analysis focusing on the post-fire behavior of the joints, since the temperature field of the joints was got, special program was used to extract the maximum temperature of each element in the whole process of heating and cooling. Based on the maximum temperature of the element, the correct constitutive model of concrete were chosen in structural analysis.

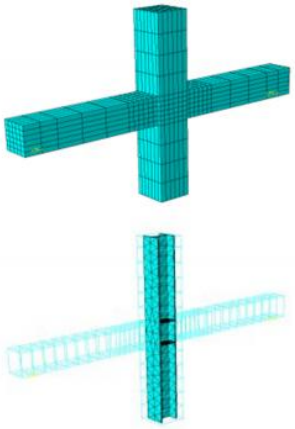

Figure 3. Model of the finite analysis

Table 2. The parameters

NO	Heating Time	Axial compression	Concrete strength	Steel ratio
	min	ratio	MPa	%
J1	-	0.35	40	4.96
J2	60	0.35	40	4.96
J3	90	0.35	40	4.96
J4	120	0.35	40	4.96
J5	60	0.45	40	4.96
J6	60	0.55	40	4.96
J7	60	0.35	30	4.96
J8	60	0.35	50	4.96
J9	60	0.35	40	4.23
J10	60	0.35	40	6.72

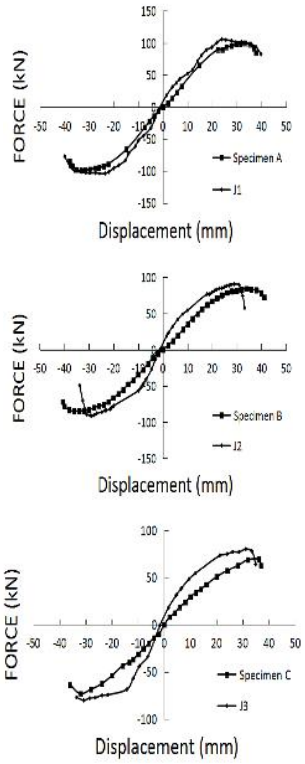

Figure 4. envelopes of the test and numerical simulation

For structural analysis, the concrete and steel are considered as isotropic materials, and the bond slip between concrete and profiled steel is not considered, as well as slip between concrete and reinforcement bars. The relationship of stress and strain of concrete after high temperature was adopted given by Zhoudao Lu (Zhoudao Lu et al. 1993) and the constitutive relation model of steel after high temperature recommended by Linhai Han (Linhai Han. 2007) are used as the material constitutive relation. In order to simulate the damage process of concrete better, the concrete damage plasticity model is used, which could simulate the concrete cracking and crushing conveniently.

Figure 4 compares envelopes of the test with numerical simulation. From the comparison, it can be seen that the bearing capacity give close prediction of the test results. The stiffness from numerical simulation is larger than that of test, which mainly because of some simplification in numerical model, such as homogeneous material of concrete, no slip between concrete and profiled steel, no slip between concrete and reinforced bars and the support restraint ideally. And the different of results from test and numerical simulation is in control scope and acceptable.

4 Parametric study on bearing capacity and stiffness of joint after fire

4.1 *Heating time*

Figure 5 illustrates envelopes of the joints with different heating time. It can be seen that the bearing capacity of the specimen decreases gradually with the prolongation of the heating time. The bearing capacities of J2, J3 and J4 are 85.2%, 75.6% and 54.2% of that of J1 respectively. The stiffness from simulation of J1-J4 are 3.71 kN/mm, 3.15 kN/mm, 2.59 kN/mm and 1.69 kN/mm for three specimens, which show that the prolongation of the heating time would lead to the stiffness degradation. The properties of material deteriorate at high temperature, which cause the degradation of the bearing capacity and stiffness. And when heating time exceeds 120min, the bearing capacity and stiffness decreased sharply, about half of those of specimen J1.

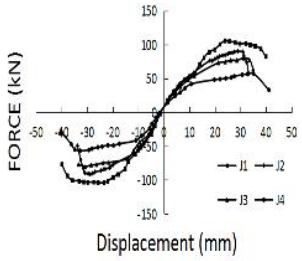

Figure 5. Effect of heating time on strength and stiffness

4.2 Axial compression ratio

According to specification regulated date in the Technical Specification for Steel Reinforced Concrete Composite Structures, the axial compression ratios were assigned as 0.35, 0.45 and 0.55, respectively. Figure 6 shows envelopes of J2, J5 and J6. It is found that the influence of axial compression ratio on the bearing capacity of the joints is very small. The bearing capacities of all three specimens are 90.3 kN, 93.6 kN and 101.39 kN. And the simulation gives stiffness values of 3.15 kN/mm, 3.25 kN/mm and 3.43 kN/mm respectively. So the axial compression ratio increases can lead to the stiffness increase, but the amplitude is very small.

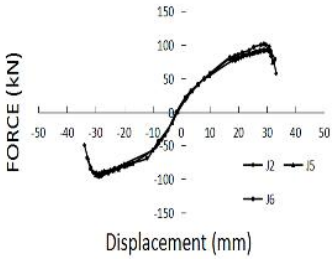

Figure 6. Effect of axial compression ratio on strength and stiffness

4.3 *Concrete strength*

For this investigation, the concrete strengths, C30, C40 and C50, are selected, which are commonly used in engineering practice. The envelopes, as shown in figure 7, of specimen J7, J2 and J8 illustrate that the bearing capacity and stiffness of the joint increased effectively when the strength of concrete is improved, as well as the stiffness, which are 3.07 kN/mm,3.15kN/mm and 3.36 kN/mm respectively. The bearing capacities of three specimens are 82.1 kN, 90.3 kN, and 107.6 kN.

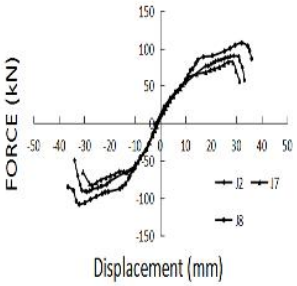

Figure 7. Effect of concrete strength on bearing capacity and stiffness

4.4 Profiled steel ratio

The influence of profiled steel ratio was examined by changing the cross section of steel. The type of profiled steel is based on the regulations of the Technical Specification for Steel Reinforced Concrete Composite Structures, and profiled steel ratio of J9, J2 and J10 are 4.23%, 4.96% and 6.72%. The results in figure 8 indicate that the bearing capacity and stiffness is greater when the profiled steel ratio enhanced. The values of stiffness are 2.25 kN/mm, 3.15 kN/mm and 3.71 kN/mm, respectively.

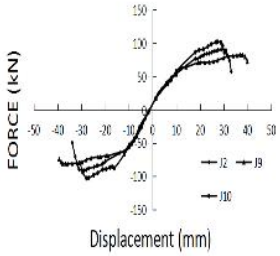

Figure 8. Effect of profiled steel ratio on strength and stiffness

5 CONCLUSIONS

The fire test and pseudo-static test were conducted to study the seismic performance of SRC column-RC beam joints. And then, based on the test, the finite element analysis was developed to have parameter study. The test and numerical simulation results indicate that the bearing capacity and stiffness will degraded after fire, and the longer of heating time the more deteriorating. The increase of axial compression ratioconcrete strength and profiled steel ratio of column all can lead to the improvement of bearing capacity and stiffness the of SRC column-RC beam joints significantly, except for the axial compression ratio, which has small influence on the bearing capacity.

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DAMAGE STUDY ON NUMERICAL MODEL OF REINFORCED CONCRETE BEAM BASED ON ABAQUS

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Key words : reinforced concrete beam; damage; ABAQUS

Abstract

The challenge of the finite element simulation study is the simulation of concrete plastic damage constitutive relationship. This paper carried on the theoretical analysis on the problem based on ABAQUS finite element software, and has set up a model of reinforced concrete beam to study its damage failure process. The results of finite element simulation conform to the damage characteristics of reinforced concrete beam .

1 Introduction

Reinforced concrete structure is one of the most widely used in civil engineering structure, however, as the growth of the service time of reinforced concrete structures, a growing number of reinforced concrete structure damage appeared . Damage accumulation eventually caused the destruction of the structure . Some large reinforced concrete structure, such as high-rise buildings, long-span Bridges, etc., in the event of damage will lead to serious consequences, therefore, research workers around the world have been study the damage of reinforced concrete structure.

2 Theoretical study of reinforced concrete beams

To realize the destruction simulation of concrete in the finite element simulation analysis, the reasonable constitutive relation is required.The constitutive relation of concrete is based on the destruction criterion of concrete.At present, there are no fewer than ten models of concrete failure criteria.

The formula for the destruction of the material is shown below：

$f(\sigma_1,\sigma_2,\sigma_3)=0$ ， σ_i ——Principal stress ；

2.1 Research of concrete constitutive relation

The tensile stress and strain curve are the most important parameters of concrete . The Chinese concrete structure design (GB50010-2010) defines the stress-strain curve of the concrete single-axis tension:

$$\sigma = (1-d_t)E_c\varepsilon \quad (2\text{-}1)$$

$$d_t = 1-\rho_t(1.2-0.2x^5), x\le 1 \quad (2\text{-}2)$$

$$d_t = 1-\frac{\rho_t}{\alpha_t(x-1)^{1.7}+x}, x>1 \quad (2\text{-}3)$$

$$x=\frac{\varepsilon}{\varepsilon_{t,r}} \quad (2\text{-}4)$$

$$\rho_t=\frac{f_{t,r}}{E_c\varepsilon_{t,r}} \quad (2\text{-}5)$$

α_t —The parameter values of the reduced section of the concrete tension and strain curve ；

$f_{t,r}$ —Concrete tensile strength value ； $\varepsilon_{t,r}$ —Concrete peak strain ；

d_t —Parameters of tensile damage。

3 Analysis of damage simulation of reinforced concrete beams

3.1 conditions Establishment of finite element model

The concrete plastic damage model is very extensive in the application of elastic-plastic time analysis .In this paper, the constitutive relation of concrete material is constructed by using concrete plastic damage model.

Use ABAQUS finite element software to create a concrete beam of concrete as shown below .In order to prevent the partial pressure damage of the concrete beam, the steel plate with 6mm thickness is set in the support and stress point.

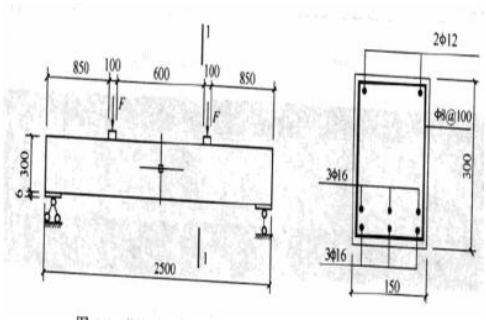

Figure 3-1 Reinforced concrete beam

According to the stress strain relationship of concrete in chapter ii, the constitutive relation of concrete is defined.Concrete USES the C3D8R unit.A total of 1,164 units.Steel rods are used in pole units.Total 900 units.The finite element model of reinforced concrete beams is as follows:

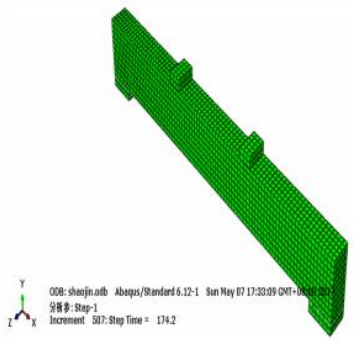

Figure 3-2 The finite element model

3.2 Damage of reinforced concrete beams

Load and boundary conditions are applied to the finite element model.Add constraints on the left and right sides of the beam.Put the displacement load on the block of the top of the beam.The displacement curve is shown below:

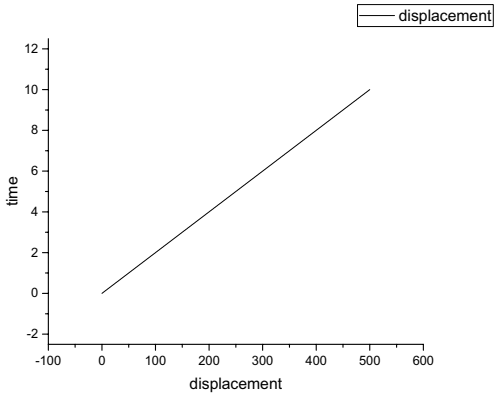

Figure 3-3 Displacement curve

The stress cloud diagram of the reinforced concrete beam is shown below:

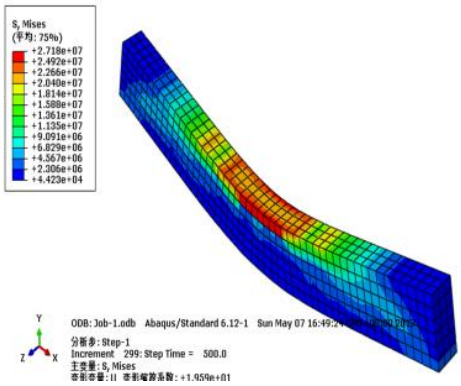

Figure 3-4 stress cloud diagram

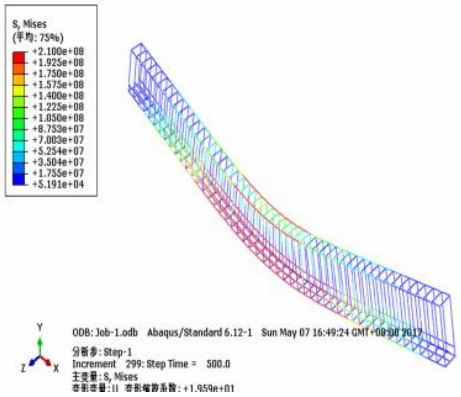

Figure 3-5 stress cloud diagram

We can see it in the figure above : The maximum stress of the concrete part occurs on upper margin .The compressive stress reached 27.2 MPa . This indicates that the concrete has been crushed . And the maximum stress of the bar occurs in the lower part .The tensile stress is 210MPa.It has reached the limit of tensile stress of the steel bar .It shows that the bar is pulled down.

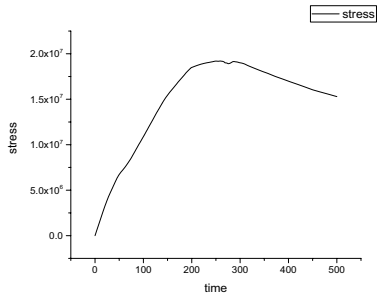

Figure 3-6 condensate stress strain curve of the upper part

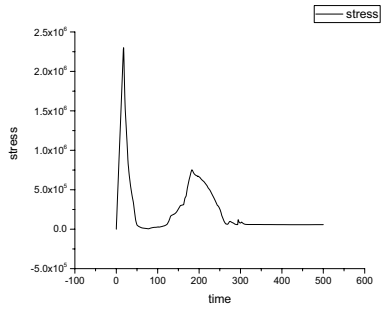

Figure 3-7 condensate stress strain curve of the lower part

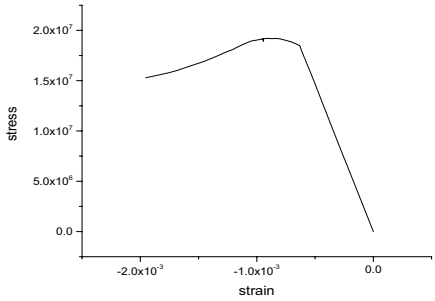

Figure 3-8 Stress response curve of concrete compression area

We can see it in the figure above .The tension area of the reinforced concrete beam was pulled down during the second 20s.The pressure zone worked until the 260s.

We can see it in the figure above . The steel bar first folds. With the development of the deformation of the tensile steel, the concrete meets the extreme pressure strain and the concrete is crushed . This form of destruction has obvious signs before the destruction .The crack and deformation of the anterior fracture are rapidly developed . This is also known as ductility destruction.

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Rock cut versions of Armenian churches and mythological and social grounds of the origin of their architectural composition

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Abstract

Armenia has a considerable number of rock cut churches. A small part of them are operating ones, For example, Geghard has been continuously operating for nearly one thousand years. From purely architectural point of view churches are of two types: basilica and central domed. Basilica churches are more popular, with the origin and functionality of their composition being attributed to the social demand and the structure of hierarchic classification of social relationships: the king, priest elite, nobility, priests, and people[Toramanyan1942]. The composition of a central domed church by its origin is perhaps anchored in the ancient Armenian mythological notions of the structure of Universe[Sahinyan2012] . Compositions of both these types have dictated the selection of structures and the architectural tectonics.

Key words: rock cut churches; Armenia; basilica and central domed cruciform planned churches; composition and tectonics; social demand, mythological basis

The scientific objective: Architecture of sacred structures and in particular architecture of temples is anchored in social classification and politics, faith and its dogmas. Depending on the faith of a particular people and changes in that faith in a historic period, as well as serious change in the demographic composition inside the state or radical change of ethnoculture, architecture of temples and sacred structures has been changed respectively. For example, architecture of Pharaos, Ptolemy and then substituting the latter Muslim Egypt has undergone fundamental changes despite the fact that the main ethnic composition of population remained unchanged. We can see changes in the architecture of sacred structures also in case of monuments of Etrusk, Roman, Christian Italian, Persian Ariarbarzan or Ariarmen[Nersisyan2011], Cyrus[Xenophon of Athens 1970], Zoroastrian[[Nersisyan2011; Shahinyan2012;Agatangegos 1987] and Muslim period. In Armenia, we can see vast changes in architecture of sacred structures and temples of 4^{th}-3^{rd} millennia, Kingdom of Van (Urartu), Hellenistic and Christian periods. And this is despite the fact that, starting from 6th century B.C. up to Late Middle Ages, the ethnic, native population and the principles of royal administration and social and class order have not undergone any fundamental changes. Only religious views and the religion itself have changed[Shahinyan 2010, Marr 1922, Dyakonov1989, Movsisyan2002,].

Thus, objective number one: what kind of natural and anthropogenic factors have influenced the architectural formation of sacred structures and in particular churches.

Objective two: what is the reason for the longevity of the style and composition of sacred structures and particularly Armenian churches over considerable time periods, more than 1,600 years.

Research: Armenian churches have been continuously written about since the 5^{th} century, during the entire period of Christianity, and starting from the 19^{th} century, numerous scientists have been involved in issues of church architecture and a great amount of materials were published about the results of scientific researches.

Among the vast amount of scientific viewpoints published about basilica, TorosToramanyan's scientific hypothesis on architecture of basilica type temple structures is favorably distinguished. According to Toramanyan, architecture of basilic churches is a result of thousands years of development of temple construction and the main impact on its formation have been the social relationships in empires having clearly differentiated hierarchic class structure. To support this viewpoint, he gives the example of plans of Ancient and Middle Egyptian Kingdoms, Hellenistic and Greco-Roman temples [Toramanyan1945].. From his explanation the functionality of temples is clearly

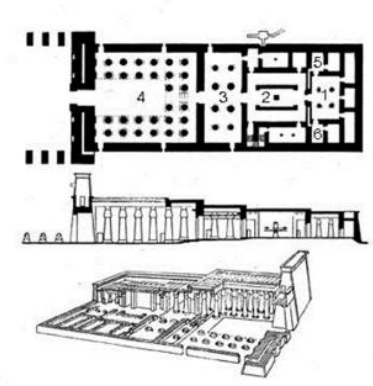

Figure 1:The plan of EgyptianTemple. 1.Area thePharaoh-priest 2. The area for Priest and Pharaoh's family 3. Priest 4.The aristocracy and military elite 5-6. Treasury

emphasized which is anchored in the empire's hierarchic, religious-civil class structure and functions and rights of each of its strata.

The other principle that has served a basis for formation of the architecture of center domed cruciform planned churches is based on the mythological worldview of Armenians since ancient times about the creation and structure of the Universe and the Earth. Reasonable figures available in Armenian petroglyphs, rugs, decorative art of medieval manuscripts and in mold frames of khachkars (cross stones), and the constructive structures of the composition of mausoleums with circular semispherical stone-hill and stone-box burial chambers built in 4^{th}-3^{rd} millennia B.C. become meaningful only within the ancient mythological notions of Armenians about the Tree of Life, Dragons, the quadrate structure of the Earth, the Fruit of the Tree of Life, and the structure of Universe.

Architecture of center domed churches, as a classical model and as structures radically different from other compositions were recorded by TorosToramanyan. Later on, in the 20^{th} century, scientific researches and publications were provided by Strzhigovski, Tokarski, Jaconson, Yaralov, Mnatsakanyan, Chubanishvili, Safaryan, Taghayshvili, Khalpakhchyan and others. VahagnGrigoryan published in 1982 a monography "Early medieval central domed small monuments of Armenia", where he brings together all works previously done and performs a typological classification, as well as TelmanGevorgian's "The proportions in Armenian Architecture", which studies the numerical relations and proportions of constructive structures of churches based on units of measurement used in ancient and medieval Armenia: dzogh ("rod"), krknaqayl ("double step") girk ("embrace"), votn ("foot"), tiz ("span"), etc. [Gevorgian2012]

However, the works of all these scientists are based on the tectonic peculiarities of the structure which eventually leads to analysis and description of achievements of engineering technologies of structures and cannot have national features, while studies of the composition do not indicate any traditional, ethnocultural bases of origin and establishment of the form and style. Therefore, it is difficult to ground with this methodology the purely Armenian origin of this type of churches.

In case of decoding of the esoteric meanings of the all the following is comprehended: the traditional constructive trinity of churches, church windows, doors, defensive towers framed with depictions of two cosmic mythological elements (), Black Water, and Purple Sea that separate the regulated universe of evil and good and protect it from endless chaos) boundaries of the field occupied by depiction of Tree of Life appearing in the form of khachkars and cross images, etc. Numerous examples of application of the symbol of the sacred act of creation in small size architectural monuments (Odzun) and in stylization of church entrances; application of the Sun – the fruit of the symbol of the Tree of Life in the form of a new symbol of Holy Cross adapted to and meaningful in Christianity; application of the symbol with rectangular drawing of the Earth in petroglyphs, esoterics,
pictograms, and planning under the domes of churches
that characterize the Earth's sphere.

Figure 2:Tree of Life and Dragons

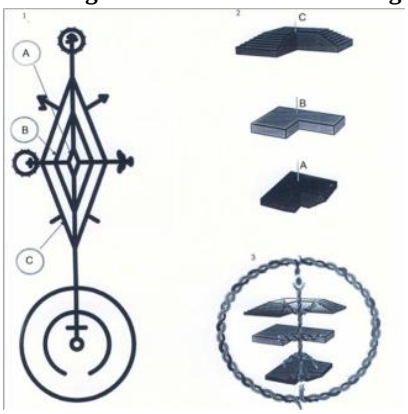

Figure3: 1:Mythologicalconstruction of Universe 2:A.UndergroundB. EarthC.Sky3: Universe

Furthermore, the portrayals of man, goat, bull, eagle that characterize the four sides of the Earth and, simultaneously, the four elements – soil, fire, water, and ether, as well as, characterizing the sphere of bowels of the earth, three, five or seven platforms of square drawing and developing gradually upward and placed one on the other in the base of churches or in compositions serving as a pedestal for khachkars and monuments.

Social bases for architecture of basilicas and temples: Architecture of basilicas has more often been anchored in social structure and respectively on public demand. The functionality of the structure was a public demand, which had been formed on the bases of exercising the rights of people inside the church or temple according to the class structure of the society during pre-Christian and Christian periods. This point of view is shared by part of the 20th century researchers of the Armenian architecture, while it was most convincingly presented and well-grounded by TorosToramanyan.

Toramanyan believes that the cult of deceased has appeared among the tribes that had mastered agriculture and metalwork; in the architecture of the mausoleums dedicated to them legends associated with faith and worship were already outlined. In the church architecture, starting from the ancient times, the concept of the eternity of life and rebirth was valued since the idea of immortality and heavenly life still holds an important place in people's mentality. From the plan structure of temples, three functional components are clearly distinguished: sanctuary, functional areas for priests and prayer hall.

It is natural that the prayer hall is the largest both in ancient temples and in churches, while the sanctuary that contains a sculpture to God and the treasury with different items gifted to a particular god by believers were inaccessible for the public at large.

Mythological bases of the compositions of Armenian churches: In the ancient Armenian mythological notions, the Earth has a square drawing; there are references on this in medieval books. Bowels of the earth consists of a number of layers; mainly seven layers are brought up, where the lowest layer is followed by the Black Water. The sky looks spherical, where the top of the Tree of Life reaches. Every day the Sun is born from a bud of the Tree of Life and is protected by the White Dragon which also protects the Earth and the Sky from external sinister forces or from chaos. Every night the Black Water of Bowels of the earth or the Black Dragon steals and takes the Sun away. The Black Dragon also has a protective function though: it protects the world in the Bowel of the earth or the mythological Dark World and people living there. The Black Dragon is also the protector and the bearer of waters and gives birth to lots of springs from his wounds after he dies.

Thus, according Armenian and in some case common mythology preserved among the Indo-European tribes, the Universe has a form of Sphere or consists of two spheres. In the surface of the cross section of the sphere the surface of the Earth is placed with square drawing, with its landscape arising upward, and below that plane there is the Dark World or Bowels of the earth. Thus, the Universe is three-storied: Bowels of the Earth, Earth, and Sky.

Results of the research: Thus, an important situation is outlined both for temples and churches built on the ground and for the rock cut ones: the architecture of part of mausoleums, antique Egyptian and Greco-Roman temples as well as basilicas is anchored in the concept of the eternity of life or immortality or the rebirth. While from the functional standpoint, the sections of temples where the "presence of God" is ensured and the small cell where treasures offered to God are kept as well as areas prescribed for the functions of the priest and pastor class were inaccessible for believers. The prayer hall for believers did not have any direct linkage with the sanctuary section from the functional point of view, while in some cases it was not linked with the temple at all. We can see this phenomenon in Egyptian (Temple of Ra), Sumer (Zikkurats), Urartian (Ayanis in Van, "Sussi" or Arevi in Erebuni), Assyrian, Mesopotamian, and Hellenistic temples. Architecture of these temples is mostly dedicated to a particular god, and only the chosen people from the class of priests and pastors were entitled to enter the temple and the room where the gifts dedicated to that god were put. The prayer hall was located mainly outside the temple and it could be a facility with columned roofing, such as in the case of Ayanis Temple or simply an area adjacent to the temple. This phenomenon can be seen even more vividly seen in the

case of holy temples placed on the top of pyramids in Aztec, Olmec, Ink, Maya cultures, where the faithful people were nestled in the area outside the temple and the pyramid. Thus, it is quite difficult to find the spiritual bases of the architecture of basilica and national features in the principles of functional differentiation of constructive components of composition given that the concept of immortality or rebirth is universal. Therefore, starting from the ancient temple construction up to the basilica, architecture is based on the entreaty from a particularly worshipped God or gods about eternal life or immortality. This means basilic church is basically a result of an international thinking, and only on case of availability of a few details of national culture we can perform classifications for distinguishing it as Italian, Greek, German, Georgian, Armenian, etc.

The situation appears to be different when architecture of central-domed cruciform planned Armenian churches are viewed under the light of Armenian ethnoculture and myths preserved by other Indo-European peoples, where we can see that in the architecture of sacred structures myths about the Tree of Life, World-Destroying and World-Creating Dragons, Life Seed or Cosmic Egg, square structure of the world, the three-storied universe, and the fruit of the Tree of Life, i.e. the Sun, are still alive at least in the Indian mythology and Vedas. We can observe this phenomenon in case of the architecture of earth-hill and stone-box mausoleums both in the Armenian Highland and in Iran, in the ancient civilizations of the Mediterranean Sea basin and in the Irish Newgrange. All these contain all or part of mythological components: cromlechs arranged along the external circle of mausoleums of 4^{th} to 3^{rd} millennia B.C. in the Armenian Highland or the enormous fined stones arranged along the circle of the mausoleum in the basin of River Boyne in Newgrange, Ireland, which probably embody the Dragons. The planning structure or the plan of the burial chamber or burial area with square or cruciform drawing as well as the deceased contracted and placed like an embryo in the maternal womb that can often be seen, which portray the mythological seed from which the Tree of Life will grow and yield fruits.

All world-creating elements the architecture of central-domed cruciform planned Armenian churches as well as all world-creating elements have preserved in the mythology of Indo-European peoples.

Summary: In the bases of the architecture of sacred places and temples lies the need to create necessary functional volumes and space for worship of gods or God. Basilic churches with their medieval planning and planning principles are directly linked with the architecture of sacred places of the Old World, where the sanctuary is separated (the stage and the altar or niche where the image or a sculpture of a saint or Christ is placed), with the priests' rooms and the prayer hall to the right and to the left of the stage. Developments of the architecture of basilica in different countries, because of creative

approaches and social order and demand, have formed this diversity of styles such as gothic, baroque, etc. And to resolve issues of tectonics, engineering solutions and technologies have been found such as use of half-arch supporting column to strengthen the external walls to prevent the ceiling from collapsing in French and English churches in the period of Renaissance.

The architecture of the Armenian central-domed cruciform church was immediately formed in the 5th century and has never undergone any compositional change to date since the constructive structural details are directly anchored in the ancient mythological model of the structure of the Universe. Occurrence of the rock cut versions of the central-domed church in the Armenian Highland and other places was due to the intention to hide from enemies or the unorthodox. However, for rock cut central-domed churches it was a mandatory requirement to have "yerdik" (a round or square opening through which smoke goes out and the light penetrates) in the central part of the dome of the ceiling, in the place of attaching the cross, wherefrom the light was falling on the believers.

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ELEMENTS OF CAPITALS' ORDER SYSTEM IN THE ARCHITECTURE OF THE ANCIENT ARTASHAT

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Keywords: Ionic order, module system, temple.

Abstract
A podium of three meter height has been cleared on the upland of a rocky ledge during the excavations of recent years on the territory of the lower town of the capital of ancient Armenia, Artashat (1st century AD).

Later, the following reused architectural fragments have been found in the walls of nearby later-constructed buildings: a large number of fragments of Ionic capitals, as well as the whole capital, fragments of an architrave, a frieze, fluted column shafts and bases (attic and toroidal) etc.

Discovered fragments were measured and researched. Based on the measurements and taking into account the proportions of the Ionic order, the module of the supposed Ionic temple was established. It is equal to the lower diameter of the column shaft in accordance with the modular system of Vitruvius.

Based on this module, we attempted to reconstruct the order system, i.e. to determine the column absolute height, as well as to establish the height of the entire temple construction. We carried out a comparative analysis with the only remaining Ionic temple in Armenia, the temple of Garni constructed in the first century A.D.

Presumably, the podium belonged to an ancient temple that was the dominant structure on the territory. With the adoption of Christianity, the temple was totally destroyed, while the podium ground with dimensions of about 22.8 m x 32 m., where the temple was probably located, was preserved.
These artifacts actually belong to the second Ionian temple of the same period (1st century AD) found on the territory of Armenia.

1. Introduction

Since 1970, the systematic excavations of the capital of ancient Armenia, the town of Artashat, have been conducted. During these years, separate sections of the citadel hills were excavated [1, 2]. A podium of three meter height, composed of local semi-processed large marble blocks, has been cleared on the nine-meter upland of a rocky ledge on the territory of the lower town [3, 4] during the excavations of the recent years. We assume that an ancient temple was constructed on this podium. It was destroyed in the 1st century AD by Roman commander Korbulon and was then rebuilt in the second half of the same century [5].
The reused architectural fragments of the temple were revealed in masonries of the wall upper rows of nearby later-constructed structures during the excavations, which indicates the destruction of the monumental construction after the adoption of Christianity.
Before this discovery, the only preserved ancient temple on the territory of modern Armenia was the temple in the fortress of Garni, which was restored in 1975 [6]. The temple of Garni manifests similarities between the architecture of Armenia and those of Greece, Roman Empire, and first neighboring Hellenistic states (Asia Minor and Syria) [7,8]. Based on the ancient records, the temple of Garni was erected in the second half of the first 1 c. A.D. in the place of an ancient sanctuary. The same written sources state about large construction works in Artashat at the same period [9].
The study of the architectural fragments discovered in Artashat showed the fragments are of different architectural orders. Two fragments of the capitals of the Corinthian order, a large number of fragments of Ionic capitals, a fragment of an architrave, fragments of various friezes, cornices, many fragments of smooth-surfaced and fluted shafts were discovered.

Among the found fragments of column shafts were also fragments in which the flutes are separated by a sharp edge, and not by a flat fillet. This indicates their belonging to the Doric order. The toroidal bases of various sizes discovered during the excavation also belong, apparently, to the Doric order. Fragments of the Corinthian order and more archaic stone fragments

of decorated by a rope ornament were also found. The material of all found fragments is the local white finely porous limestone.

All the fragments we found were sorted by order types. There were much more fragments of the Ionic order among the findings, which made it possible for us to take them away, study, measure and restore the rack-and-beam system of the presumed temple, dated back to the 1st century AD.

The findings also include a large number of fragments of Ionic capitals, as well as a one-piece capital, architrave fragments, fragments of various friezes, cornices, attic bases of columns, and a large number of fragments of smooth-surfaced and fluted shafts. There are fragments of shafts, the lower part of which is smooth and the upper – fluted.

The study of these fragments made it possible to establish the module of the presumed Ionic temple, which should be equal to the lower diameter of the column's shaft in accordance with the Vitruvius's modular system [10]. Based on measurements and taking into account the proportions of the order, the size of composite parts of entire order system were determined and restored.

2. Column shaft

About twenty fragments of column shafts of different degree of preservation and sizes were found and measured (Fig. 1, 2).All of them were found in later-constructed buildings where they were reused in the foundations of the masonry of the walls. For this reason, one or two side of these drums were cut out, so that a stone was laid on them. Discovered fragments of the shafts are carved, as was mentioned above, from white finely porous limestone and are very poorly preserved. The stone is scattered from dampness and, obviously, from fire. For this reason, it is very difficult to determine their absolute sizes.

Fig. 1 Fragments of column shafts

Fig. 2 Partly fluted column shaft

As the study showed, the shafts of the columns were built from several drums, which were vertically connected between themselves by metal or bronze pins (the pins were not preserved). For this purpose, rectangular grooves were made in the middle part of the upper and lower horizontal surfaces of the drums. The reserved dimensions of the grooves are 6cm x 9cm, 7cm x 6cm, 5 cm x 5.5 cm, 6cm x 5 cm with a depth of groove up to 7cm; one smooth drum, 112cm in height, has a groove of 11cm in depth (the bottom drum).The drums of the shafts are broken off and it is difficult to determine the absolute height of each drum, as well as the number of drums of the whole shaft. Among the shaft drums, drums with smooth surfaces were also found. Along with the broken drums, a single holistic drum was found, 142cm in height and with a diameter of 69-70cm. We found fragments of shaft drums with a smooth lower surface, and fluted upper surface. The flutes are separated not by an acute edges, but by smooth strips – fillets – that confirms their belonging to the classical Ionic order. Measurements showed that the bottom diameter of the column shaft drum is 69 cm-70cm, which determines the modulus of the entire temple. It is interesting that the module of the Garni temple is also equal to 0,69m.

According to the findings of the drums of the columns and to the presence of flutes under the capitals, it is evident that the shafts of the Ionic order were smooth at some height and fluted higher up to the capitals. The width of the fillets at the bottom is 3cm and, gradually decreasing, becomes 2cm under the capitals. The width

of the flute groove decreases from 6cm to 4.7cm at the top (under the capital) the depth of the flute half-oval is about from 3cm to 2.5cm. Because of the poor preservation of the drums, which were broken off from both sides in antiquity, it is difficult to determine the height of each drum of a shaft, as well as the number of drums of the entire shaft. Calculating the entire height of the column (shaft) in accordance with the module, and having a holistically preserved drum (height of 1.42 cm), we concluded that the entire shaft consisted of five drums, which makes it possible to restore the entire column.

Since the module is 69cm, and the height of the entire column is 9.5 modules by Vitruvius [7], it turns out that the entire height of the column was 6.55m, of which the column shaft is 5.9m long, the rest are the base and the capital. On the lower, non-fluted part, the marking of flutes was done. The height of the column drums, according to our calculations, decreases with height.

3. Column bases

During the excavations, attic bases of columns were also discovered (Fig. 3). As it is known, an attic base was used for the Ionic order, an example of which is the base of the Garni temple in Armenia [6]. We discovered a whole base 25 cm in height without a plinth.

Fig. 3 Attic base of column

The base has two convex moldings called tori which are separated by a scotia. The diameter of the base is 47cm, which does not correspond to the lower diameter of the shaft under discussion. Small fragments of attic bases with a larger (than 47cm) diameter were also discovered, unfortunately they were found in the form of small fragments without a plinth.

The restoration of one such fragment showed that its original diameter corresponds to the shaft lower diameter investigated above. Proceeding from the calculations and taking into account the theory of Vitruvius, it is possible to determine the approximate height of the base of the column, which according to our modules is 0.345m.

4. Capital

Reused fragments of Ionic capitals of various sizes and preservation degrees were discovered during the excavations of the walls of the later-constructed buildings (Fig. 4,5,6).

Fig. 4 Ionic capital of order

Fig. 5 Bolster of Ionic capital

A well-preserved capital was found. Based on the entirely preserved Ionic capital, it is possible to establish its absolute dimensions.

Fig. 6 Ionic capital with column shaft

The crowning part of the capital was an abacus with dimensions of 0.61 x 0.61m in the form of two not wide slabs with a total height of 6.3cm. The capital consists of two symmetric volutes (spiral scroll with a small circle in the middle– an “eye”). “Eyes” of the capital volutes are flat with a diameter of 4.1cm. They do not go beyond the general vertical line of the capital. The same dimension and shape have all the

fragments of volutes, which indicates their belonging to the same order of the temple.

Spirals of volutes are edged with a thin strap, and the planes of volutes and central pillows are deepened with scotia (hollow concave molding), which is peculiar only for the architecture of the Roman period, where the contrast of the image was achieved by the light-and-shade play. Only the echinus, compressed by volutes, is significantly forward compared to the vertical line of the capital; it is located in the middle part of the facial sides with a width of 24cm, enriched with a necklace of three egg-and-dart ornaments, made in relief. The echinus ovolo has a profile with carved eggs in well-rounded borders, separated by arrows.

On each side, there is an ornament of palmettes in the form of leaves, covering the side eggs. The sides of the capitals are joined together by so-called bolsters, which are very similar to scrolls. These bolsters are smooth and not enriched by ornamentation, in the middle part they are decorated with two bundles strapped at a distance of 12cm. The middle part of the bolster is convex due to tightening. Under the volutes there is a narrow profiled profile, and then flutes with the following dimensions: the width of the groove is 5cm, the width of fillet is 1.5cm. The entire capital, together with the abacus, is carved from solid stone. The echinus of the considered capital was carefully decorated with a cut, a narrow convex half-molding and a concave scotia.

There are rectangular recesses on the lower horizontal surface of the capital in the middle part for the vertical fastening of the capital with the column shaft with the help of a metal pin (metal traces are not preserved). Dimensions of the recesses are 2.5m x 3cm with a depth of 4.5cm. The upper surface of the preserved entire capital has two rectangular recesses in the middle part spaced from each other by 11cm, with dimensions of 6cm x 6cm and a depth of 7cm. These recesses were used for vertical fastening with architrave. The entire height of the capital is 34 cm, the width of the capital from the center of the volute to the other center of the volute is 50cm.The total width of the facade of the cornice is 0.83m. The diameter of the lower part of the capitals is 51cm, while that at the base of the shaft is 69cm. It indicates the narrowing of the shaft and determines the enthasis of the column.

5. Architrave

A fragment of an architrave of a small size was also found (Fig. 7, 8). Architrave is broken, it is divided into three bands or fasciae, each projecting beyond the other, with a distance of 2cm between them.

Fig. 7 Fragment of lower part of column architrave

Fig. 8 Fragment of upper cornice of architrave

The lower part of the fragment is well processed; the upper band is broken and the real height is not preserved. The preserved height of the found fragment is 30cm, and the width is 36cm. The height of the first fascia is 8cm, the second – 10cm, and the preserved height of the third broken one was 12cm. According to the calculation based on our module of 69cm and proceeding from the proportions of the Ionic order, the height of the architrave should have been equal to 59cm. Considering that the real height of the broken upper part was for example 14cm and not the preserved 12cm, then the height of the architrave cornice could be approximately 27cm.

The band of "*dentils*" was usually located between the architrave and the frieze. A large number of small fragments of broken dentils of different sizes were found during the excavations. These fragments were measured and studied, and the value for complementing the architrave cornice height was approximately determined. The preserved height of the stone with a dentil is 25 cm. Both the lower and the upper parts of the stone are broken. The lower part of the front side of the stone is plain, then there is a row of dentils with a height of 8cm, the width of the dentils is 8cm, the space between dentils is 4cm.Since the stone is angular; the last dentil in the row has a width of 3 cm. An astragal with a width of 2cm is above the dentil row spaced from it by 1.5cm; then there are

smooth ribbon grooves, 3cm in width, 1.5cm in depth and again an astragal – 2cm wide. Above the last astragal, there is a preserved corner with a broken egg (the height of the preserved egg is 5.5cm, and the width – 8cm), the depth of the stone is 23cm. The facade width of the stone is 25cm.

6. Frieze

Various ornamented fragmentary debris, presumably friezes, broken from both sides, were found. Among the findings there is a fragment ornamented with archaic ornamentation, which does not correspond to the common ornamental style of the temple in question. Presumably, it is a fragment of the frieze of a more archaic temple. Other fragments having a rich ornament with a deep cutting are very small (fig.9).We chose a stone (a continuation of the architrave cornice) with egg ornaments at its lower part.

Fig. 9 Fragment of column frieze

The height of the egg is 13cm, which corresponds to the height of the fragments of the preserved architrave eggs. Complementing the architrave height with those of dentils and eggs, we get a value of 59cm, which is proportional to our module. Above the eggs, a smooth band of 3cm in width and a ledge of 3cm is projected. The band then deepens by 2cm.The plain surface of the frieze is ornamented with a spiral ornament spaced from the surface by 1.5cm. Unfortunately, the whole picture of the ornament is not visible due to the poor preservation of the stone. The stone is broken from above, as well as from the sides. The height of the upper preserved part of the frieze is 10-12cm. Proceeding from our module, the height of the frieze should have been 31.74cm. Restoring the whole picture of the frieze ornament, we get the desired value. The investigation reveals that the discovered cornice of the architrave and the frieze were carved from one stone.

7. Cornice

Crowning projecting moulded horizontal top of a column is cornice (Fig.10). Among the discovered fragments of stones, a corner stone with a preserved height of 23cm, a width of 32cm was found. The upper and lateral parts of the stone are broken. The lower part of this stone is decorated with dentils of 8.5-9cm wide, spaced from each other by 3cm, the projecting part is 6.5cm. The height of the dentil is 7.5cm.The bottom surface of the stone is well processed.

Fig. 10 Fragment of column cornice

The upper face of the stone above the dentils has a preserved projecting part of 5cm; it is broken and it is impossible to restore the ornamentation. Both the size of the projecting part and the size of the dentil confirm that the stone was a cornice. Processing from the calculation based on our module, the height of the cornice should have been 72cm, of which only 23cm remained. Thus, the height of the entire entablature of our order is 1.63 m, which corresponds to our calculations.

A proportional analysis of the preserved details confirms that all elements can be of the same order system. The ratio of the upper diameter of the columns adjacent to the capitals and the lower diameter of the columns resting on the base is 11/15, which fully corresponds to the requirement of a proportional ratio of these parts.

8. Conclusion

Thus, an initial attempt was made to restore the entire order system of the Artashat temple. The discovered fragmented derbies of the ancient temple in Artashat are the fragments of the second known temple structure in Armenia.

The comparative analysis of the fragmentary elements of the Ionic order found in Artashat with the elements of the Garni temple showed their similarity. They have the same module equal to 0,69m. However, while

basalt was used in the Garni Temple as a building material, in Artashat it was fine-pored limestone. Ornamentation of order elements are also different. Both the architecture of the Artashat and the Garni Temples have a certain commonality with the Asia Minor similar structures of the same period.

The Armenian architecture of the antique epoch, being in close contact with architectural art of Greece, Roman Empire and particularly neighboring Hellenistic countries (Asia Minor, Syria, Persia etc.) produced remarkable monuments peculiar to the period. However, based on rich local traditions, characteristic of Near Asia art, it had its original features of development.

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THE CHURCHE IN PAINTER SLAVIK PARONYAN`S LANDSCAPES

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Keywords: landscape, painter Slavik Paronyan, church, Saxmosavanq.

Abstract

Churches-the standing and tumble-down bearers of the Armenian faith, having survived through century`s hardships, permanently have been depicted by Armenian painters. The national artist, honorary professor of the Armenian National University of Architecture and construction Slavik Paronyan, maybe a not very well-known landscape painter for many of us, thought it important to paint churches in his painting. In these exceeding one another works, even in the farthermost corners we`ll see one of majestic Armenian churches. Among medieval Armenian prominent sacred and cultural centers especially Saghmosavanq was the master`s rudiment, the central figure of his landscapes. And it was mainly reasoned by the fact that the painter was living just next to the church. So he had the opportunity to admire the powerful and imposing presence of this noble building in all seasons and at any moment of the day… So the main purpose of this paper is to identify those artistic characteristics, that are applied to the ecclesiastical theme in his works. Besides, there are comparisons with other painters` landscapes, where churches are depicted too.

1. Introduction

People`s Artist of the Republic of Armenia (2012) Slavik Paronyan (1929-2014) who was a member of the Artists` Union of Armenian SSR (1956), Artists` Union of the USSR (1960), a Merited Artist of Armenian SSR (1983), is one of those outstanding representatives of Armenian Soviet art, who spent his whole creative life painting Armenian nature. His contribution to the history of Armenian art together with the development of landscape painting is quite a lot. Only the fact that in one of All-Union exhibitions taking place in Moscow in 1983, Slavik Paronyan was awarded as the best landscape painter, is telling more about him and his landscapes. Now many of these works are wonderful exhibits of different museums, galleries and private collections. During his lifetime themaster repeatedly exhibited his works not only in Armenia, but also abroad. From 1971 Paronyan permanently settled in Saxmosavan, living here an interesting creative period. Especially from this point he started painting churches, and most of all Saxmosavanq. Not all his landscapes are represented in this article but especially those, in which he painted churches. Among them there are many works, that have never been shown to the public before.

2. Results

As has been previously mentioned, the rudiment of Slavik Paronyan-the representative of Armenian classical realistic art - was landscape, although he has worked also in other genres, creating many unique portraits and still life's. It's interesting, that all his works tell only about Armenia, about its unique nature, about high style highlands, about Armenian fertile soil and in general about "Armenian". In this context, it is impossible not to include Paronyan's images of stone giants-churches and monasteries, for centuries silently bearing on their shoulders the Armenian long-suffering faith embodying the spiritual life. As we know, these Armenian magnificent architectural monuments were displayed still in the Middle Ages. In this way the presence of "spiritual" has been easy to make most picturesque in icons and miniatures. In 19-20 centuries the image of the church has also been interpreted differently along with the classical understanding of the landscape. It will acquire a new artistic quality, aligning with miniature, generalizing national identity and national consciousness. So, instead of purely religious, spiritual understanding of the concept of them modern Church's image is replaced the alternative interpretation expressing infinite love and care for the motherland, through it emphasizes the national concept.

On the other hand, if we desire to apply Slavik Paronyan`s landscape genre, it is worth considering the important fact that the young artist has studied at Hovhannes Zardaryan, the prominent representative of Armenian Soviet art, whose name is linked with landscape monumental-decorative development, come from "Saryan school". Let's remember, that the founder of the national school of Armenian painting M. Saryan created broad spatial landscapes, where at the same time we can see monumental character of collective, bright and decorative native land [1]. Master`s such pantheistical ideas, the importance loudly to praise national spirit in his works passed from generation to generation and roots in the works of numerous artists, created the history of Armenian art.Slavik Paronyan`s even the most chamber landscapes also have epic sounding. Among them there is no landscape where in the environment of residential or abandoned houses, or of churches, monastery complexes, composed Armenian crown of architecture are found as extensive fields, covering the whole painting, giant massifs, flying clumps of clouds. These paintings complete his innermost feelings about his native land. This is manifested both in architectural images of mountains Ararat, Aragats, lake Sevan, considered as symbol of homeland, and in simple landscapes, presenting everyday life. And as art critic Knarik Avetisyan says in her introduction of the book, dedicated to the 150th anniversary of the birth of prominent Armenian architect Toros Toromanyan: "The images ofthe Biblical Mt. Ararat, lake Sevan,Aragats, and the Armenian churches, besides being artistic embodiment of specific site and a spiritual structure, are also perceived as an important part of the cumulative picture of our country" [2].

With the exception of student ten years, we can say that Slavik Paronyan`s mature creative life lasted 60

years, of which 45 he lived in Saghmosavan – in this beautiful nature of Aragatsotn region away from the urban noise. In many ways the artist's life remind his teacher`s Hovhannes Zardaryan`s passed way, whom the artist connects his first steps in the art. This prominent representative of Soviet Armenian art lived throughout his life far from the town - in Byurakan. There are many parallels between these older and younger artists: if in the teacher`s landscapes, as the protagonist presents the Byurakan Observatory, which was located next to the studio, then in Slavik Paronyan`s case as a companion of his creative life, the "main actor" of his paintings stands Saghmosavank, founded by Vachutyan princes in the 13th century, this magnificent medieval building, next to which he lived. Of course, H. Zardaryan, as well as a number of fine prominent representatives (M. Saryan, V. Surenyants, E. Tadevosyan, P. Terlemezyan, H. Kojoyan) have also created a lot of landscapes, where they praised the Armenian nature, but in some of them only we can see the image of the church. The core of Slavik Paronyan`s creative legacy, the major part of it composes paintings with spiritual depicting architectural structures of 1970-1990s. Surprisingly, in 1970s the painter was established in Saghmosavan, giving the opportunity to his eyes about half a century every day, every hour to admire the indescribable nature of the rocks of the Qasakh River, namely, the size, monumentality and influential position of Sagmosavank towering above this rock and so harmonious with the surrounding nature. How can artist with aesthetic values pass quite unexpectedly and can`t notice this unreal, cosmic, no earthly creation. This undoubtedly caused the large number of his paintings representing Saghmosavan from different aspects, in different weather conditions, at different times of the day. Artists Artashes Abrahamyan, Hakob Hakobyan, Henrik Mamyan and others, also depict the monastery, but we definitely agree with Knarik Avetisyan`s qualification, that Slavik Paronyan created "clean" landscapes, where in one aerial perspective, through the smooth transitioned rotation of bright and saturated colors, through the contrast of lively color specks created with one spontaneous stroke of brush have embodied the artist's own personal approach [3].

The first image of Saghmosavank relates to 1970. We speak about landscape "Our yard" where away of the foreground of trees we can see the domes of the church. With their presence they are as aaegis, as give unique blessing and promises fruitful year to the trees. In some places impressionistic and even puantelistical solution this planer etude, built with small brush touching, is overcome in "Saghmosavank", done already six years ago, where the low aspect gives the picture monumental sound and rigorous monumentalism. Unwillingly we remind Hovhannes Zardaryan's landscape "Amberd" (1971), where again it is selected low point, and the figure on the rocks of the mountain is perceived as national spirit [4].

Fig. 1 Saghmosavank, 1976

A number of paintings especially concerning to 1980s represent impressions, received from Paronyan`s Saghmosavan, in two of which identical "Saghmosavank"-s (1980, 1983) are presented in the winter landscape background.

Fig. 2 Saghmosavank, 1980s

In these paintings monotonous snow white and sky cold colors are so close to each other that they almost merge, yielding its predominant role in the picture, the church, with warmer shades. The other two landscapes "Mountain Ara in autumn" (1987) and "Autumn in Saghmosavan" (1985), the colors of which are conditioned by choice of weather, is completely dissolved with hot autumn vibrant, bright colors, represent the same scene. In other three landscapes, concerning to the same decade: "Saghmosavank" (1981), "Qasakh Valley" (1984), "Aragats" (1988), the artist painted the church from different perspectives, especially underlining the study of deep valley, steep rocks, though later they`ll serve for the Master as a material for finished thematic paintings. We think that for Slavik Paronyan any landscape of this genre is

unique thematic work, as each of them is not just seen and painted, but lived and passed through sensual inner world, where there were many things to say, "actors", rational thought compositional system.

"Autumn" (1998), created in 1990s depicts the same scene, as above mentioned landscapes "Ara mountain in autumn" and "Autumn in Saghmosavan" but here in the foreground are included, it seems, animated pair of trees as if in dialogue.

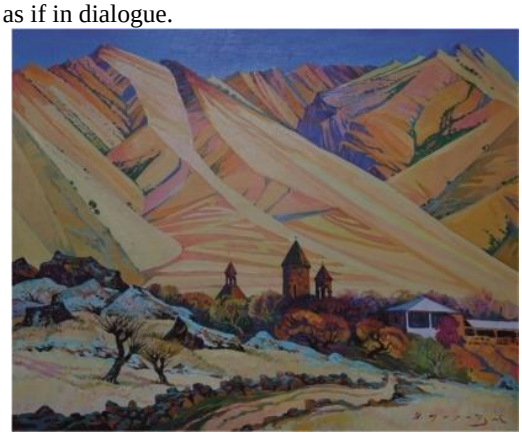

Fig. 3 Autumn, 1998

Paronyan with broad generalizations, bold, confident touching of the brush in the broad sense has "built" large volume of Mountain Ara. In 1994 in forefront of "Saghmosavank", unlike the same landscapes of the previous decade depicting the same scene, as an "actor" there is an Armenian cross-stone, which show the growing interest of artist towards the medieval historical monuments. The paintings "Summer in Saghmosavan" (1996), as well as "Saghmosavank" and "Qasakh valley", created in the previous decade, are stood out with panoramic resolution, wide spatial range and development master plans, in the most remote corners of which is seen the silhouette of the monastery, and in the upper parts, it seems, as a guard, stands the Biblical Mt. Ararat. "March" is the unique landscape, concerning to 2000s, where we meet image of Saghmosavank (2002).

Of course, alongside with Slavik Paronyan`s landscapes, depicting spiritual architectural structures of Saghmosavank, we can also mention two images of Haghartsin Monastery (1965), churches of Crimea St. Hripsime (1981), Yeghegnadzor St. Zorats (1976), Hovhannavank (1970), Ayrivanq (1970). In etudes depicting Haghartsin and Hovhannavank, is noticeable that the young artist is somewhat rude in the drawing, planned structure is broken, but already in images of Eghegnadzor and Crimea his creative style has been completely furbished and established. Especially noteworthy "Crimea St. Hripsime Church" (1981), where it seems that each line and brush strokeis as accurate as possible, for a moment not forgetting the fact, that he paints church, constructed on the basis of Armenian national historical art founder - V. Surenyants sketches.

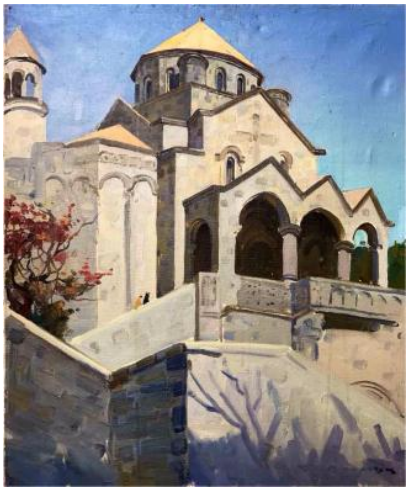

Fig. 4 Crimea St. Hripsime Church, 1981

Over the years in landscapes of Slavik Paronyan is further thickened the smell of Armenian soil, water, sun vital force, which once again proves, that the person who does not bears nature within himself, does not adore it and is indifferent to it, actually does not live a full life.

3. Conclusion

Thus, in the most of Slavik Paronyan`s landscapes are depicted number of Armenian historical and cultural monuments, showing the country's collective character. Most often we see the figure of Saghmosavank. These finished landscapes done in ally prime way, evoke various moods of sadness and grief on the one hand, and the patriotism and pride, admiration for the noble feelings of centuries past on the other hand.

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THE ROLE OF ARCHITECTURAL TERMS AND THE PROBLEMS OF THEIR APPLICATION

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Keywords: Architectural terms, architecturalconceptions, use of architectural terminology.

Abstract

This article has reflected the origin of architectural terminologies and the problems of their use. Architectural terminologies are considered the conceptions which are peculiar to the country by means of which independent peculiar national architecture developed and which have mainly national nature. The architectural conceptions which can be observed as terminologies are necessary to articulate without translation with the same possible transcription.

1. Introduction

In different historic epochs and social systems historians and architects have continuously studied, collected and created the history of architecture. Those constructions and elements which have had great importance in formation and development of architectural styles and directions have deserved special attention. The proper interpretation of those conceptions has created great number of architectural terminologies in architecture.

Such ancient European and Asian states as Egypt, Greece, Byzantium, China etc. are especially notable for the following terminologies: abak, antablement, antefix, agora, acropolis, atrium, barroco, geghatun, darbazi, zhamatun, zikurat, kreml and so on. These terminologies are numerous and being highly valued. They can be ranked as architectural founding ones. Many new modern terminologies such as keson, acustica, ampir, antresol are also arisen in the science culture and the city building throughout the course of development of new technologies.

Fig.1 GAVIT (from the righ) and Master Plan of Haghpat Monestry

Having an idea how numerous those terminologies are the significance of their proper use and presentation of the translation becomes clear.

2. Discussion

While studying architectural peculiarities of every country we have to do with such constructions and elements and constructive solutions which are peculiar to that country by means of which independent, destructive national architecture is developed. The fact that architecture is shaped and characterized by the dictation of secular and spiritual life of the given nation founds that historic terminologies have had mainly national nature from the very beginning.

Such names of architectural conceptions are necessary to observe as terminologies and articulate without translation with the same possible transcription.

With this approach when architectural conceptions are viewed as terminologies we create possibility to perceive architectural elements with all the contents meaning and with the exact artistic and architectural description.

While translating terminologies wholly or partially they lose their meaning, artistic value and peculiarities. Often, while translating same names of the constructions and elements which have exceptional meaning they are liken to the constructions and elements having the same kind of meaning and which with architectural, aesthetic and constructive solution, often by their meaning are strongly differed from each other.

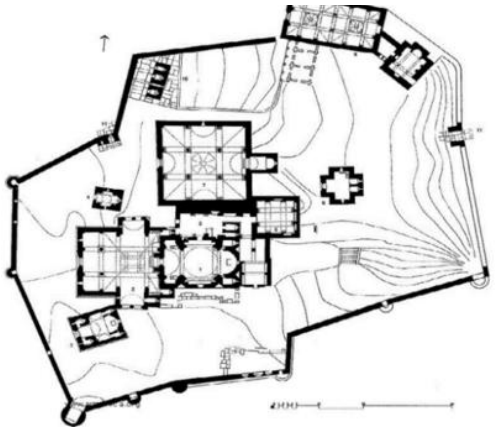


Fig. 2 GAVIT (from the righ) and Master Plan of Haghpat Monestry

Fig.3 GAVIT of Hovanavank Katghike Church

Being devoted to this principle let us analyze the following construction which is very specific and distinctive in Armenian architecture:
Gavit – (Fig.1-4) is now translated into Russian: 1 двор, 2 паперть, притвор: into English: courtyard, parvis. and into Latin: nartex, making similar to Russian, European modern constructions, Greek and Roman Cathedrals' thresholds (doorsteps).

According to historic facts the prototypes of gavit served to Armenian Apostolic Church ritual demands as halls for confessions repentances [1]. The most ancient (5^{th} century) gavits are : Tekor, Yereruyk, Bash Aparan, Odzun, Ashtarak and other church gavits. They were mainly built in the western parts of the churches.

Fig.5 Papert of Feodorovsk church in Yaroslavst Suzdal

But in the 7^{th} century accompanying with the weakening of the repentance halls came to an end. Gavits appeared with bigger demands in the 11 – 14^{th} centuries. Gavits are more square rectangles vaulted column halls [2]. No matter when gavits were built [3, 4], they have never been built together with churches and while constructing them they were joined to the façade of the existed church. Very often, without paying attention, those decorations, ornaments and records have covered columns, niche and other architectural elements. On the whole gavits have excelled in their sizes the church, sometimes making some disproportion (asymmetry). One more detail: this indicates the strong necessity for constructing gavits and also for their exceptionally secular significance. Meeting burial processions, educational and scientific events have been held there. We consider that sizes of gavits are depended on the number of the population of the community. Gavit is translated into Russian as притвор, паперть (Fig.5, 6).

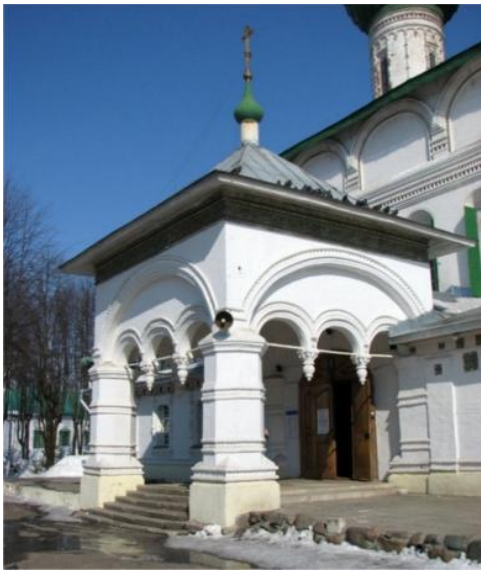


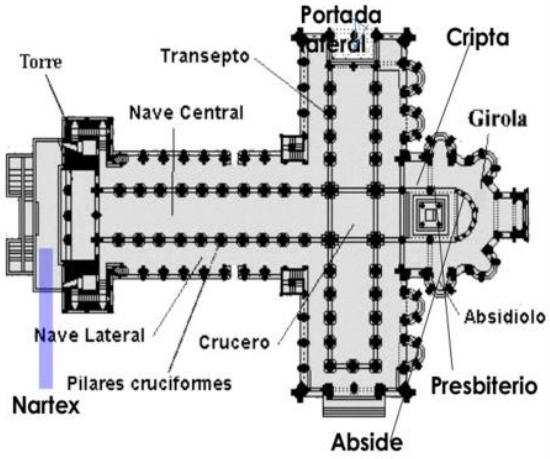
Fig.6 Papert of Rasplozhenskoy church in

Притвор – is a threshold (doorstep) built in Russian Cathedrals in 15 – 17st centuries and the open square in front of the doorstep was called паперть [5]. Притвор. Served as mere threshold was built simultaneously with the church with sizes comparative ph-s).

Gavit is translated into English as courtyard, parvis.

Courtyard is an inside yard surrounded by dwelling constructions (look at com. Ph-s).

Parvis – 1. Having special significance open platform or square in front of the cathedral. 2. Square, surrounded with buildings and constructions. Gavit is translated into Latin as nartex.



Nartex – in Roman architecture is a doorstep, built in western side of the cathedrals, provided for those people who have no right to enter the cathedral (Fig.7)

As you noticed gavit's foreign equivalents are поперть, притвор, courtyard, parvis, nartex and they express doorstep conception. But having some unification they are strongly different in historic and architectural point of view. These translations can't completely express the above mentioned basic meaning, architectural and artistic peculiarities of Gavit.

3. Results

With this approach when architectural conceptions are viewed as terminologies and they are articulated without translation with the same transcription we give an opportunity to concept architectural elements, the whole contents, significance, artistic and architectural elements with exact description.

4. Conclusion

Given example is a bare analysis of a term. It can be applied to the formation of other new architectural terms classification and use of which will give possibilities to have a wider idea about right conclusions and make revelations in architectural construction as well as in adjacent sphere.

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Fig.7 Narteqs in German Romanian tample

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THE CONCEPT OF SYSTEMS WITH THE MULTITIERED CIRCULAR ORGANIZATION OF FUNCTIONAL FLOWS

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Keywords: concept, modeling of rational systems, non-intersecting functional flows, multitiered circular organization, spiral and sloped roofs, inventions.

Abstract

The problem of modeling rational systems is analyzed.

The aspiration in architecture to perfect forms (circle, square, cube) is emphasized which is a reflection of the universal principle of least action, that any process or phenomenon in animate and inanimate nature tends to the minimum necessary energy expenditure.

Attention is drawn to systems with multitiered circular organization of non-intersecting functional flows (human and/or technical).

Conceptual solutions at the level of inventions with spiral-shaped sloped roofs in their formative basis are provided, the application of which in practice will provide a multilateral economic effect with the creation of significant architectural works.

1.Introduction

Throught the history whether through the use of logic or intuition architects always preferred simple geometric shapes such as the circle or the square in planning as they are the most capacious and cost-effective. The shape with the lowest perimeter in relation to its area is the circle, and the square is the winner in this category when we compare it to other rectangles

In this work, the proposed fundamental conceptual form-building solutions are disclosed that open room for the creation of new structures with an aggregate final effect (economic and architectural).

The mentioned concept with circular organization of functional flows can be seen in such iconic buildings such as the Guggenheim Museum, New York (F. Wright, 1959) (Fig. 1) [8], the Marina City complex, Chicago (B. Goldberg, 1964) (Fig. 2) [9], the City Hall, London (N. Foster, 2002) (Fig. 3) [10] and many more.

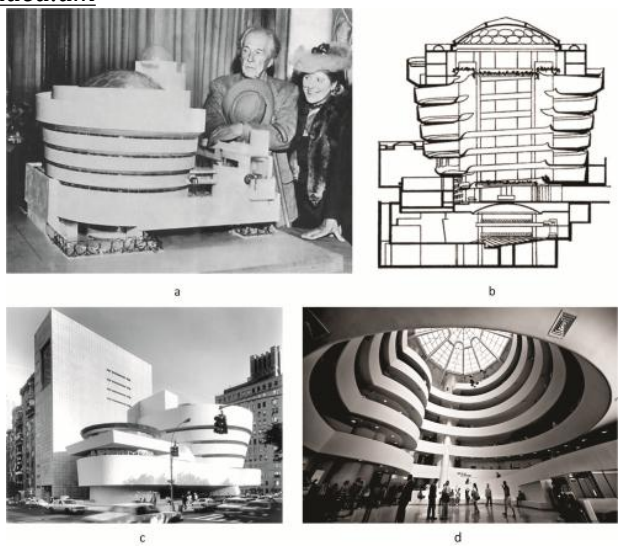

Fig.1 Guggenheim Museum, New York (F. Wright, 1959)

a) F. Wright uses the model toexplain the composition of the museum
b) Section
c) General view of the museum
d) Interior

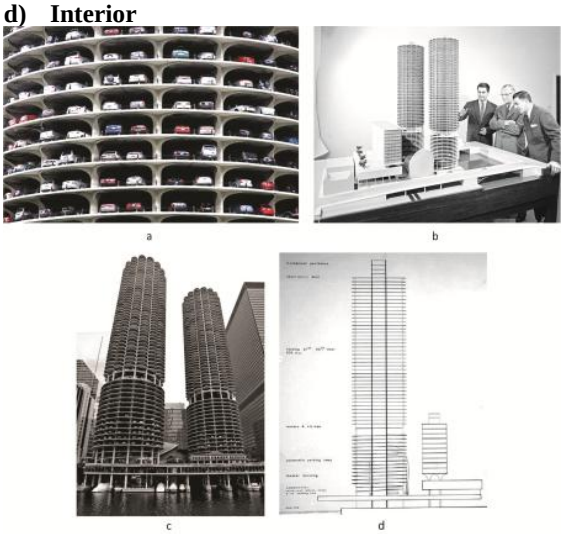

Fig.2 the Marina City complex, Chicago (B. Goldberg, 1964)

a) Garages of the complex
b) B. Goldberg explains the concept of the complex on the model
c) General view of the complex
d) Section

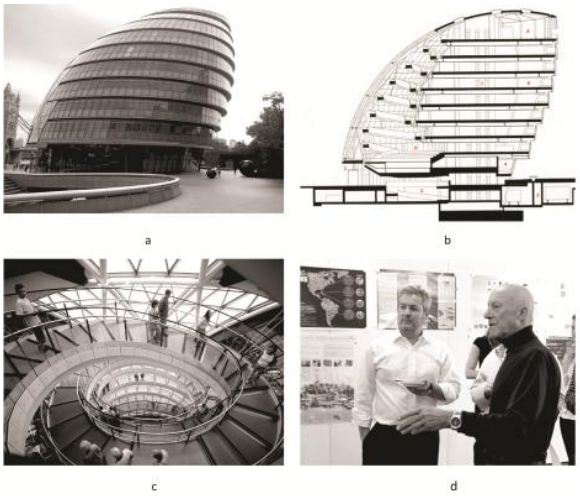

Fig.3 **City Hall, London (N. Foster, 2002)**

- ***a) General view of the city hall***
- ***b) Section***
- ***c) Interior***
- ***d) N. Foster performing a master class***

2.Methodology and Results

The research was done using the proposed methodological scheme (Fig. 4) as a guide.

The theoretical aspect of the organization of the inner structure of project systems was given serious attention starting from the 1950s [1…7] and the rise in the level of development of monolithic and prefabricated construction technologies that allowed to realize the entire range of available architectural decisions in practice played a major role in that.

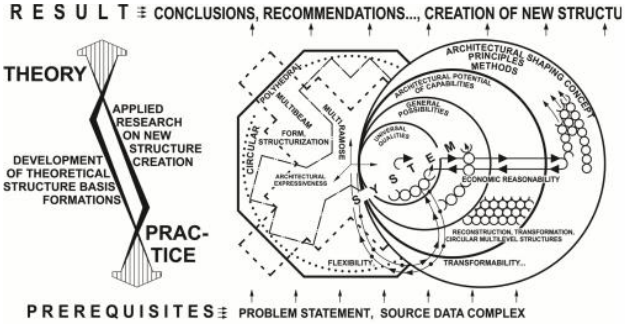

Fig.4 **Research methodology scheme**

With this in mind it is neccessary to mention that economy in architecture is often related to the type, the method of realization and the efficiency of usage of the connecting and mobile components of buildings. Questions like this always need to be solved when projecting objects with distinct functional flows that act as the glue that makes the entire object function.

In 1973 a project was developed in Soviet Armenia of a 245 place parking garage at the "VPEKTI" institute based on various research [1,2,4] and a patent (AC USSR N 514081) that featured 2 functional intersecting flows with the possibility to connect to a flow from any layer which was provided by the connecting horizontal bridges on every floor. The flows that were directed upwards and downwards acted as spiral ceilings and the entrance and exit for every flow was provided independent from each other (Fig. 5).

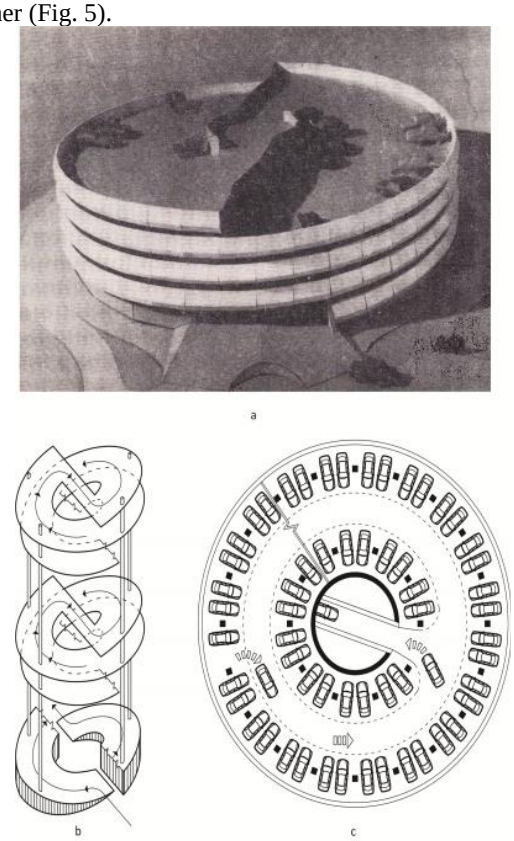

Fig.5 Project of a Parking Garage for 245 automobiles (Yu. Safaryan, R. Ohanyan), 1973

- ***a) Model***
- ***b) Functional scheme of the traffic flow***
- ***c) Floor Plan***

One of the main criterias of increasing the efficiency of a technological flow is its usage (inclusion in the functioning of a structure) on the biggest possible area of its traffic. The solution to this problem is ensuring the continuity of the flow while excluding intersections with other flows. The continuity is achieved in structures with "continuous" ceilings that are either based on the usage of inclined ceilings or spiral ceilings and their conjuction with horizontal ones.

The research [1,2,4], allowed to model completely new structural solutions with simultaneous independent functioning of 2 and more functional intersecting flows with the possibility to move between them. The best way to fulfill this criteria is by using structures with spiral (AC USSR NN 499392, 514081(fig. 4), 838076) and inclined (AC USSR NN 958634, 927336) slabs. These monolithic ceilings also have the advantage of giving the building hardness and sustainability thanks to their continuous structure across the entire flow. The wide functional capabilities of volumetric and spatial solutions also provide the combination of inclined and horizontal ceilings. In addition to the inclined spiral ceilings curved surface profile ceilings can be used too (AC

USSR applications 382245133 and 3885298/33). It's neccessary to mention that the deciding role in creating the composition in these structures lies on the ceilings.

These ceilings can have any closed curve shape in the plan which allows to adjust their composition based on the location and nearby architecture. Since you can enter these structures from any level they are really convenient to use underground since the entrance and the exit can easily be situated on the ground level, which is timely considering the unprecedented growth of cities and the following lack of area which the underground structures can be used to compensate for especially when used as a garage. These underground structures can also be used for greenhouses and orangeries.

The variability in shapes of the discussed structures and the huge variety of their compositions leaves room for new functional decisions. The variability is even greater when we consider the fact that they can also be combined with classical vertical structures with horizontal ceilings.

3. Conclusion

The multitiered circular functional organization concept (Fig.6) revealed at the level of inventions [1…6] allowed to figure out its main principles:

1. Centricity–the structure/building/complex tends to be more efficient when its plan is more centric.
2. Density – the plan of the building should try to have the largest possible area relative to its perimeter.
3. Capacity – the structure in the plan should try to have the biggest possible amount of functional planning cells while having the shortest connections between the cells.
4. Symmetry – the plan structure should be formed to be as symmetrical around its geometric center as possible.
5. Unlimited urban planning mobility – the typological structure should ensure the organic adaptions of its shape in the biggest possible range of urban planning situations

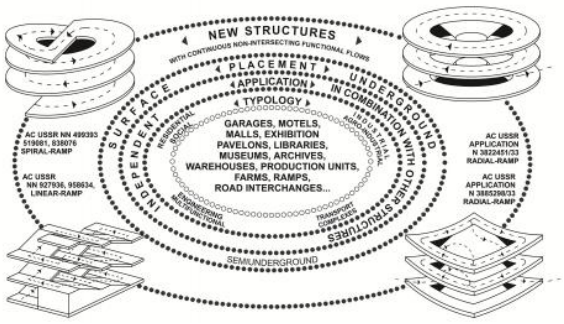

Fig.6 Concept of multitiered structures with circular organization of continuous non-intersecting functional (human, technological) flows

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TRENDS OF DEVELOPMENT OF RESIDENTIAL BUILDINGS IN ARMENIA IN THE NEW SOCIAL AND ECONOMIC FORMATION

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Keywords: dwelling, classification, new buildings.

Abstract

This is the first article to suggest the principles and creation of the classification of dwelling houses for the Republic of Armenia on prospect in modern conditions.

The article raises the issue of the development and construction of dwelling of new formation, its architectural embodiment, and the problem of providing each family with well-maintained dwelling is also being considered, which is always the most important problem of the country, the state, society. The article identifies and analyzes the classification of new-built residential buildings in the cities switched to the new formation. It is noted that in the early years (since1991) switch to a new socio - economic formation the multistory block of flats occupied a major niche in dwelling construction in the post-Soviet republics. It was stressed that the new socio-political formation and the market economy, led to a stratification of society and the sharp differences in the welfare of the population, and the redistribution of property and commercial dominance soon led to the appearance of its own elite, homegrown businessmen and businessmen.
The current state of the quality of residential structures are generalized, in cities built in the post-Soviet period, a number of serious problems are noted such as poorly managed urbanization, demographic changes in the population, social destabilization and etc.. The article noted that the imbalance has created social discrimination, which is directly affected as well in the architectural environment.
The article reveals the problem of the formation of the modern real estate market. It presents the trends and experiences of the last 25 years of both Armenia and other former Soviet Republics on addressing dwelling issues aimed at eliminating the issues of lack of dwelling for poor population.
An important part of the article is devoted to the emergence of new trends in the construction industry, aimed at meeting the demand of private companies and as a result stimulating
the development of the sectors of business and commercial real estate in the former republics of the USSR.

1. Introduction

The residential environment is a complex, continuously evolving system that has a direct link to the physical and social space of the city. However, the high cost of urban land raises the question of the need to improve the efficiency of its use [1]. The new social and economic conditions of cities are characterized by the lack of integrated development, the income inequality of the urban population and the inadequacy of the service system, unfortunately, do not contribute to the development of the residential ambiance, thereby pushing people responsible for urban development to find new promising ways to improve dwelling units. That is why, first of all, it is necessary to consider the residential ambiance and the living space, not just as a part of the city, but as a living environment, formed by many factors, namely natural and climatic, social, urban-planning, technical and economic, etc..

2. Methodology

In confirmation of the above, we note that in numerous studies (research) of foreign and local authors, taking into account the fact that today dwelling construction is one of the most dynamically developing segments of the real estate market, the relevance and historical inexhaustibility of the dwelling problem is noted [2-4].
Today, the problem of development and construction of dwelling, the architectural embodiment of the latter, along with the task of providing each family with a comfortable home dwelling, remains the most important problem of the society, the country and the state.
This paper studies and analyzes the classifications of new residential buildings in cities switched to a new social and economic formation.
In the first years of transition to a new socio-economic formation (namely, since 1991), multi-storey residential buildings in the post-Soviet states occupied the main niche in dwelling construction. The new socio-political formation and market economy led to the stratification of society and a sharp inequality in the well-being of the population, while the property redistribution and the commercial dominant soon led to the emergence of its own elite and homegrown businessmen.
Analyzing the current state of the quality of residential structures, namely the cities of the post-Soviet period, it is

necessary to identify a number of serious problems such as straggling urbanization, a low level of the legislative base of the real estate market, a decline in investment activity, the socio-economic consequences of destabilization processes, and so on.

In the last 25 years studying the experience of the construction of residential complexes both in Armenia and other republics of the former USSR, it is necessary to note that the most promising forms of organizing the residential environment of cities prevailing in the construction sector include the erection of multi-storey dwelling block being built both in the redeveloped and new residential areas. It was meant that this trend in construction most harmoniously reflects the need of a modern man in the diversity and significance of the urban environment, satisfying his need for dwelling, work and leisure. It was assumed that the design, the subsequent construction and their integration into the urban living environment will allow to solve many urban-planning, architectural and construction and socio-economic, ecological problems in the territories of a new construction and existing buildings undergoing reconstruction [5].

However, along with the previously discussed space-planning and design features of the dwelling, it is important to emphasize that at present the classification characteristics of multi-storey residential buildings are formed related to its value [6].

In dwelling policy the transition from distributive to market practice in a mixed economy, namely, the dependence of value on the forms of ownership of real estate, put forward the need for the formation of classifiers of dwelling in the post-Soviet space.

So, in the Republic of Belarus the following five types of dwelling have developed: "Economy", "Standard", "Comfort", "Prestige" and "Lux". As for Ukraine, we note that in 2006 the Ukrainian Construction Association (UCA) developed and adopted the current classification of dwelling: social, economy, business, premium, de lux. However, on February 17, 2015, "UKRAINIAN BUILDING COMMUNITY" and consulting company "City Development Solutions" held a roundtable "Formation of common standards for a new classification in the residential real estate market" and in April 2015 a renewed Classifier was presented, taking into account the round table participants' comments. The new Classifier provided two main segments: budget and prestigious dwelling, which included 5 classes: economy, comfort, business, premium and deluxe. When forming the classifier among the main cut-off criteria, experts indicated the location of the house, the level of comfort, technical characteristics, and so on [7].

Considering the classifier operating in Kazakhstan, we note that residential buildings in the republic are classified into 4 large subgroups, numbered from I to IV - all of them correspond to the usual elite, business, comfort and economy classes. Analyzing the dwelling market in Armenia, it should be noted that the above-mentioned divisions of the primary dwelling market into classes are relatively arbitrary, since unfortunately, as such, there is no classifier of dwelling, each real estate developer assigns a class, namely economy, business and elite. It should be borne in mind that the division of dwelling into classes is a feature that is inherent in one or another type of dwelling, which in different countries have different criteria for justification.

Every year, the classification of the market is becoming increasingly important and relevant, certain characteristics are developing in each segment with improved classification parameters. It seems that this message should work for the benefit of the real estate market, as some experts believe that some construction companies "overestimate" the class of the residential complex, in order to raise prices for apartments in it.

Exploring a fairly large dwelling market in the Russian Federation, first of all we note the existence of a uniform (united) classification of multi-apartment residential buildings discussed and approved by the Russian Guild of Realtors (RGR) on development and consulting in St. Petersburg on 14 December 2012 and former testing in 17 regions of the Russian Federation.

The above-mentioned document was supported by the Federal Fund for Promoting Dwelling Construction. We should emphasize that for the first time the methodology of consumer classification of multi-apartment residential buildings under construction was developed in 2002. With its use, the criteria for the classification of new buildings in Moscow were determined. Following upon this issue, we cite a number of factors that determine the class of dwelling in Moscow. It is important to understand according to what criteria this or that object has been assigned to a specific class of dwelling.

Let's consider an "economy" class dwelling. Usually this kind of dwelling assumes typical multi-apartment buildings of different number of storeys - from 5 to 12. In such houses there are usually more than a hundred apartments of different size and layout. "Economy" class

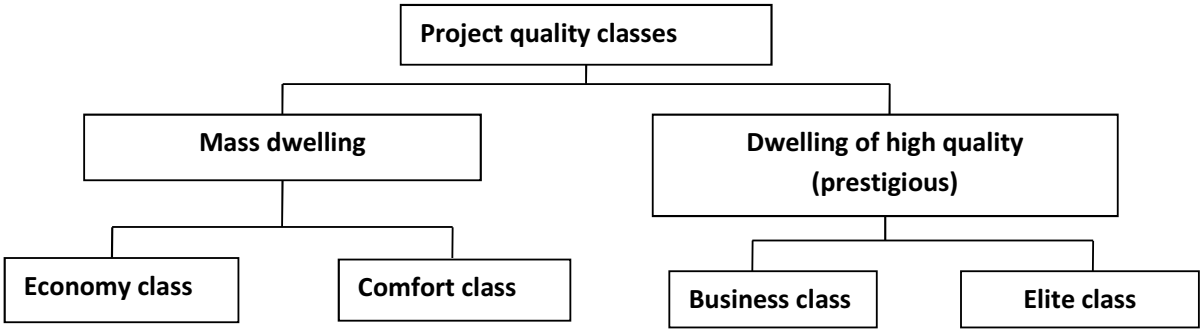


Fig.1 Classification of dwelling

houses are panel or brick buildings in residential complexes, where the project does not provide for underground parking, but an adjacent territory equipped for parking cars on the street. The height of the ceiling in such apartments varies between 2.5 and 2.7 meters.

The redevelopment of such dwelling is limited due to the nature of the project. The area of apartments in houses of economy class has the following standards: one-room apartments vary between 30-44 square meters, two-room apartments - within 44-70 square meters, three-room apartments- 60-90 square meters, and four-room apartments in such houses are very rare. "Comfort" class dwelling are panel, brick and monolithic frame houses. Projects of these houses provide for mandatory presence of parking spaces, and not less than a third of one parking space for each apartment. Ceilings in apartments of this class are high - from 2.7 m.. The area of the apartment is the following: one-room - within 38-50 square meters, two-room - 50-80 square meters, three-room - in the range of 70-110 square meters, four-room - from 90 to 150 square meters.

Dwelling complexes of business class are usually located in the central areas, where many objects of administrative and commercial importance are situated, or the most environmentally friendly areas of the city, so to say, closer to nature. Usually it is an apartment monolithic-frame or brick building erected on individual projects. Usually the number of apartments in such houses does not exceed 120. Such residential complexes are built in the most picturesque places, therefore buying a house in such a house, it is quite possible to count on a beautiful view from the window. The ceilings are much higher than envisaged dwelling projects of "economy" and "comfort" classes, from 2.8 meters. The area of the apartment is also different: one-bedroom - from 44 square meters to 60, two-room - from 60 square meters to 100, three-room varies in area from 100 to 200 square meters, four-room - from 130 square meters to 240. To buy such dwelling can afford people with high income.

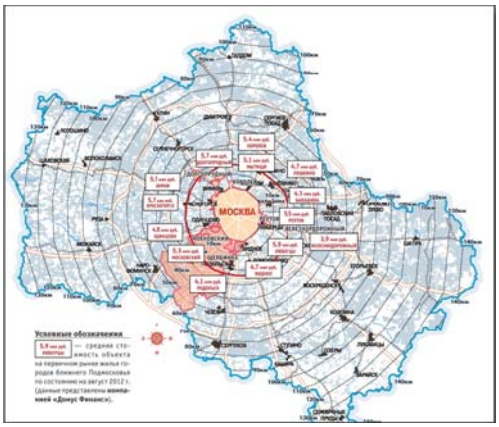
Residential complexes of "elite" class are always built in comfortable areas: closer to the center, in historic or most green areas, and sometimes near business class houses. Such arrays are usually made up of brick apartment buildings, executed according to an exclusive design with the use of advanced construction technologies and high-quality finishing materials. The number of such houses is not regulated; it depends on the project. For the convenience of those living in the house, usually not more than 50 apartments, including penthouses, are provided.

It is also important to note that the classification developed for Moscow was subsequently used by regional real estate market analysts when creating their own classifications that take into account the regional specifics of the objects and the market. In 2010, Sternik S. G., with the participation of the director of the Analytical Center "Kamskaya Dolina -consulting" » Yepishina E. D., as well as certified analysts of the Russian Guild of Realtors) and experts of the committees for consulting and development, RGR began work on a single classifier of multi-apartment residential buildings. The development of the classification of residential buildings of the Russian Federation for consumer quality was based on strict uniform application of the Dwelling Code of the Russian Federation, the Town Planning Code of the Russian Federation, the current technical standards (TS), GOST (State standard) and sets of regulations for the design and construction of dwelling, as well as orders of the Ministry of Regional Development of the Russian Federation [8].

3. Results

Thus, it should be noted that many experts agree with the need for classification, since orderliness facilitates understanding of the market situation and serves as a reliable indicator of the civilization of the market. Nevertheless, practice shows that it is much easier to agree on common terms with reference to the "star" designation of hotels or letter grouping of office premises than to determine the classification of residential real estate. Each developer, relating his house to a particular category (elite, luxury, business, premium class, etc.), proceeds from his own understanding of the quality characteristics of the object, the vision of the target groups and their needs. Therefore, the expert community in the field of dwelling construction strives to develop a universal classification.

The buyer will choose the dwelling that meets his criteria



Map of Moscow with pricing dwelling zones

Economy

Residential complex "Alekseevo"

Comfort

Residential complex "Albatross"

Business class

Residential complex "White Park"

«Premium»

Residential complex "Aquamarine"

Residential Complex "Barkli Gallery"

Fig. 2. Examples of classification of dwelling

At the same time, the developer from his own considerations is interested in individual classification. for convenience

and comfort. And most likely this subjective assessment will not coincide with those definitions of classes that experts suggest and try to operate. One of the factors hindering the introduction of universal classification is the so-called territorial heterogeneity of the real estate market. Unlike stylistic megacities in the regions, this sphere develops more slowly, according to its economic and social laws. The dwelling needs of a particular class (and hence the understanding of classes) in the center and on the periphery are different. As a rule, they are determined by the economy of the region, the subsistence level and the general standard of living of the population. As a result, local structures of the market are formed locally with their division of dwelling into categories.

4. Conclusion

The absence of uniform parameters raises serious problems. So how can the capital builder who decided to come up with the project for the region, describe the required category of the building when issuing the assignment of the regional project organization? After all, everyone has their own ideas about the class. Summarizing the above, we should note that for the creation of a classifier of dwelling in Armenia, it is necessary to develop concepts, to develop the most common standards and criteria for classifying dwelling. However, this is a necessary but not sufficient condition, since the most important goal is the introduction of the classifier in the practice of dwelling construction, which will allow differentiating the price policy in the residential real estate market, and as a result, provide dwelling for the broad layers of population.
This classifier will be a document that will be a guide for designers, city planners, design organizations, builders, developers, customers, realtors and banking structures. The adoption of the classifier at the level of state construction regulations is the creation of facility certificate for each house under construction, in which information will not be given only about the developer, the customer, the contractor organization, but also the class of dwelling to be built and the reliability of this information. Adherence to all the standards of the classifier will allow the specialized dwelling classification commissions to estimate the objects and assign a class to them [8].

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ARCHITECTURAL-PLANNING ISSUES OF ORGANIZATION OF OPEN SPACES IN HISTORICAL CITY OF GYUMRI

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Keywords: public open spaces, heritage, cultural identity, reserve-museum.

Abstract

The article based on the complex analyses of open spaces: way of its organization, level of its physical preservation, its modern modifications and act. in a sample of historical core of Gyumri city. The research is mostly based on the comparison of nowadays "quality and quantity" of public open spaces and with its historical situation. The value of open spaces occurring in an urban situation has many aspects like their form, use or location. The targets for sustainable development applied to the development of open spaces; which is done in two different ways. Firstly, a sustainable use of district for the current generation is required and secondly, aresponsibility towards future generations must be taken into account. The analyses show that nowadays unfortunately the architectural-planning structure that was created in the previous period has partly changed, due to development of city life. In these circumstances the modifications are also visible in organizations of public open spaces, which planning have to deal intensively with the question of how much open space network has to be public and publicly available to allow for a sustainable and community oriented urban development. Urban renewal and revitalization as planning process has to deal with manifold interests and touches societal and architectural problems in consideration of historical development, and always being in close connection with separate monumental buildings situated in that appropriate area.

1. Introduction

The introduction will focus on the specification of public open spaces, complex analyses of its historical development and nowadays requirement, the significant role of each one in sustainable development of urban environment in historical core of Gyumri city.

Gyumri is the capital city of Shirak Province, the second largest city in Armenia which history dates back to the II century BC [1]. Gyumri city considered to be "historical" not only due to long term of its historic, but also due to the urban, architectural and artistic aspects, such as valuable monument, ensembles, municipal complexes, urban and natural landscape, cultural areas that has also archeological values, act [2]

From the urban points of view city environments classified as: urban areas that are totally open to the natural and climate influences, without fences walls and covers, such as streets, squares, residential courtiers, parks, aquaparks, act and close areas such as buildings and structures, so the open spaces are the main guarantee of life quality and sustainable environmental conditions of cities.

In the borders of the article the main understanding which is going to be observed is urban open spaces in historical core of Gyumri city, Kumayri Reserve-museum. Kumayri historic district: is the old part of Gyumri with its unique architectural style. It has more than a thousand buildings dating back to the 18th and 19th centuries. The district is one of few places in the Republic of Armenia, with authentic urban Armenian architecture. Almost all the structures of the Kumayri district have survived two major earthquakes, in 1926 and 1988. The historic district of Kumayri occupies the central part of modern-day Gyumri. Although suffering severe damage during the disastrous earthquake in December 1988, Gyumri is still preserving its own architectural characteristics. And nowadays the interest to the historical city increases due to truism development, the Gyumri city has 1023 monuments (27 points) [3].

For understanding of open spaces in landscape and natural engineering there is not the exact term, it's called "fragment of the urban environment", "Urban areas", "City landscape", "Urban interior", Urban ensemble", act.. Each of this terms defines the phenomenon from their point of view, but three of them are most popular: "open space", "Urban environment", "Urban interior", because this ones most precisely define the "environmental" essence of the object. So in each city, as well as in Gyumri sense of architectural open apaces are securing the-streets, alleys and parks, stadiums and squares, where human creativity set borders in the "emptiness", so anything that visually limits the viewer, whether it is walls of buildings or rows of trees. All this things allow us to understand the essence of the concept "Urban open Space" or "Urban interior". The difference urban interior from usual one is the combination of elements that creates by the architect such us fragments of the landscape, horizon, street prospects, panorama, and act. It is clean that the main task of architect is to isolate, to limit the certain part of the space by the material forms.[4]

Historically public spaces considered to be cities centres, squares, surroundings of spiritual-cultural structures (nowadays protection zone of monumental buildings). This kind of areas used to be compositional centre of whole urban environment.

This sort of areas was the place of public meetings, have role of cultural and social activities. Also the street, pavement, parks, aqua parks, sport complexes which were open to public use

Fig.1 Considered to be public open spaces, Spandaryan street, Gyumri city

In the borders of article public open spaces will be divided into two groups:

- public open spaces which are open to public use without any convention such as streets, pavement, public parks, squares, pedestrian zones of monuments, open moles and act.
- Semi-public which are open to the public use but with precondition (need to pay for the entrance) such as sport complexes, zoo parks, aqua parks and act.

The city's "Urban open spaces" mostly are created by buildings and structures and especially when we deal with historical cities, these terms automatically becomes into if not monumental buildings, the significant ones for the appropriate city environment. With this as a background, the main research questions of the article are outlined.

2. Methodolog

The methodology of article is mainly based on observation of contemporary tendencies of public open space development, complex analyses and typological classification of its historical development in Kumayri reserve-museum of Gyumri historical city.

During historical phases the borders of city as well as the amount of open spaces are evaluated according to the needs of time.

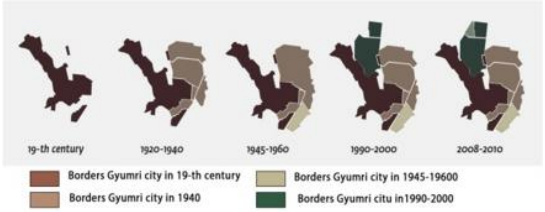

Fig. 2 Development of Gyumri city

Enhancement of urban landscapes and public open spaces is part of the urban culture of the city. Wide squares and several pedestrian streets, realised during the last years of the Soviet Union, make Gyumri a pleasant town to walk around. The use of public art and sculptures is something which might enhance the cultural urban experience of the town.

The further analyses of development of public open spaces in Gyumri city will be observed in a contest of cities various phases of its development process in 19-20 centuries. (figure2)

Parallel to the city boarders expand the surface of public open spaces get rise but at the same time stands out whit the poor condition of the existing public open spaces and buildings in the "Kumayri" museum-reserve area, as well as in all city including a large number of uninhabited structures and structures belonging to the vulnerable population. Problems regarding street, pedestrian roads and pavements in the historic district, landfills, lack of sidewalks and no integrated solutions.

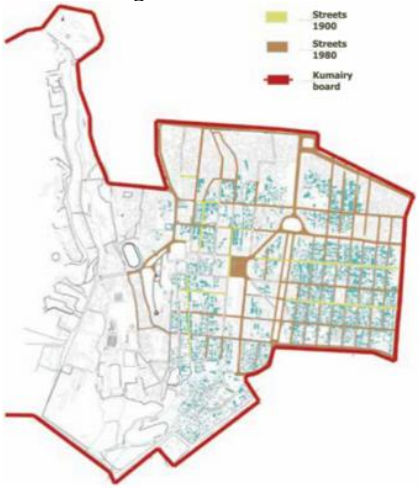

Fig. 3 Development of public, street net spaces in 1980-90

Fig. 4 Existence of street sheath

From urban-planning point of view street nets in a historical center of the Gyumri city are kept as the figure 3 shows, not depend on in which stage of preservation are nowadays. The same situation is in publicly available green areas, parks pedestrian paths, and squares, from architectural side its shapeare preserved, but is in a bad state of conservation.

This sort of problems closely connected with the social-economic situation of all country, but never means that simple citizen doesn't have his own aim. The purpose of feasible study is:

Fig. 5 Public green spaces actual situation

- Proposing a practical plan for city revival
- To analyze Gyumri's main attractions, then assessing and proposing their potential as tourist destinations.
- To identify urban projects that require immediate attention from local and international stakeholder[5].
- To set a new standard in the region for preservation and reuse open spaces and proper practices for renewal and construction.
- Realize social programs in a field of public awareness of heritage valuation and preservation.

3. Results

Du to city life changes and nowadays population needs the architectural-planning structure created previous centre in an active process of development that leads various modifications in public open spaces, which planning has to intensively deal with the question of how much open space needs to be publicly available. In the other hand contemporary tendencies shows that nowadays increase the nterest of valuation historical environments, that's leads to preservation and reuse of not only public open spaces, but also monumental structures, because they are closely relates to each other. At the same time the social-economic situation of the country not allows totally restore and keep historical significant monuments and open spaces, and properties often with their activities ruin not only the monuments but also the all image of public spaces.

In the borders of article revealed actual situation and its architectural planning developments features of Gyumri city. The main phases of its Planning- volumetric development are outlined.

4. Conclusion

The organization of urban environment which based on orderly transformation and progress of social needs, preservation of historical and cultural heritage within the architectural and planning organization of urban open spaces must be realized with this steps:

- The existence of a tourism sector, especially inserting an information center and other infrastructures, as well as the development of projects and initiatives emanating from it.
- Preserve and renovate historical houses that are in poor condition by making them usable for living and for businesses, that also will improve also the condition of open public spaces[7].
- Provide a sustainable source of income for project beneficiaries.
- Create successful business stories to become a model and be replicated by other Kumayri residents.
- Bring back the vision identify & secure funding.
- Improving of the mechanism realizing and controlling urban planning documents of Gyumri city.
- Formation of united planning connections of historical zones of Gyumri city's historical core.
- Existence of vision for the revitalization of historic districts, which could be realized by a complete reconstruction of one block or one street, making their consequences more visible.

4. Acknowledgements

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EXAMPLE OF ARCHITECTURAL PROJECT OF REHABILITATION OF STONE-DOOR CAVES AND ITS USE FOR TOURISM PURPOSES

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Keywords: Stone door caves, tourist routes, protection of history and culture monuments, local authorities (local self-governance bodies), alleviation of social burden.

Abstract:

Stone door anthropogenic caves are spread throughout the entire area of Mount Aragats and in Shirak province, in addition to a few more examples found in the north-eastern part of Gegharkunik province. In Mount Aragats these caves are dug at 1,000m to 2,000m altitude, in volcanic-sedimentary and volcanic-contact rocks, under magma flows.

The project of using these anthropogenic underground structures for tourism purposes seeks to achieve several goals:

1. Protection of material values of history and culture;
2. Making traditional tourist routes more attractive;
3. Demonstration of medieval and antique daily living and life style for research and educational purposes;
4. Creation of jobs for local population, reduction of social burden, and promotion of the role and reputation of local authorities.

1. Introduction

Caves are natural cavities or empty spaces in the thickness of geological layers, which occur as a result of earth-forming processes or weathering. To meet his needs and to improve the the quality of his lifestyle, in addition to natural caves, man has created underground dwellings, storages, basements, mills, hiding places and even tombs, churches and monasteries. From the professional point of view, both natural and anthropogenic cavities are called caves. Natural caves are classified according to the peculiarities of their origin [1], while the anthropogenic ones are classified according to their functions or social and religious significance [2, 3].

There are some unique examples of anthropogenic caves on the southern and southwestern slopes of Mount Aragats, at the altitude of 1000m - 1700m. These caves are unique, with the heavy doors made of stone, tunnels/passages, and casks for liquid and dry food with 6-9 cubic meter capacity. Stone doors in the medieval structures are often found both in the Armenian Highland and in Europe; however, they are different by their form and are not a constructive part of the architectural composition. The passage/tunnels leading to the caves with stone doors were masked, and the doors are placed along the entire passageway, at certain distances from each other. According to the legend, some caves had up to seven doors, for example, the Seven Door Cave in Kosh, where three of the doors have survived [4].

For those who study the Armenian history and the history of the Indo-European peoples, these caves reveal such manifestations of social relationships and lifestyle that are not possible to restore from other archaeological sources. The caves give the complete picture of the medieval everyday life of people [5].

From the scientific-cognitive and historical point of view, protection of these caves is of critical importance. For this reason, it is necessary to restore and reconstruct several samples of these caves having historic value as monuments and being on the verge of extinction and include them into the tourist routes. The only way to save the stone door caves from final demolition is to use them [8].

2. Results

Monuments of culture, history and nature are subject to be used but in no case exploited [6]. This does not mean the monuments will not generate incomes. We are speaking about preservation of the original state of the monument. If the original state of a monument is distorted, then, based on reliable scientific information and documentary material, it is necessary to restore the monument and make it suited from the scientific-cognitive perspective and from the perspective of meeting the aesthetic requirements of the society.

Monuments of history and culture will never be complete without availability of the social and natural environment as it was during the period of their formation, and in some cases they can become meaningless at all. Therefore, the monument should be restored together with its historic environment.

Stone-door caves in Aghdsk. Stone-door caves are rare rock-cut structures that have survived only in Aragatsotn, Shirak and Gegharkunik marzes (provinces) of Armenia. They consist of three main parts: a cave-hall or halls, a stone door and a secret entrance which is a unique artificial tunnel, barely passable and very well masked. The used part of the cave is a natural cavern, which was formed in the contact point of the magma flow and sedimentary rock layer as a consequence of decomposition and weathering of sedimentary rocks. Normally, their sizes are not large, with a surface of 40m2-45m2 on average; however, there are also caves with many rooms and a large hall. The secret entrances

leading to the cave are rather long and narrow; up to 60m long passages can be found. The tunnels were built in the following way: a trench was dug, with its bottom reaching the parent rock, then from two sides the walls were laid using ragged, misshapen pieces of stone, and the walls were covered with 20-40 cm thick long slabs. The last phase was covering this whole structure with rocks. Regular stone-door caves used to be mainly hiding places for small groups of people or for food for community, where the inviolable supplies were kept. Only a few reliable people had access to these caves [4].

Stone doors are unique both in terms of their form, the selection of the material and from the functional point of view. Stone doors are made of basalt, with a size of up to 1.6m high, 1m wide, and 18-30cm thick.

The technology of construction and improvement of the cave is as follows: the cave has been enlarged, dug and brought to a normal state. In some cases, not far (5cm-10cm) from the rock walls, a parallel wall was built with misshapen pieces of basalt stone and fortified with mortar. Then the cask-like granaries were dug and laid. Then a door with its frame was placed. If the entrance was large, then walls were laid on both sides of the door. Then the construction of tunnel began. Slabs of regular shape were laid parallel to each other and covered with similar slabs. If the tunnel was too long, then a second door was placed in the middle of the tunnel. Another door was placed in the end of the tunnel. Stone doors typically open inward. The last phase was masking the structure. It was covered with pieces of basalt so that to play into the environment both by the colour and by the shape and appearance of the rocks.

The stone doors have bulges that are rounded at the top and have a hemisphere shape. The assembled stone beam, serving as a frame, is from monolithic basalt plates; in the upper and lower pieces of the plate, holes are carved to enchase the balges of the door. Opposite to the axis of rotation of the stone door, there are furrows from outside to enchase wooden handles.

The stone door cave in Aghdzk (Fig. 1) is located in the north-eastern part of the village, in the right side of Amberd Canyon [7]. The entrance opens at the foot of a precipice rock wall, at a height of 30m from the riverbed. It is a large cave with tree halls and three doors (Fig. 2), which were working up until 1987, while in 2004 only the small door of the tunnel/secret passage was working.

The entrance is wide enough to pass bended. Passing the first stone door, you can see on your right the 2-meter survived part of another collapsed tunnel. Proceeding further you reach the second stone door, wherefrom a 2-meter sector is completely built. In the western sector of the first hall, there are three cask-like pits on two artificial platforms. On the first platform, at 1 meter above the bottom of the cave, the mouth of a cask opens, 1.8m deep and 2m in diameter (Fig. 3). On the second platform, which is 1.8m above the bottom of the cave, there are two pits, 2m and 2.2m and 1.8m and 2.1m in diameter, respectively. In May 2004 the platforms were destroyed and the pits were damaged. The 2nd and the 3rd masked halls suffered a greater destruction. The small stone door leading to there was still working, however 5 of the 6 pits were damaged considerably, with the 6th one still in good shape.

Fig. 1. Stone-door caves in Aghdsk

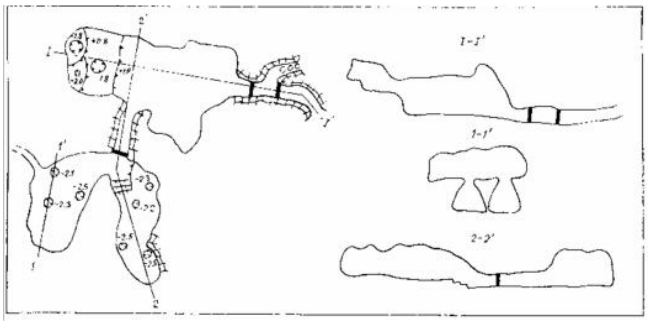

Fig. 2. The plan of Stone-door cave in Aghdsk

Fig. 3. The pit of cave

3. Conclusion

The priority of restoration of stone-door caves is explained by two factors:

- According to the results of monitoring by the Armenian Speleological Centre, stone-door caves are continuously and consistently destroyed by black archaeologists" - treasure hunters. These processes have been accelerated over the last five years.
- Stone-door caves are not simply storage or hiding places but a wonderful means to identify medieval ethnocultural relationships; therefore, it is necessary to simultaneously rehabilitate the medieval grape varieties and terrace orchards, the mill, the oil-mill, and the traditional Armenian glkhatun ("main house") with its "Hazarashen" dome.

The project of rehabilitation of stone-door caves and the medieval ethnocultural environment. Using the reports of the speleological expedition of the Speleological Society of the Arm. SSR Academy of Sciences, archives of the ASC, video films shot during the monitoring and the monitoring reports as well as other sources, implement archaeological investigations (as far as the cultural layer has been completely destroyed, excavations become useless) and create an architectural project in line with the initial appearance.

Sequence of the envisaged processes:

1. Geodetic extraction of terrain.
2. Speleological research: Description of the cave and the environment, peculiarities of rocks.
3. Geology and geomorphology of the terrain.
4. Environmental impact assessment: There is no need for EIA since stone-door caves can be equaled with the storages, while EIA for the rest of the structures of the complex will be implemented by the Chair of Geoecology of NUACA.
5. Archaeological works.
6. Architecture of caves and the architectural drawings of rehabilitation.
7. Terrace grape growing, establishment of orchard following medieval grape growing principles.
8. Study of the materials in the location of the cave and its tectonics.
9. Issues of safety and seismic stability.

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FORMING OF CONTEMPORARY SOCIAL HOUSING ESTATES – CHOSEN ISSUES

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Keywords: social housing, architecture, historical examples solutions.

Abstract

Social housing for poverty classes in its genesis is not recognising by the history of European architecture.

In European conditions, the idea of the creation of settlements / communities for the poor was the teaching of Christ and the social, cultural and civilizational revolution that was initiated by the European Christianity. Since the Byzantine era, ideas and practices concerning the settlements for the poor have matured. They developed in the modern period, especially during the period of capitalism.Not until the period of modernism the concepts of social housing appeared. They are taken up in various aspects of sociological design and phenomena. Nevertheless, so far, there has been no focus on the so called social housing for the poorest issue.

The author presents selected issues related to issues of social housing for the poorer social classes.In the time of globalization, such solutions should take into account the paradigm of sustainable development in relation to social housing, draw on available resources, local culture, meet the conditions for co-creating the artifacts of the mentioned culture in terms of active participation of people suffering from precarity in the process of building residential buildings, as well as fulfill eco-energy requirements (economic, ecological and energy).

These issues also seem to affect the social housing of the Caucasus states (eg Georgia, Armenia), Eastern Europe (including Poland), and so-called emerging economies (China, India, etc.), where various

1. Introduction

The article contains the results of the author's work on selected issues related to the genesis and development of the concept of housing estates, which have been formed since the Middle Ages in the European area. The article presents historical examples as well as the relationship between historical and contemporary tendencies of social housing. It is important to point out that issues particularly relevant to the concept of a European settlement are, in principle, not fully recognized by the history of architecture. The authors' efforts concern the presentation of these materials with reference to perspectives of the development of the concept of a social settlement.

In-depth assessments of the current social housing situation in the context of the development of civilization and European cultural heritage are the starting point for seeking solutions to social housing in Europe and the possibility of projection for the future.

2. Conception of social housing estates

A synthesis of the successive stages of the formation of the concept of social housing in the Middle Ages, the Renaissance, the pre-industrial and industrial periods is presented below. The period of modernism as well as modern tendencies in social construction in Europe are briefly discussed.

2.1. Historical studies

Historical studies allow to distinguish some ideas presenting the lines of development of social housing in Europe. These include the first centuries of Christianity up to the twentieth century. The analysis

processes (political, social, economic, etc.) were involved.

The article presents materials illustrating contemporary examples of solutions to social housing, particularly in relation to historical

The beginnings of the idea of building dwellings for the poor have their origin in the communities of religious character. These communities are the precursory social structures of programmatic poverty, that developed functional and spatial solutions for chosen way of living. The communities, especially in the first centuries of the Christianity in Eastern Europe can be seen as the archetypes of housing for the poor [2].

Milan's edict (313 AD) issued by Constantine the Great closed the period of persecution of Christians. This has led to the dynamic development of the monastic community, taking activity in monasteries and monasteries. The rise of the Christian Byzantine Empire has also brought together the Greco-Roman heritage, including the social movement. It had a remarkable cultural and civilization value by dynamizing the help for those in need. During the Byzantine period priests and nuns were the only representatives of the intelligentsia in the contemporary society. [12]

The Eastern European homesteads (the first centuries of Christianity) can therefore be understood as archetypes of social housing for the poor. Their reflection was charity activity led by monks in hospitals, orphanages, and schools. It also included the catering for the poor and hospice for travelers and pilgrims, along with medical care for the sick and lepers.

Basileias (gr. Basileas , 372 A.D.) that owed its existence to the initiative of St. Basil the Great (330-379 A.D.) in Caesarea in Cappadocia is the example of such community. It was a complex including care facilities such as hospitals, orphanages and nursing homes [1].

However the idea of cheap settlements seems to be the side-effect of the development of monasticism and the idea of Eastern spiritualism especially in the of this period allows to present the synthesis of the phenomenon of social communities.

medical books were stored and preserved. These were the beginnings of the first period of medieval medicine, the so-called. Monastic medicine-also called monk. One of its symbols was the Monastery of Monte Cassino (529 AD, founded by St. Benedict).

The duty, especially the Benedictine Order, was to care for the sick, the poor and the pilgrims. Shelters, hospitals, hospices, hospitals, infirmeriae were set up at the monasteries, where the needy received bodily and spiritual help. The Rule of the Benedictine Order proclaimed, "infirmorum cura ante omnia adhibenda est" ("care must be taken first of all"). In medieval times, health treatments were done out of mercy, not for reward.

Around the monasteries were established gardens providing essential herbs. A great role was attached to herbariums. The so-called. Monastic medicine was officially completed at the Synod of Clermont (1095 A.D.). The monks were forbidden to do medical work, and the medicine went into lay hands [12].

This has in some way changed the paradigm of caring for the poor.

In any case, the social space in the Middle Ages refers to the principle of "the love of neighbour," which was the basis of the doctrine of Christianity. The "Long Middle Ages" was the base period from which other forms of social housing grew.

Fig. 1 Cappadocia, Christian settlements from the period of early Middle Ages, phot. J. Halbiniak

early Middle Ages in the Eastern and Western Roman Empire [2]

The most spectacular example of the volcanic cities is Goreme Valley in Anatolia (fig. 1), in the 7th century A.D. inhabitable by 30.000 inhabitants forming monastic community [3].

The functioning of the monasteries of the East and the West is therefore referring to Basilia and is intended for habitation / or residence for the poor.

In monastic libraries, the contents of ancient

was built in 1521 A.D. on the initiative of Jacob Fugger (1459-1525) called The Rich (fig.2,3),[4].

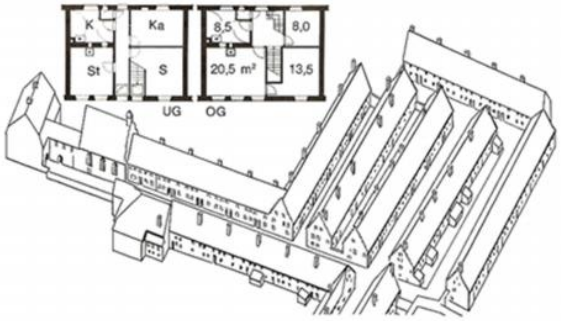

Fig. 2. Fuggerei, Augsburg, 1516-1525, Housing axonometry and projections of flats. Investor: Jacob Fugger [5]

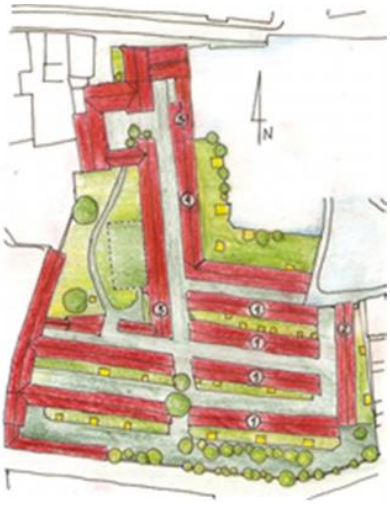

Fig. 3. Augsburg- Fuggerei. Estate plan., drawing: K. Bak [2.4]

The Fugger Foundation has allocated housing for the impoverished Catholic families who have fallen into

2.2 Housing estates in Augsburg-Fuggerei (1521 A.D.)

The oldest settlement for the poor founded according to the Christian axiology seems to be the housing estate in Augsburg-Fuggerei, which

first time in history, the elements of modern management of the settlement structure and its financing have been linked together. These conditions created a new quality in shaping the social housing of modern times:

- emergence of precursors - modern banks of global financial- financial firms (Medici-Florence, Fuggers - Augsburg) acting as an investor,

- introduction of a modern administration / management system in relation to the new-created estate

- the use of Christian axiology in creating a new quality structure.

It should be stressed that the estate is still in use today. It is an extremely important example of the development of the housing concept, which has become a reference for later residential structures.

2.3. Development of social housing in Europe throughout the century

The residential or social building began to develop again in the 18th century, especially during the First Industrial Revolution in the industrialized countries of Europe.
The process of technological, economic, social and cultural change was linked to a transition to an economy based mainly on manufacturing engineering.
The idea of social housing in Europe can be divided into the following periods: end of Middle Ages (church split -Reformation, 16th century), enlightenment (1688-1789), French Revolution (1789-1799 - "freedom, equality, fraternity"). Changes in social consciousness caused by Enlightenment / French Revolution have become the basis for the concept of utopian socialism [6,9]. This was reflected in the direct ideas of Utopian housing estates of the nineteenth century, as well as attempts at their implementation in Europe undergoing the

poverty.

The settlement in Augsburg consisted of 56 buildings. Each of them has been equipped with 2 separate apartments with a separate entrance and a garden. This solution provided independence and personal freedom [5].

The amount of rent fixed by the Fuggers - 1 Gulden (equal to a worker's weekly wage) is valid until today.

In addition, fees for water consumption, energy, waste disposal and cable TV are charged. Phone charges, central heating and electricity are paid according to their consumption. There are three main conditions to be eligible to live in Fuggerei: need help, live in Augsburg and be a Roman Catholic [4]. It seems that in this period for the

solving the problem of social housing [10].

The importance of development of housing and social housing for the poor draws attention to the solution of the "mass" social housing. These are related to the period of European modernization in the twentieth century. Selected examples such as Le Corbusier's realization - Pessac housing estate (Bordeaux, 1925) is an example of social housing - a duplicated artifact, to a certain extent in the history of 20th century social housing [11].

During the development of the modernist movement, represented by such creators as Walter Gropius or Peter Behrens, construction techniques have had a great impact on form / architecture. Hence, during the Bauhaus's operation, the housing architecture was developed rapidly. Preference has emerged for prefabrication technology, which, consequently, led to the creation of blockades in the long horizon.

This type of construction was criticized, especially since the 1960s.

During the second half of the twentieth century there was some stagnation in the development of the concept of social housing and architecture of this kind. Despite the great construction movement characterizing this period in European cities, the new structural model of social housing has shown a crisis.

process of industrial revolution [2].

The industrial revolution launched in Great Britain triggered the emergence of new urban planning concepts. We can distinguish four main concepts: Ville sociale, Garden City, Garden suburbs, workers' housing [7]. They refer to the period of industrial revolution (2nd half of 18th century to World War II) [8].

These concepts point to more complex ways of

Fig.6 House from rammed ground. Domaine de la Terre estate. Phot. B. Kryjan [2]

Fig.7 Service and residential complex TBS in Katowice, Poland, (S.A.R.P competition-realisation 2004), phot. F. Szpak. [2]

Only the end of the twentieth century and the turn of the century -XX and XXI seems to have brought new developments: projects and relationships based on the new paradigm (fig. 7, 8).

The Domaine de la Terre (France) housing estate built in the 1980s and 1990s is undoubtedly the trend (fig. 5, 6).

Fig.5 House from rammed ground. Domaine de la Terre estate. Phot. B. Kryjan [2]

activities, the principle of participation of the inhabitants was present. It also concerned later design activities. This resulted in the achievement of appropriate quality of space, adapted to the individual needs of residents of the estate [13].

The concept of "sustainable development" housing estate included the postulate of designing a housing estate without individual cars. Regardless of this assumption it is possible to park cars on the edge of a housing estate in a collective, solar powered garage. Mass transit traffic provides tram traffic (fig. 8,9) [13].

The "sustainability" assumptions also include the use of green and natural building materials and economical heating technologies, solar energy technologies, etc. These solutions have provided

low energy consumption throughout the entire estate.

The estate has an exquisitely designed green area, which complements the sustainable solution of a contemporary housing estate. The housing estate also has a service area designed in pavilions between residential buildings, with a program of School,

Fig.8 Service and residential complex TBS in Katowice, Poland, (S.A.R.P competition-realisation 2004), phot. F. Szpak. [2]

2.*4. Social housing Vauban (Germany 1994- 2006)*

The Vauban housing estate, built between 1994 and 2006, is one of the leading centres for social housing in the European continent. The basic assumptions related to the project and realization of the estate concerned the issue of sustainable development, also in the context of the social aspect. Already at the beginning of planning

Fig.9 Vauban estate in Friborg, Germany [13].

kindergartens, shops, clinic, etc. [13].

It is worth emphasizing that the settlement is inhabited by the younger part of the population (the predominance of people up to age 45 is 76% of the population, 33% of children within).

On the other hand, the inhabitants are also seniors older than 60 years old (2.2%).

Residents of the settlement mostly represent the middle class, which gives up the use of an individual car for bicycles and trams.

The issue of social participation (foreseen in the process of establishing a settlement) is valid in practice, encompassing the various initiatives of the inhabitants forming the community.

The involvement of all parties involved in this innovative venture has made it possible to carry out this complex, highly demanding investor. The joint action of residents, municipal and national authorities of Baden-Württemberg, as well as financial support from the European Union in creating a new model of social housing was crowned with success [13].

Fig..8 Vauban estate in Friborg, Germany [13].

4. Conclusion

Economic globalization prompts us to rethink the consumer society that creates closed, elite housing estates.

Fig.10 Vauban estate in Friborg, Germany [13].

Fig.11 Vauban estate in Friborg, Germany [13].

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In this way we are losing opportunities for development of settlements in their social dimension.

The currentness of this issue in Europe proves the title of a conference organized in 2005 by the Warsaw architectural community, "From the social housing estate to the closed housing estate." This means that the neoliberal economic doctrine dictates and decides the shape and direction of today's preferential social settlements in the landscape of postmodern cities. It requires from

architects of solutions and projects of housing estates suitable for the spirit of time and changing needs. They are designed to provide residents and users with decent forms of living adequate to their present needs as well as the state of holdings and financial resources at their disposal [2].

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GEOMETRY OF SACRAL BUILDINGS

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Abstract

The analysis of architecture in the geometrical aspect is reduced to the separation of elements of the Euclidean space from the examined object. Geometric forms found in architecture can be divided into flat figures and spatial elements: solids and surfaces. Flat shapes and their assemblies are found in structures directly in the form of architectural details (eg. window and door openings) and indirectly on two-dimensional projections of objects (projections, sections, elevations). Solids and surfaces or parts of them appear as components of a building, which is always a composition of different spatial elements.
An analysis of the Christian church architecture, carried out for the use of a variety of geometric forms, is presented in the paper on the example of sacral objects situated in the archdiocese of Czestochowa.

1 Introduction

Geometric analysis of an object can be done in two main areas. The first one is the use of geometric figures and bodies for modeling 3D architectural objects and the mutual relations between them. The second area is the recording of space on a plane, in accordance with the principles of projective geometry (e.g. Monge's projections, axonometry, perspective). In both cases, the symbolic aspects of the spatial and flat elements can also be taken into consideration.

Architecture has been closely related to geometry since ancient times. In those times, special attention was paid to proportions and symmetry of buildings, on the basis of Euclidean geometry. The followers of Plato and Pythagoras believed in the existence of a deep relationship between God and the world through geometry[1]. Analyzing the historical development of sacral buildings from the point of geometry, one may notice a visible development of the forms used for shaping the body of buildings in the respective eras. This is affected not only by the changing esthetics, but also by the development of technical means, allowing to erect more and more complex structures.

Moreover, we may note that in the respective historical periods, there alternately appeared: love for simple forms, and love for more complex and richly decorated forms. The respective changes were introduced by negating the ideals of the previous era, which resulted in the application of innovative forms, and often led to borrowings from much older times.

This study will present the analysis of geometry of the sacral buildings erected after 1945 in the archdiocese of Czestochowa.

2 THE POST-WAR CHURCHES OF CZĘSTOCHOWA

Right after the Second World War, the sacral buildings in the area of the archdiocese of Czestochowa were constructed based on two different esthetic conventions. There were erected the churches of conservative character, referring to the stylistics of the previous eras, in the style called syncretism, and modernist buildings. An example building designed based on the first stylistic convention is the Exaltation of the Holy Cross Church (Fig. 1). It is a Neo-Renaissance building, designed in 1949 by engineer Sachse from Dąbrowa Górnicza. The geometry of the main body of the building is fairly simple – it's a cuboid covered with a gable roof. In terms of geometry, the most interesting elements are the volutes placed over the frontal and back walls. On the wall finished with stairs, there are placed spiraling stone elements. The detached cuboid belfry, erected in the fashion of an Italian campanella, is connected with the body of the church with cloisters. The openings to the cloisters are rectangular in shape, adjacent to semicircles. The same shape appears in the openings of the belfry at the bell level.

Modernist buildings, erected in the area of the archdiocese of Czestochowa after the Second World War, are represented by the Divine Providence Church (1948-1950) designed by Edward Usakiewicz (Fig. 2). Its body is angular in shape, its form is simple, without any decorations. The openwork church tower, made of reinforced concrete, is clearly dominating and is the most complicated element in the geometric sense. It is situated to the left of the body of the church, near the presbytery, and consists of cuboid posts, and, in fact, constitutes the symbol of the tower, without having any actual function [3].

In the 1960s and 1970s, the tendency that dominated the style of new sacral construction was modernism. A typical example of such a building from 1960s is the St. Stanislaus Church in Biała Górna (1966-1968), designed by M. Witek and S. Woch (Fig. 3). The church was erected based on rectangle, and is covered by a flat slab roof. The only decoration of the cuboid body of the temple is the benched side walls, which results in directing the side windows towards the alter, resulting in better lighting. To the side of the main entrance, there is situated a tower in the shape of a cuboid.

3. CHURCHES IN 1970s AND 1980s

In 1970s in Poland there was erected a relatively large number of sacral buildings with tent—shaped bodies. One of the first innovative examples in Poland is the St. Adalbert Church in Czestochowa (1975-1990), Designed by Antoni Mazur (Fig. 4). While designing that church, the architect used the concept of crossing prisms based on scalene triangles. The dominant feature of that monumental building is the openwork belfry integrated with the multi-body structure of the church. The upper parts of the prism are also openwork, which is used for lighting the church, with lattice structures in the windows [2].

A typical example of the modernism of 1970s is the Our Lady Patron Saint of Poland Church in Zawiercie (1974 – 2000) designed by Andrzej Zdzienicki (Fig.5). It is a building without any exterior sacral characteristics. It might be considered a cinema or theater building. Its function is reflected only in the church attached to the roof ridge in the top part of the building. According to the designer, the church is to resemble Noah's Arc. Around the building, there are situated roofs resembling waves, on which the Arc is standing. The arc is a prism based on a hexagon, whose lateral wall is resting on the cuboid part of the basement. The entrance area is heavily glazed and

Together with large parish churches, in the 1980s there were erected small sacral buildings – filial churches. They are situated on site of a parish, but are not the main parish churches. Priests serve in them on Sundays and church holidays. The functional program of such buildings is usually poorer than that of parish churches.

An example of that type of structures is the Our Lady Chapel of Ease in Załęcze Wielkie (1980-1982) (Fig.9). The design of that building has been lost, so the designer of church is anonymous. The geometric form of the body of that filial church is very simple: a cuboid covered with a gable roof. The side walls have elongated, narrow windows with vertical, convex frames. The color diversity of those rectangular convex frames and the fact that the windows end right above ground, make the walls seem very light. The openwork symbolic tower at the main entrance also intensifies that resembles the transparency of the waters of the deluge [4].

In the 1980s, there appeared postmodern designs of churches, and modernism started to be of conservative character. However, many modernist buildings were still being erected on site of the Czestochowa Archdiocese. There appeared several buildings that are interesting in terms of architecture, with the projection in the form of a circle or a fragment thereof. Aleksander Holas designed two such buildings in Czestochowa: the St. Albert Chmielowski Church (1981 – 1990), the Maksymilian Maria Kolbe Church (1990 – 2002) (Fig. 6), as well as the St. Barbara Church in Rudniki (1988 – 2000).

In turn, Marian Kruszyński designed the St. Maksymilian Maria Kolbe Church (1984 - 1999) in Kuleje (Pic. 7). The geometry of that building is interesting. It was designed on a projection similar to a circle. The building lines are set by two open curves constituting parts of displaced circles. The outside walls are leaning to the top, resulting in a curve at the roof level which bends under variable angles. The church constitutes a compact asymmetric body which, according to the author, resembles Our Lady's crown. A part of the wall adjacent to the presbytery projects outside the roof and ends in a tower crowned with a cross.

The horizontal projection of the St. Barbara Church in Rudniki (Fig.8) constitutes a fragment of a circle. The roof with a significant inclination angle, has a slope towards the short arched walls housing the main entrance. The walls limiting a segment of the circle have variable height and the shape of adjacent triangles. At the top of the roof, there is situated a cross. The compact body of the St. Barbara Church resembles a tent or a hut, supported pointwise in the highest part of the roof. Can be noticed references both to the Old (Ark of the Covenant) and New Testament (tents in the desert, test on Mount Tabor). It is a symbol of a place for meetings, a tent for prayer.

impression. You can see the geometry, in which the architect was interested, even though the forms used are very simple and angular.

An example of application of geometric diversity in a design, but without a good artistic effect, is the St. Joseph Church in Popów (1980-1987), designed by Zbigniew Bać (Fig.10). The projection is a triangle with rounded apexes, and with the vestibule situated in the oblique base of the triangle, between two parallel load-bearing walls. The most important element is the tower in the shape of triangular pyramid, at which there is situated a cylindrical wall which constitutes a part of the side altar. The roof is of changing height. The untypical tower, lower than the main body of the church, based on a circle, was erected on the eastern wall of the aisle. The artistic effect of that geometrical diversity is rather debatable.

Apart from the above-mentioned modernist buildings, since 1980s in the archdiocese of Czestochowa there have appeared postmodernist churches. One of the first ones was the St. Stanislaw Kostka Church (1986-1988) designed by Jerzy Kopyciak (Fig.11). It is situated on a small plot corner, among compact urban structures, and covers exactly the whole area of the plot, and its overhang over the main entrance is located above a street. The overhang is shaped as an elliptical cylinder, the bases of which are shaped as halves of ellipses. A fragment of the cylinder is separated with two vertical planes and situated right over the entrance. That interesting form makes the building look different from the neighboring cubic urban buildings.

Another example of postmodernism in sacral architecture is the St. Joseph Church in Radomsko, in the parish of the blessed Michael Kozal (1987-2000), designed by Zygmunt Fagas. The horizontal projection of the building constitutes overlapping circles and circle segments which may refer to the symbolism associated with the Holy Spirit. The spiraling church tower also emphasizes that symbolism. The compact body of the building clearly refers to organic forms [5].

4. CHURCHES ERECTED AFTER 1990

The second church designed by Domagała is the Sacred Body and Blood of Jesus Church in Czestochowa (1995-2000). There are visible the elements symbolizing give sets of priest's raised hands with bread. The geometry of that symbol is very simple – a symmetrical heptagon, in which the external lateral outline was thickened and painted in various colors. The simplicity of that solution is a huge asset of that building. The frontal façade of the church is not associated with the shape of its interiors. The wall which symbolizes priest's hands, protrudes beyond the outline of the lateral external walls. The motif of hands appears in four more places, including in the form of extensions of the pillars between the windows, on both sides of the lateral facades. The presbytery part of the building is cylindrical.

The same architect co-created another interesting postmodern building – the St. John Sarkander Church (2002 - 2004) in Czestochowa, designed jointly by Z. and W. Domagała. That building demonstrates both prismatic and cylindrical forms, harmoniously overlapping. That small, compact building presents a significant diversity of the summary of geometric forms. The effective prismatic portal turns into the church tower situated right above the entrance, even though the form of the tower is far from traditional. The only literal reference is the church towering above the whole building.

The Divine Mercy Church in Zawiercie (2001-2008), designed by Agnieszka Bednarska-Szkarłat, was erected in the neo-historical style (Fig.15). The basilica of traditional shape, with the quasi-transept of tie-in character, refers, with the horizontal projection of the building, to historical models, and is based on a Latin cross. The dome with a lantern, the round windows and rounded portals, are typical for old-Christian Latin architecture [4].

An example of a church that represents regionalism in sacral architecture is the St. John Kanty Church in Czestochowa (199—2000), designed by Maria Musialska. A steep multi-hipped roof, triangular dormer windows, arcades and triangular cappings of windows constitute references to regional architecture. The walls of the basement constitute a prism based on a non-symmetric complex polygon. The elements of the roof which constitute fragments of concave polyhedrons are interesting from the geometric point of view, but questionable from the esthetic point of view. They are visible in the lateral roof slopes and over the entrance, and one constitutes the roof of the openwork church tower.

5. Summary

The forms of contemporary churches, erected after the Second World War, are very diverse. One might say that "we've seen it all", which might result on the one hand in the need to look for the forms that have never been applied in sacral buildings yet, and on the other – in the application of well-known elements, in literal form, or by denying that literality.

In many contemporary sacral buildings has clearly departed from the traditional method of presenting the sacred. The building of a contemporary church is very often so different from the archetype of such a building, functioning in the collective consciousness, that sometimes only the existence of a cross allows to identify its function. An example is the Our Lady Patron Saint of Poland Church in Zawiercie. In the archdiocese of Czestochowa there are relatively few such buildings, and a more traditional approach prevails.

After the Second World War in Poland, the attitude of the authorities towards the Catholic Church was unfavorable, so few churches were erected. The sudden development of sacral construction occurred in 1980s and 1990s, as a result of the political transformation. It resulted in the same number of interesting buildings and quite unfortunate ones. Over that time, in the archdiocese of Czestochowa there were constructed over 200 parish churches and filial ones. This study only presents several of them, thus attempting to illustrate the existing stylistic and geometric diversity. The selected buildings include both very interesting and quite unsuccessful structures. Individual assessments may vary, but it seems that the conclusion which may be drawn from this review is that the application of too many geometric forms does not always lead to a desired artistic effect. This is consistent with the common saying that "less is more".

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Fig. 1. Church of the Exaltation of the Holy Cross in Czestochowa

Fig. 2. Church of Providence of God in Czestochowa

Fig. 3. Church of St. Stanislaw BM in Biala Górna

Fig.4. Church of St. Adalbert in Czestochowa

Fig.5. Church of Our Lady of the Queen of Poland in Zawiercie

Fig. 6. Church of St. Maximilian Maria Kolbe in Czestochowa

Fig. 7. Church of St. Maximilian Maria Kolbe
in Kuleje

Fig. 8. Church of St. Barbara in Rudniki

Fig. 9. Church of St. Mary of Czestochowa
in Zalecze Wielkie

Fig.10. Church of St. Joseph Worker in Popow

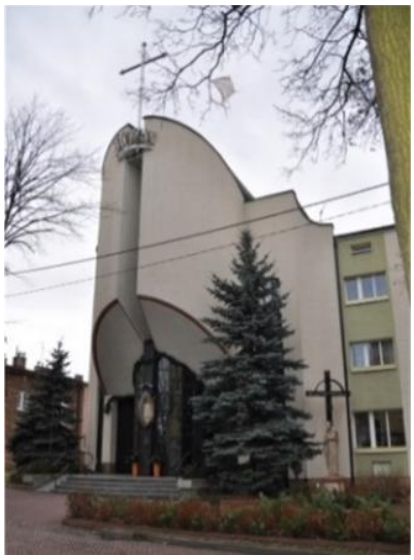

Fig. 11. Church of St. Stanislaus Kostka in Czestochowa

Fig. 12. Church of St. Joseph the Bridegroom of the Blessed Virgin Mary in the parish of Bl. Michal Kozal in Radomsko

Fig. 13. Church of the Most Holy Body and Blood of Christ in Czestochowa

Fig. 14. Church of St. John Sarkander in Czestochowa

Fig.15. Church of Divine Mercy in Zawiercie

Fig. 16. Church of St. John Kanty in Czestochowa

ILLUSTRATED CONSTRUCTION EXPLANATORY DICTIONARY AND ITS INFLUENCE ON THE DEVELOPMENT OF GEORGIAN CONSTRUCTION CULTURE

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Kakhaber Khmelidze (Independent researcher-designer)

Abstract: Construction is such a field of national economy, which unconditionally reflects the level of material well-being and culture of the country. In particular, construction entities preserved for humanity the history of society formation, its forward moving and harmonious development from ancient period until now.

The big construction ranges, objectives to create logistic support for the development of country economics puts in agenda undeniable growth of construction industry and qualification of human resources. The first goal for achieving this is to have contemporary high quality field guide, statutory and lexicographic literature on native language.

As a result of longstanding work, multivolume construction explanatory dictionary with etymology and colored illustrations was prepared at the construction department of Georgian Technical University (placed in internet). It includes app. 14200 terms and 2300 colored illustrations. Dictionary mainly comprises construction (structural, building materials, technology, architecture, underground buildings, roads, airports, construction documentation, hydrotechnics, water supply, military engineering and etc.) terms, as well as the terms connected with construction (geology, geodesy, metallurgy, economics, business and et. al.).

Big ranges of construction also requires preparation of qualified, expert stuff, improvement teaching quality and scientific directions, computerization, creation of Georgian construction norms and rules and et.al. Preparing and publishing construction explanatory dictionary in Georgian language is undoubtedly big national affair.

Introduction

Construction is the field of national economy that unconditionally describes country's material welfare and cultural level. From ancient period up to date there were construction buildings that retained humanity history of society formation, progress and harmonious development.

Big scopes of construction, objectives of country's economic development, creation of resource-and-technical base puts in agenda inevitable growth of construction industry and human resources qualification. The first place task in achieving this is to have contemporary, high quality field guide, normative and lexicographical literature in native language.

In construction segment of Georgia, the quantity of employed people is 20…25% of working population and that's is the reason of interest to have available contemporary electronic and printable literature, especially explanatory and encyclopedic.

Preference of explanatory and encyclopedic dictionary is that with the help of refined structure of the dictionary a reader delves into content well and gets information fully especially when the dictionary is enriched with visible materials and illustrations.

We must admit, that currently, in terms of basic compulsory dictionaries we are drastically behind of European level and this gap becomes irrecoverable step by step. We have not contemporary, competent always renewable dictionaries (printable or electronic) regarding Western and Eastern languages and it's especially hardest situation in publishing of technical explanatory dictionaries.

Basic Part

As a result of hard work on construction faculty of Georgian Technical University, construction explanatory dictionary was prepared with multivolume term etymology and colored illustrations (chairman of the editorial board and compiler prof. Tamaz Khmelidze, computer support Kakhaber Khmelidze, it's located in open access internet with the name "Construction Explanatory Dictionary"), which incorporates appr. 14200 terms and 2300 colored illustrations. Mainly it includes construction and building terms, like structures, building materials, technology, architecture, design, expertise, underground buildings, roads, airports,

construction documents, hydraulics water supply, heat supply, ventilation, construction equipments, mechanics, construction norms, military engineering, geodesy, metallurgy, energetic, geology, labor safety, emergency situations, economics, business, computer sciences and etc.

Necessity of creation of construction explanatory dictionary in Georgian language was conditioned by following factors:

- The most part of Georgian population studies English now and practically any explanatory literature can be found on that language;
- Big part of this literature is not available for ordinary users in internet;
- Georgian lexicography in that direction is poor, in reality there are no perfect explanatory dictionaries in that direction;
- To create construction dictionary makes user thoroughly study computer science and become more closed to technical intelligence and science spheres of developed countries;

The aim of explanatory dictionary (printed or online) is:

- To provide Georgian builders and others subject explanations and terms concerning construction in printed or electronic forms;
- To use contemporary electronic data bases for construction design and implementation;
- To create more or less perfect explanatory dictionary for users which would be reachable, handy, illustrated and easily understandable for readers.

tLTerm (Professional Termbase Editor) program was used during working on dictionary. This program is based on South African Republic company "TshwaneDJe" founders, on program provider developers and lexicographic experts - David Joffe and professor doctor Jill Morris de Shriver - terminology management computer program TLex Suite.

It will be possible to use online dictionary in any computer having internet which is equipped with convenient searching system that gives an opportunity perfectly display Georgian language web-sites and means of interaction with them.

In Georgian reality construction literature – explanatory guide, recommendations, articles and etc. is contaminated with calque from foreign languages (especially from Russian language), that disintegrates Georgian language with its world importance. An attempt has been made to eradicate this problem. As an illustration we have too much examples.

It must be noted that if translation of terms is necessary and not founded Georgian corresponding word, it's preferable then to use a word from original language from whence term comes from. With the effort of the authors this problem is maximally reduced. Most foreign term is attached by its etiology (word origin) which is created with the help of Latin, Greece, English, German, French, Spanish and other sources

selected and processed colored photos placed in text in most cases are taken by authors (Kakhaber Khmelidze), the photos by other Georgian and foreign photographs like Lasha Gebelia, Zurab Papiashvili, David Mikava, Kakha Mchedlidze, David Mirzashvili, Jaba Labidze, Gia Grigalashvili, Goga Chanadiri,Lazare Gagnidze, Greta Howard, Klaudio Margotini, Gabriel Silva, Arian Zvegers, Salvador Fornelli, Juan-Carlos Briseno, Angey Shandomirski and others are also used.

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The authors thank Academic of Georgian National Academy Archil Frangishvili - chancellor of Georgian Technical University, vice-chancellor prof. Zurab Gasitashvili, director of arn. Chikobava Linguistics Institute – Academic of Georgian National Academy Avtandil Arabuli, the head of department of science terminology and in translation dictionaries of the same institute prof. Lia Karosanidze, reviewers: Professors of Georgian Technical University Archil Moconelidze and Vaja Papaskiri, for supporting in the process of preparing the dictionary.

Conclusion

1. Construction explanatory dictionary will be the first completed, illustrated dictionary in Georgian Language, which will help effectively Georgian customers working in construction sphere – engineers, doctoral students, master candidates, bachelors, architectures, designers, scientists and experts, businessmen, investors, public registry, notaries, advocates, courts, prosecutors, ministries, municipal services of towns and regons, for foreigners who wants to study Georgian Technical language, journalists, building workers and etc.

2. Represented dictionary will unconditionally raise the level of Georgian engineers' construction culture, will give them solid, systematic and dynamic (generation) knowledge, will raise quality, and positively reflect country civilization development ways on the level of education and publication of such kind of work is really very big national affair in the history of Georgian construction science.

THE ROLE OF URBAN LOGISTICS IN THE DEVELOPMENT OF SOUTH GEORGIAN TOURISM AND RECREATION CLUSTER

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Abstract

Communications and logistics topic, that influences today all fields and directions, is of vital importance in the context of sustainable urban development.

Strategyc cross-border position of South Georgian Samtskhe-Javakheti region in the resettlement system of the country is expressedin the large number of roads with different ranks and destination. Region is crossed by historic roads, connecting West,Eastand South Georgia. Paper reviews the dependency of tourism and recreation cluster of Samtskhe-Javakheti region on the communication and logistics systems; the role of the region in the resettlement system of the countryand the State "Spatial Planning" Program.

Using Cluster Analysis methodology, relation between medical tourism development and communications - logistics systems was considered; cases of well known resorts of the region – Akhaltsikhe, Borjomi and Abastumani were presented. The importance of road communications and related communal pipelines, their role in the determination ofprospective development directions for resort settlements of Samtskhe-javakheti region were mentioned. Recommendations for Sustainable Urban development were suggested.

The novelty and originality of this research is the assessment of the impact of communications and urban logistics systems in well known Georgian resorts and potential resort places. The topic is actual on the background of the State "Spatial Planning" Program.

Keywords: Urban Logistics, Resort Development, Tourism and recreation cluster

1. Introduction

Among the four main directions of the sustainable development of ourcountry, which were announced by the Government of Georgia, in framework of "Freedom, Rapid development, Prosperity, the Government Program 2016-2020", [1], the "Spatial Planning" is presenting strategic vision in terms of the spatial development of the country [2]. In "spatial Planning", which is themain framework of the country development, Samtskhe-Javakheti region is crossed by three spatial structure schemes. Based on this it can be concluded that this region has one of the key roles in the spatial development of the country. On the basis of "Spatial Planning" plan, communication system of Samtskhe-Javakheti will be considered. The importance of the spatial planning and communication systems is reviewed in numerous scientific researches [3,4,5,6], as well as in the State and regional development strategies [7,8].

Using cluster analysis method, tourism and recreation cluster of Samtskhe-Javakheti is defined. For the development of tourism and recreation cluster in Samtskhe-Javakheti, that is the region with strong spa-industry in past, considering contemporary reality, therevival of medical tourism is necessary. Its formation is significantly influenced by the communication and urban logistics system.

For thousands of years the living environment was created by the human along communication axles and their intersections. From ancient times to the present day is obvious, how the road, the communication, affects development of the urban network. On the Georgian territory, on the crossroads between Europe and Asia, from ancient times the main trade roads were intersected.

The impact of the communications was discussed for the cases of three well known resort settlements of South Georgia – Akhaltsikhe, Borjomi and Abastumani. The topic is depending on the implementation of the State "Spatial Planning" Program. [1].

It is important to consider the communication system, as visionary factor towards to tourism development, which is the priority sector of the Regional Development strategy [7]. In the Regional Development Strategy, as well as in the Georgia Tourism Strategy [8], the prospects of the medical tourism developmentare not clearly mentioned. At the same time, more than 50 countries in the world have the development of the Medical tourism on the State policy level [9]. Because in Samtskhe-Javakheti, due to the multiplicity of protected areas and spa resorts, is

less possible to develop industrial fields, excluding agricultural products processing, and therefore, spa industry should be developed according to new conditions.

By the assessment of the communication structures impact on the tourism and recreation cluster and medical tourism development, it is important to consider spatial structure schemes related to Samtskhe-Javakheti region. [1,2]

During the study of communication system impact of Samtskhe-Javakheti, cluster analysis method wasapplied. Many works were dedicated to the clustering and interdisciplinary development [10, 11, 12]. Among many definitions of the cluster one of the definitions by M. Porter was used here: "Clusters are geographic concentrations of interconnected companies and institutions in a particular field." [10]. According to the opinion of some authors, "Creating clusters in the region should be considered as a best economic mechanism of the sustainable development of administrative territorial unit".[12]

Objectives of this paper are following provisions:

- In context of "Spatial Planning" plan of "Freedom, Rapid development, Prosperity, the Government Program 2016-2020", concretization on one of the regions and components, in particular, analysisthe impact of Samtskhe-Javakheti region communication system on the prospects of tourism and recreation cluster;
- Research of the problems of communication logistics, in order to strengthen regional spa and resort potential;
- Growth of the national and international importance of medical tourism in the tourism and recreation cluster of Samtskhe-Javakheti;
- Analyzethe influence of communications and logistics on the resort development, considering positive and negative influences of road systems, elaborating recommendations for the strategic development of the region.

The article aims to perform a supporting role in the creation and development of urban planning documentation for the region. To achieve this purpose, using cluster analysis methodology, the article sets out several tasks.

2. Methodology

Analyse of existing state territorial arrangement system, expert assessments in fields of medical tourism and spa industry; critical and comparative analysis, questionnaires survey, interviews, cartography data processing.

3. Results

The main results of the paper are: suggesting functional and planning decisions of the problems, defined through multilateral analysis of region's communication system, for the sustainable development of resort settlements. The map of the main communications of the region considering State Spatial Planning Program was included. [1],[13].

3.1 Discussion

The inportance of the communications is vital: if these connections are not regulated, urban unit can weaken and come to complete degradation. Development of the urban attraction centers is determined by city-forming and city-serving factors. For example, the main city-forming factor of the administrative center of the region, **Akhaltsikhe,** historically was the strengthening of the southern border area. In Saatabago(province, governed by Atabagi, head of the province in medieval Georgia).of Samtskhe were included not only Samtskhe and Javakheti, but also provinces, which are currently in the territory of Turkey. [14].

Akhaltsikheis climate and spa resort. The word "Akhaltsikhe" means "New fortress" in Georgian.[15],Inhabitants – 17,9 thousand [16].Its role is still very important for the strengthening of the border region. Akhaltsikhe is mentioned as a city since middle ages (10-12th century)[17].From 1930 Akhaltsikhe becomes a district center, from 2014 it gains a status of self-governed city [18].

During Soviet times Akhaltsikhe was a closed city, due to border location and military-strategyc disposition. Resort potential of the city could not develop under these circumstances. After removal of the border regime, the rate of economic growth in Akhaltsikhe is faster than generally in Samtskhe-Javakheti, what is conditioned by developed communications and logistics network, trade relations with neighboring Turkey and Armenia, the hub position.Its potential is defined by the analyse of the road system. Due to all these factors, Akhaltsikhe could coordinate together with Tbilisi touristic and freight traffic flows inside the country and from neighboring countries.

Akhaltsikhe is a multicultural city, with access of cultural heritage monuments of outstanding importance. Akhaltsikhe can be considered as educational, tourism and recreation, logistical and distribution center.

Etsablishment of Akhaltsikhe University was a foresighted strategyc decision of first independent

government in 1991.In 1990 in Samtskhe-Javakheti the first high education institution, the Akhaltsikhe branch of Tbilisi Ivane Javakhishvili University was established.

It is desirable to develop learning programs of international capacity in fields of spa andresort medicine, tourism, astronomy.

In terms to increase tourism potential is proposed to develop South Georgian medical tourism centers, connected, for example, to Akhaltsikhe.

Borjomi is well known as climatic and balneology resort. (Inhabitants: 10,5 thousand) [16]. Recently, after stagnation of the crisis years, several hotels with spa-services were arranged. However, it is not sufficient for the development of spa industry. It would be benefitial to develop affordable medical tourism for the cityzens of Georgia, what requires the cooperation with state health care programs, as well as state and private insurance companies. For this case, it is possible to use different European examples, like German Union of Resorts - Deutsche Heilbäderverband e.V. system,[19] Lithuanian medical tourism cluster [20], European Spas Association [21] etc.

Development is impossible without communications, but the influence is not always positive. Cargo transit is adversely affecting the ecological condition of the resort city Borjomi. Resort doesn't have a bypass road and the entire transit flow directly runs along the center of the city.

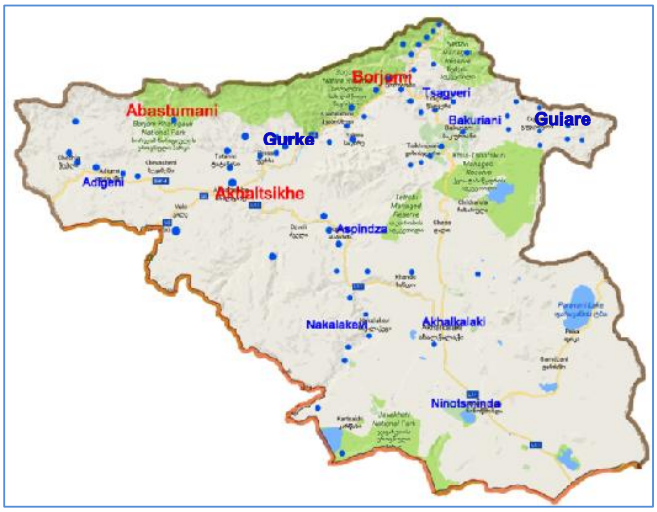
In 70-es of the last century, there was an idea about bypass road of Borjomi. About 6 km long tunnel arrangement was proposed. This proposal was rejected, because the damage of Borjomi volcanic rocks could endanger valuable mineral water resources. Cargo transit function of the South Georgia should be partly undertaken by the "South axis" [4,5].

Abastumani is the unique climatic and spa resort, withastrophysical observatory and up to 122 cultural heritage monuments. To increase its importance is nessecary to restore proper management system. (Inhabitants: 937) [16]. Abastumani is attraction center with historical importance, it could also attract tourists interested in art.

In 20-th century in Abastumani were operated 10 sanatriums, hospital and clinic. At present, there is a total decrease of medical facilities. Although the problem of pulmonary disease was not completely eliminated in Georgia [22], healing functions were preserved only in sanatorium "Zekari". Recently, the trend is the expansion of so-called "Wild tourism". Decrease of medical facilities in Abastumani is coused by lack of funds for this purpose. Arrangement of contemporary resort medical center will serve to the development of medical tourism and strengthen local spa, consultancy and therapeutic sector.

General plan of land use of Abastumani was created in 2007-2008. In 2015 strategy for sustainable development was elaborated [23]. For the branding of the resort is envisaged to support performing of Abastumani in the cultural and education center, to use main treatment profile and astrophysical observatory factors for the organization of international scientific events. Important healing potential of Abastumani thermal water, the discovery of which in 19-th century was the first reason for resort development, today is unused. Revival of spa-direction could be implemented through coordination with Tbilisi Balneology Resort Medical-diagnostic rehabilitation center – Tbilisi SPA.

Fig. 1. Scheme of existing and potential resorts and resort places, mineral water sources. [24]

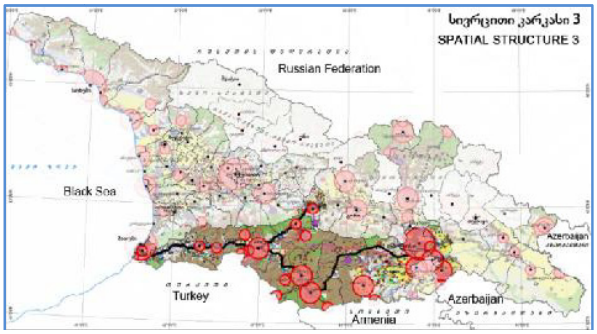


To support and strengthen local medical tourism, it is possible to create medical-spa cluster in the region: development of contemporary, competitive resort clinic, with ambition of the center of wide regional importance. Development of the appropriate spa and health-care functions in resorts and resort places, whose potential is underused or not used at all, in the form of hotels with spa-services, is an interesting prospect. These small local centers may be related to, cooperate or be subordinated to the regional resort clinic, for example, in Akhaltsikhe resort. Recently underdeveloped are climatic and balneology factors of Akhaltsikhe and Borjomi surroundings:

Uraveli, Nakalakevi, Gurkeli thermal water with neurological effects, Gujareti valley springs etc. This proposal is based on the spa and climatic scientific researches for the resort area [25]. So, Samtskhe-Javakheti resort medical cluster could be created.

3.2 Roads and communications analyse and hierarchy

Samtskhe-Javakheti region is crossed by the highway of international significance, which comes from capital city of Georgia, passes Borjomi and through Akhaltsikhe and Vale border station goes to Turkey. This way can be considered as a majorway in Samtskhe-Javakheti region. “Through the territory of the region passes the Baku-Tbilisi-Ceyhan oil pipeline, the Trans-Caspian gas pipeline, Marabda-Akhalkalaki-Kars Samtskhe-Javakheti, one of the important touristic and key regions is widely represented in the State “Spatial Planning” Program. The schemes are given as spatial structures, the same resettlement axles. Touristic importance of Samtskhe-javakheti region will increase through the construction of new 47 km long section of Khulo-Zarzma road.[1,2].



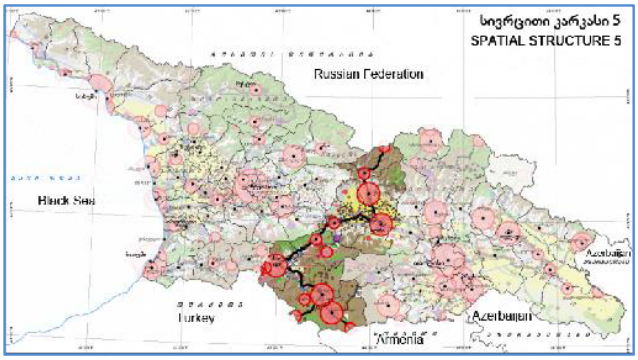
Connections of the “Southern axis” of Georgia along the border are presenting alternative way to spatial structure 1 and 2, which is the main road, connecting East and West Georgia. Khashuri-Borjomi-Akhaltsikhe section is the connection between major resettlement axis of the country and the “Southern axis”. Importance of the so called “Southern axis”[4] of Georgia, which is crossing Samtskhe-Javakheti, got into the spotlight during military conflict of 2008. Main railway” [7]. South Georgian resettlement axis becomes increasingly important in Georgian urban studies.

Fig. 2. Samtskhe-javakheti Region, Main Communications and Position on the Road Map, “Southern Axis” of Georgia. [13]

Fig. 3. Spatial Structure 3, Southern Axis: System of Samtskhe-Javakheti Communications in the State Spatial Planning Project.[1]

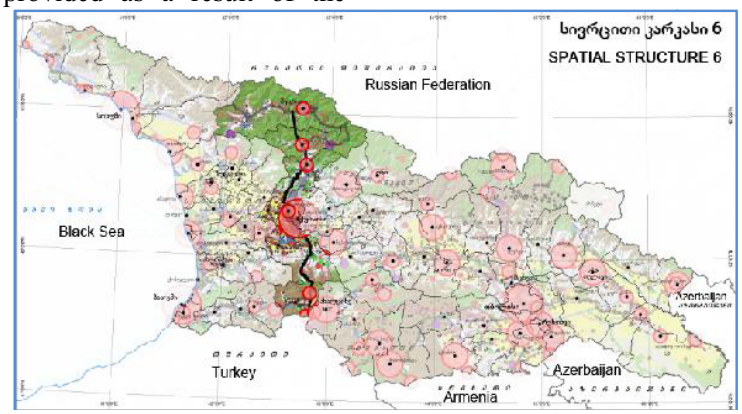
directions of “Southern axis” are: Tbilisi – Tsalka – Akhalkalaki – Akhaltsikhe – Adigeni –Khulo – Batumi. Akhaltsikhe is performing a key role between the West and East, where it could be considered as one of the main hubs of South Georgian axis. Difficulties arethe mountainous terrain and technical maintenance of the road.

Fig. 4. Spatial Structure 5, Kars-Ninotsminda-Java, Prospective Connection in Case of Deocupation. [1]



Samtskhe-Javakheti has an interconnecting position for the prospective axis of Kars-Ninotsminda-Larsi in case of deoccupation[1,2]. Connection of Samtskhe-Javakheti with North-West Georgia until Caucasus, "Akhaltsikhe-Mestia" will be provided as a result of the construction of Lentekhi-Mestia highway and tunnel and the new section of Abastumani-Bagdati 60 km long road". [2].

Fig. 5. Spatial Structure 6, Akhaltsikhe-Mestia, South-North Connection of Samtskhe-Javakheti. [1]



Strenghtening and increasing of wide regional importance of Kutaisiare envisagedhere. [1]. The section of the road in Abastumanialso has the strategic importance, but it should have primarily touristic function, because intensive traffic could create threats to the unique microclimate of the resort.

From motor roads of local significance following roads should be noted: Borjomi-Bakuriani-Akhalkalaki, Abastumani-Sairme, Akhaltsikhe-Aspindza-Vardzia etc.

Historical roads are located mainly along river valleys. They may have the strategic role not only as touristic routes, but also for the movement of people in any case, including in the case of risk.

How much the absence of roads can lead to degradation – good example for it is the Gujareti valley, located to the east of Borjomi, rich on mineral water resources, beautiful landscapes and historic monuments. Here are strongly damaged, but strategically very interesting ways that are connecting Samtskhe-Javakheti with Kartli region. For this kind of places with unused resort potential new**planning regions** should be defined. The valley could be developed after spatial planning, road rehabilitation, determining of the planning region, zoning, defining of tourism, recreation, agricultural and protected areas, arrangement of tourism infrastructure, shelters and, most importantly, as a result of proper management.

In terms of road communications interesting vision can be found in the work of S. Chikhelidze "Natural Resources of Georgian SSR, Mineral Waters":"None of the areas of Adjara-Trialeti system could be compared by its prospects of resort building capacity with mentioned region. Extension of highway Tbilisi-Tsalka to the west direction at 20-30 km-s and its connection through Gujareti valley with Tsagveri would be a quite reasonable measure - it would open the short way to the Borjomi health resorts complex and would stimulate large planned resort development in this exceptional, beautiful, healthy and rich in hydro resources region." [26].

There are more similar touristic connections in the region – Bakuriani-Tsikhisjvari, historic Tori, passing Dgvari-Tadzrisi-Sakire to Akhaltsikhe etc. Travel routes and communication infrastructure requires restoration.

Railway: Region is crossed by South Georgian railway from East-West hub Khashuri to border city Vale. The construction of Kars-Akhalkalaki railway is progressing intensively. Baku-Tbilisi-Kars is considered as a "Century project". [27].

Here should be specially mentioned the narrow railway of Borjomi-Bakuriani (1902), the valuable engineering heritage. The complete railway, with its stations, is of the historical significance, the functional-planning rehabilitation of which should revive entire existing urban network.

The main function of communication system for South Georgia is its strategic designation. Because Samtskhe-Javakheti region is the border area of high importance.

4. Conclusion

The strategic development, announced in the "Spatial Planning" plan of the country, needs to be specified and detailed on the regional level.

We envisage the cluster development as one of the functional-territorial directions of this effort. In Samtskhe-Javakheti as such can be considered the Tourism and Recreation Cluster.

Elaboration of the entire urban planning documentation for Samtskhe-Javakheti region and definition of planning regions in tourism and recreation places with unused potential could be the significant result of this research.

- In order to support the tourism and recreation attraction centers is considered to strengthenthe communication system of resorts Borjomi, Akhaltsikhe and Abastumani.
- As main strategic direction of Samtskhe-Javakheti region tourism and recreation cluster the development of medical tourism is determined.

Definition of Terms:

Urban Logistics – the same as city transportation logistics,the transportation and shipment of goods.

Cluster - a group with similar characteristic. Cluster analysis - or **clustering** is the task of grouping a set of objects in such a way that objects in the same group (called a **cluster**) are more similar (in some sense or another) to each other than to those in other groups (clusters).

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MONITORING OF BUILDING STRUCTURES BY FIBER-OPTICAL SENSORS

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Keywords:fiber-optical sensor, monitoring, construction, signal, contr.

Abstract

The modern construction is characterized by the high rate of introduction of new materials, analysis methods, constructive solutions, and methods of works performance. Experimental studies of building structures play an important role in such conditions. Recently is revealed the tendency to use more automated systems to evaluate the current technical state of building structures and buildings itself. These systems are called as monitoring systems and are mainly used for the evaluation of the technical state of the facilities for the field of aerospace and aviation equipment, railway, building [1,2,3,4].

The research aims is to develop complex system of monitoring for evaluation of building structures state based on the experimental-theoretical analysis and its implementation during the construction or operation of buildings and its implementation at construction or to provide safety of buildings during the operational process.

1. Introduction

The modern construction is characterized by the high rate of introduction of new materials, analysis methods, constructive solutions, and methods of works performance. Experimental studies of building structures play an important role in such conditions. Recently is revealed the tendency to use more automated systems to evaluate the current technical state of building structures and buildings itself. These systems are called as monitoring systems and are mainly used for the evaluation of the technical state of the facilities for the field of aerospace and aviation equipment, railway, building [1,2,3].

The research aims is to develop complex system of monitoring for evaluation of building structures state based on the experimental-theoretical analysis and its implementation during the construction or operation of buildings and its implementation at construction or to provide safety of buildings during the operational process

2. Methodology

Principal modification schemes for research, diagnosis, control and fault forecasting optical fiber systems of dynamic processes in building structures are mainly based on the principle of work of fiber-optic light waveguides, photodetectors, transmitters and other componentsof optical schemes.

The fiber-optical light waveguide represents transparent cylinder, composed of the core and casing. The light is spread from core. The light waveguide may currently have an external layer-coating that protects the fiber surface, increase its strength and facilitate operation. The core refraction index$n_1$is slightly larger than the casing refraction indexn_2, so the light from cores is reflected from the cored-casing border and fully is propagated in cor. For the full internal refraction in the light fiber the aperture angle will equal to

$$\theta_{\max} = \sqrt{n_1^2 - n_2^2}$$

$$\sin Q_{\max} = \sqrt{n_1^2 + n_2^2}$$

where theθ_{max} is calculated in radians. Because n_1-n_2=Δn<<n_1, then θmax<<1 and for aperture angle we will have:

$$Q_{\max} \approx \sqrt{n_1^2 + n_2^2}$$

Let's consider that physical properties of the optical fiber, which is more important for uits application as a sensitive element. More precisely, let's consider the character of change of propagated along along the fiberslight phase by mechanical and heat impact.

The phase change$\beta\ell$by presented measurement objectatapplication of interference type fiber-optical sensor, will be revealed as the intensity change of the interference. Therefore, for the determining of sensitivity of fiber-optic sensor due increased pressure, temperature and other influence, it is necessary to know the phase change$\beta\ell$ with the respect of these values. Let's determine the light phaseψ of sensory fibers, at a relatively small difference in the fibers refraction index:

$$\psi = \beta l \approx kn\ell \qquad (2)$$

where, n- is the refraction index of core.

Then the relative change of phases die impact objects under measurement (sensitivity) will be equal to:

$$\Delta\psi / \psi = \Delta\ell / \ell + \Delta n / n \quad (3)$$

Accordingly of the formula (3) is possible to apply relatively sensitive sensor for measurement of pressure, temperature and other values.

Fiber optic light waveguides and light energy receivers that are used to record the parameters in real time scales are characterized by hybridity and flexibility.

The functional diagram of the fiber-optical diagnostic system is presented in Fig. 1 [1].

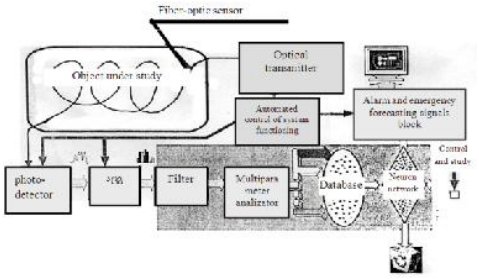

Fig.1. Functional Scheme of Fiber-Optical Diagnostic System

It consists from the following components: object under study, fiber-optic sensor, automatic control of functional systems, photodetectors block, analog-digital transformer, filter, multi-parameter analyzer, database, neuron network, alarm and emergency signal formation block and monitor.

The present system operates as follows: light impulse from optical transmitter is supplied to the fiber-optical sensor located on the object under study. After passing the sensor, the impulse will enter into the photodetectors block. The synchronization of the optical transmitter and photodetector operation is carried out by automatic control functional systems, from photodetector the signal will pass the filter, multi-parameter analyzer and get into the database where data processing is carried out by mathematical models of Fourier spectrum analysis. Then the neuron network chooses the data, which is nothing goes to the trash in the trash, and those gives the prognosis send to alarm and emergency forecasting signals formation block, results will be displayed on the monitor. Fourier and Vievlent spectrum analyzes are used for the processing of signals, and the results are comparable with the results obtained by the finite elements method. O

Physical effects, which are considered in engineering tasks, are described as a rule by the time functions, which are called realization of processes. The realization of process implies an echo that has been registered in the building structure point due the acting on it load. It is also meant that the load is dynamic, and therefore the echoing of the structure also is dynamic.

We consider the frame of building as a system and a combination of the appropriately interconnected elements of the structures (columns, girders, slabs, etc). The important feature of the system is the hierarchy. The system is hierarchical according to its nature, if each part of it can be considered as a system, and the initial system itself is the only one element of the larger systems.

Are considered all the structures included in the system that changes its physical-mechanical characteristics as a result of various impacts and also form potentially measurable signals that pass through the arranged in the system optical fiber as light impulses. Significant signals for a specific task are called as output signal. It should also be noted that the system is subject to the external impact. The passed through fiber-optical light waveguide signal that can be manipulated by observer is called as input signals. There are also other signals, they are called as noise (disturbances) that can be divided into two groups. The noise can be directly measured (measurable noise), and if it is impossible, noise would be included in the output signal (no-measurable noise). The distinctive characteristic between the input signals and measurable noise lies in the fact that measurable noise has less importance for modeling of the processes. Many tasks of modern science are solved by using a system-oriented approach. On the Fig. 2 is presented the system with signals designation.

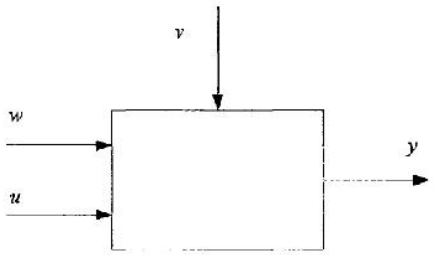

Fig. 2. Block-diagram of system with signals designation

On the Fig. 2. is presented the output signal *y*, input signal *u*, measurable noise *w* and non-measurable noise *v*.

As an example let's state dynamical system of damped and supported on spring body with one degree of freedom that is presented in Fig. 3. If the body experiences a variable in time load, then the body movement (reaction) will also be a time function. In addition, the system reaction $y(t)$ will be defined by the input impact $u(t)$ and the system's frequency characteristic (f). The frequency characteristic (f) depends only on the system's properties, such as: body mass *m*, spring stiffness *k*, damping constant *c*. The mentioned frequency characteristic is determined as a system reaction on the delta-function effect.

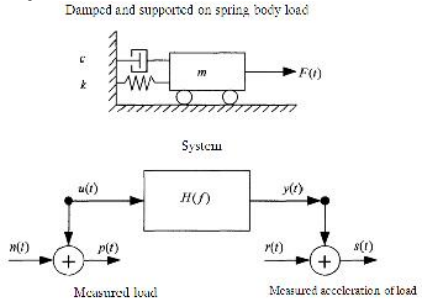

Fig. 3. Block diagram of damped and supported on spring body

The Presented on the Fig. 3 block diagram has one input $u(t)$ and one output $y(t)$, and if in the obstacles are registered at the input $n(t)$ and the output $r(t)$, then the being under observation input and output processes will be as:

$$p(t)=u(t)+n(t);$$
$$s(t)=y(t)+r(t), \quad (4)$$

where $p(t)$ – is the measured load; $u(t)$ – is the input impact on the system; $n(t)$ – is non-measurable noise at the input; $s(t)$ – is the measured acceleration of the body (speed of movement and so on); $y(t)$ – is the system reaction; $r(t)$ - is non-measurable noise at the output.

In the given case, by measurement of $p(t)$ the system reaction $y(t)$ with taking into account of noises $n(t)$ and $r(t)$, or without their consideration, may be determine the system transmission function as $y(t)/u(t)$ and simultaneously some of the function parameters (for example, the own frequency of oscillations and corresponding damping coefficient).

Since loads, and consequently reactions are changing in time, the dynamic task does not have the unique solution (in movements, stresses and other values), in contrary of static tasks. It is therefore necessary to draw a sequence of solutions that correspond to all the interesting moments in determining of the reaction of structure. From this point of view, the dynamic analysis is more complex than static. The described difference of dynamics tasks is important to take into account the at experimental studies - is necessary to apply for dynamics a device that can perform measurement of a fixed frequency in the range 0 - 1000 Hz, the implementation of that with significant reserve of is possible in experiments with developed by us fiber-optic systems [4,5,6].

Significant difference between static and dynamic tasks is that in the dynamic tasks there are inertia forces. If the hinged supported rod which is shown on Fig. 7 is loaded with a static load *p*, then its internal forces as well as the shape of bending will be determined only by the specified load, accordingly the mentioned values would be defined from equilibrium equations. In case of dynamic load impact $p(t)$, the movement of rod will be determined by accelerations that are cause by the inertia forces. Thus, internal forces generated in the rod must be balanced not only by external forces but also inertia forces that arise during the acceleration of rod.

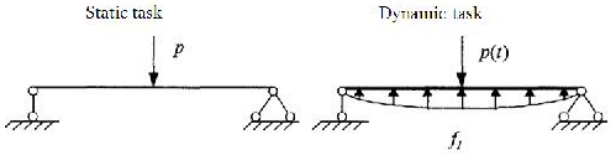

Fig. 7. Basic difference between static and dynamic tasks

If the inert forces constitute an important part of the total load that is balanced by internal forces, at solution of task should be taken into account the dynamic nature of the load. If the loading change is insignificant in time, then would be used a static analysis method.

The most common method of solving partial derivative differential equations in the structures analysis in dynamics is the method of finite elements (FEM). This method is used for any shape structures - rod, column, shell and other. The idea of FEM is in representation of structure as a finite elements system (FE) whose size can be arbitrary. At the same time, the FE are interconnected with each other in nodes, which have a finite degree of freedom.

FEM's advantage is the following:

- In the FEM models can be used different types of FE designated for modeling of rods, girders, shells, spatial bodies, etc.;
- The method is realized in many computer complex programs such as ANSYS, Lira, NASTRAN, ABAQUS and others;

-The FEM data can be easily used for applied tasks, for example by comparison of spectrum received in experimental and design models.

3. Conclusion

The developed in the paper monitoring system ensures permanent monitoring of buildings throughout the entire operation period and provides assessment of the technical state of load bearing structures, including the load bearing capability and before failure state assessment in real time.

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Several Features of Building Reconstruction

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Georgian Technical University

Abstract: In this paper we assess technological risks of reconstructing the existing ancient structures in Georgia and their influence on the ability of structural elements to withstand loads.
Keywords: Building constructions, Risks, Foundation, Soil, Buildings, Deformation,Sustainability

1. Introduction

When choosing the technology of building strengthening, following factors are important: constructional decision of the building, soil condition and equipment of the company that is currently carrying out the work. Using existing equipment makes it possible to carry out works for building and foundations reinforcement technologically faster, more reliable and using minimum manual work.

Modern methods of calculation based on geotechnical information provide the first opportunity to model competitive models of building and soil reinforcement. You can not abandon traditional technologies without the use of calculations. It often gives a positive effect when used in conjunction with modern technology.

2. Basic Part

When restoring-reconstructing buildings, it is important to ensure their long service life and reliability, which is made possible by inclusion of normal exploitation of load-bearing elements, as well as high-quality execution of technological processes and correctly chosen building standards. However, this is often not adhered to or is impossible to adhere to due to not taking into account the real risks of restoring buildings.

In today's Georgia, there are no regulations on the quality of design and construction (standards, state construction norms and rules) that would be tailored to the environment, its specifications, geological conditions, etc. All foreign companies engaged in construction in Georgia adhere to regulatory documents of their countries, while Georgian companies rely on the norms of the former Soviet Union that no longer correspond with the reality.

The main thing that is required from the methods of construction and designs of this type is selection of the most economical and modern technologies and methods of calculation, which should contribute to energy saving and preservation of the bioecology of the environment. This approach reduces operating costs and positively affects the environment.

All the traditional technologies for soil and foundation reinforcement are based on increasing the area of the existing foundation, which accordingly reduces the load on the base. Technological methods were also developed, with the help of which ground soil was improved by injecting chemical reagents into it.

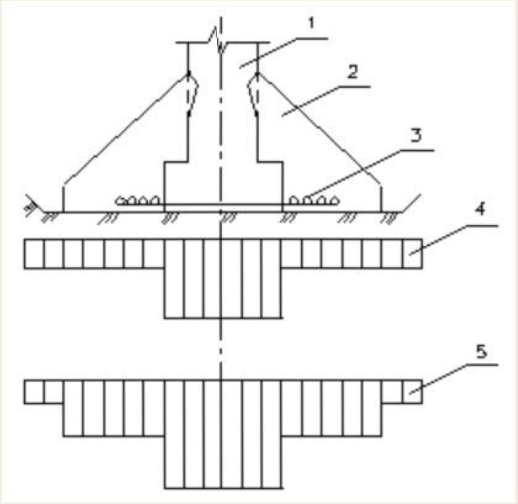

Fig.1. Scheme of increasing the area of foundation
(Diagram of load distribution on foundation according to B.I. Dalmatov)
): 1−existing foundation; 2−expansion construction; 3−fittings; 4− loaded diagram before expansion; 5− diagram after the expansion and loading of the foundation.

The drawback of this technology is the amount and variety of strengthening works carried out in limited basement conditions. In addition, as a general rule, the structure soil can be damaged due to constant humidity, which is associated with the uplift of the cultural layer. Such work should be carried out together with the reinforcement of the bearing parts of walls, which was done in the city of Kutaisi during the construction of the Bagrati monastery, which required restoration and re-construction of walls and foundations. As the designer considered: "If we set the whole structure on piles, it will be easy to

reinforce the walls, since all the load will be passed on to the piles" [1].

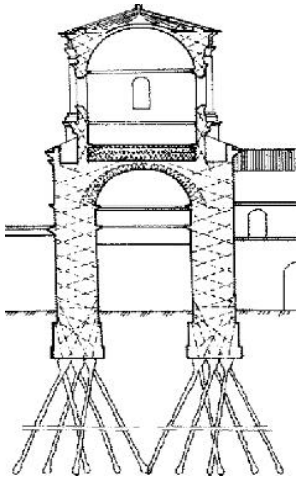

Fig.2. Reinforcement of the foundation together with the walls

3. Conclusion

Thus, for the reconstruction of buildings, and in particular monasteries, it is necessary to establish not only physico-mechanical characteristics of the soil and the ground, but also the use of prestressed roof structures for restoration, so that the damaged structure of the stone arch withstands the weight of the new roof and temporary loads (such as snow), achieving the desired redistribution of the load on the vertical load-bearing elements, reducing or stabilizing the cracks present in the stone arches or vault.

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Numerical Modeling of Ventilation Flow Distribution in Subway Tunnels with Considering of the Piston Effect

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Keywords: underground, piston effect, ventilation

Abstract: Metro is the main transport artery of modern megapolis and represented complicate underground building construction. Because of its multifunctional load, leading industrial states are focused on the issue of arranging subway under modern construction standards.

One of the main problem of metro construction and safe exploitation is to ensure the functioning of the ventilation system relevant to the increased safety norms.The influence of the piston effect on the dynamics of ventilation flows generated between metro stations, in the main part of scientific literature is described in the semi-empirical manner, is not characterized by high accuracy and inegatively affects on the underground design and construction works.

The current level of computer technology develops the possibility of the dynamics of ventilation flows in the subway tunnels by more accuracy in terms of numerical modeling of the air flow.

The paper discusses the problem of distribution of ventilation flows caused by the piston effect in metro tunnels, which give affects the determination of ventilation parameters.The task was set for the conditions of Tbilisi Metropolitan, with the following basic data: Tunnel length - 1200m; Cross section - 16 m^2, length of train 60 - 80 m; Train speed 6.0, 8.0,10.0, 12.0 m/s; Acceleration of the train – 1.0 m/s^2, the train cross section 4.00, 5.00, 6.25 m^2. The calculation was performed by the PYROSIM 2016 software.

As a result of the simulation, was determined the variability of ventilation flow caused by the piston effect depending on the speed and cross section of the train and the geometry of the tunnel.

In this workthe share of the piston effect of natural draught on the computation models and by means of same computation models are discussed. As a result of the modeling the nature of the variability of aerodynamic parameters of ventilation stream has been studied, such are: air consumption, dynamic pressure, air velocity in tunnel and velocity of back flow in space between perimeters of train and tunnel. In order to determine the mentioned aerodynamic parameters of the process we had a flow of air at one portal of the tunnel with average speed of 6.0, 8.0, 10.0, 12.0 m/s. As a result of numerical experiments, we determine the changes of dynamic pressure oscillations of the air flow along the tunnel to the front and back of the train. The base model used the following data for the Tbilisi metro: the length of the subway tunnel between two neighbor stations - L_0 = 1200 m; Train length L_{tr} = 80 m; Acceleration of train A_{tr} = 1 m/s^2.

Processing of the results, obtained by numerical modeling was performed with the appropriate values of calculations according to formulas, which were obtained from a natural experiment. In particular, the dynamic pressures difference can be calculated by formula (1) the velocity of back flow in space between cross sections of train and tunnel can be calculated by formula (2)

$$\Delta P = (0.164V_t)^4, \quad (1)$$

$$W_h = 0.84\sqrt{\Delta P}, \quad (2)$$

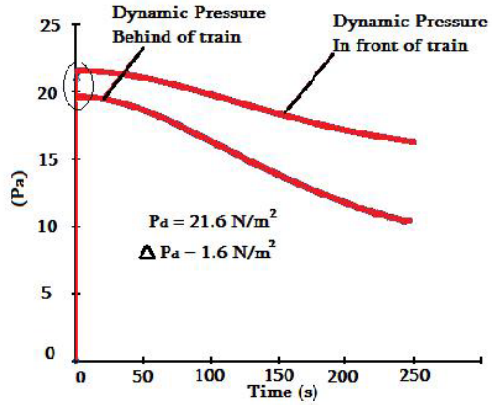
and the continuity of the stream in this case has a form

$$V_t S_t d = V_f S_0 d + (S_0 - S_t) W_h d . \quad (3)$$

ΔP is the difference of dynamic pressure in front and behind of the train; V_t - speed of trains in the tunnel; W_h-the velocity of air flow in space between perimeters of train and tunnel; S_{tr}= 4.00; 5.00; 6.25 - Area of the cross section of the train, m^2; V_f- air speed originated by the

piston effect on the front of the train; S_0= 16 - I

n these experiments by means of formulas (1) - (3) was realized the relevant boundary conditions. For the velocities 6.0, 8.0, 10.0, 12.0m/s accordingly, as the boundary

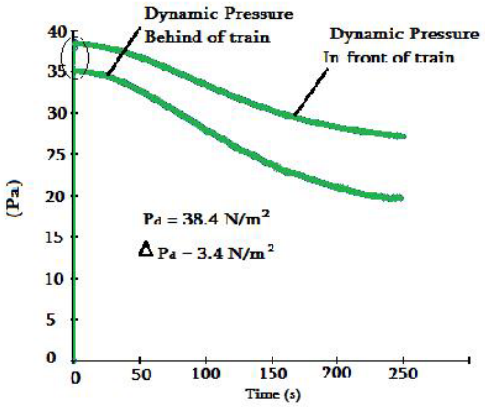


conditions, were used the dynamic pressures

Fig. 1. Characteristics of dynamic pressure variation at border condition 21.4 N/m²

Tunnel cross section, m^2.

21.6, 38.4, 60.0, and 86.4 N/m2. The train was located directly near the portal and we were analyzing the dynamic process development of



air flow near the portal in front and behind the train in the part of a tunnel.

Fig. 2. Characteristics of dynamic pressure variation at border condition 38.4 N/m²

It is noteworthy, that after the equivalent dynamic pressure modes on the portal, begin the transitional processes, which need to be considered. These processes depend on the value of dynamic pressure of ventilation air flow and rate of decreasing of its intensity.

The numerical results, obtained by modeling in the case of the different boundary condition are given in figures 1, 2, 3 and 4.

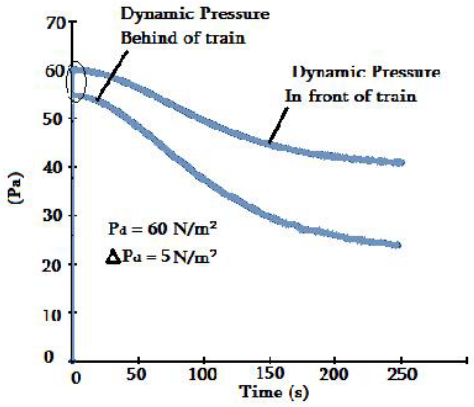


Fig. 3. Characteristics of dynamic pressure variation for theboundary condition 60.0

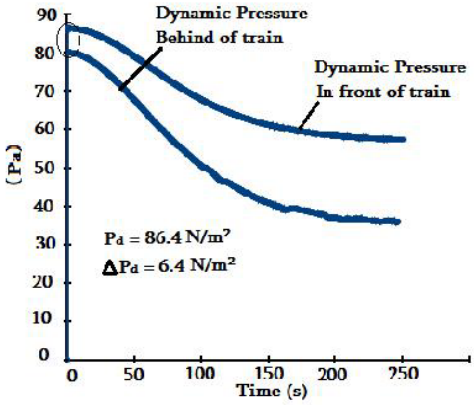


Fig. 4. Characteristics of dynamic pressure variation for the boundary condition 86.4 N/m²

As demonstrated in the results of the experimental research presented, it is possible to use the evaluation of air flows caused by the piston effect. In order to get more accuracy, it is advisable to consider the changing of pressures on the front of the train as well as behind the train for the initial phase of evolving of processes. It is necessary, as the rate of decrease of dynamic pressure is minimal in the initial phase of numerical modeling. This phase has been selected on the drawings 1-4. The results of the analysis when Str/S0 = ¼, are given in Table 1.

Table 1Numerical analysis results

N	Boundary conditions - dynamic pressure corresponding to the speed of the train on the left portal,N/m^2	Train speed in a tunnel V_{Vt} , m/s	Pressure difference on front of the train or on the end of train ΔP_a,N/m^2	The velocity of back flow in space between perimeters of train and tunnel W_h, m/s	The velocity of the flow in the tunnel that was originated by the piston effectV_f, m/s	The volume of ventilation flow that was initiated by the piston effect Q_p , m^3/s
1	21.6	6 .0	1.6	1.06	0.70	11.2
2	38.4	8.0	3.4	1.55	0.84	13.4
3	60.0	10.0	5.0	1.88	1.09	17.4
4	86.4	12.0	6.4	2.13	1.41	22.5

To determine the flow of air initiated by the piston effect, it is necessary to consider the tachogram of the train movement and determine the duration of the piston effect for the numerical values of the modeled velocities.

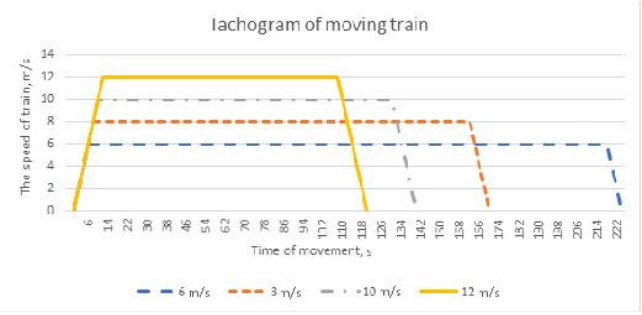


Fig. 5. The tachogram of the metro trains movement for the distance $L_0 + L_s = 1300$m

Train speed acceleration phase takes place in metro tunnel and the phase of the deceleration takes place in the metro station. Therefore, it is advisable to use the time ($T_{p)}$ required for counting the airflow caused by the piston effect. To determine this time interval, it is necessary to use times for acceleration of the train (t_A) as well as time of constant speed during motion (t_c). Accordingly, the sum of the required times is shown in the formula

$$T_p = t_A + t_c. \quad (4)$$

Taking into account the time (T_0) that is necessary to overcome distance between the neighboring stations the formula (4) will take the look

$$T_{pi} = T_{0i} - t_A. \quad (5)$$

Where the index i represents a different speed of train

The air volume originated by piston effect for the different speeds can be calculated using formula

$$V_p = T_p Q_p . \quad (6)$$

For speeds used in the present work, according to which computer models have been executed, calculating values by formula (6) are given in Table 2.

Table 2

The duration of the piston effect in according of the speed of the moving train

Train movement speedV_t , m/s	i = 6	i = 8	i = 10	i = 12
Duration of piston effectT_{pi}, c	218	162	138	108
The volume of air transported by the piston effectV_{pi}, c	2441	2170	2401	2430

Conclusions

- In the range of speeds 6-12 m/s, the variation of air flow rate, originated by the train movement has a linear character, and the volume of air intensivety, transported by the piston effect is 11.2-22.5 m3/s.
- Volume of air moved by one train in the same range of speed is practically constant and equals 2200-2400 m^3.

Exploration the possibility of using oil production waste in construction

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Abstract

The possibility for using the waste of oil and the production of gas – Sulphur in construction works when preparing concrete solution is considered. The tables for strength characteristics of sulphur concrete on light-weight and heave aggregates are given. Shown is the opportunity of sulphur soil production. Given are the data of strength characteristics. Sulphur soil is resistant to corrosive atmosphere and practically waterproof.

Key Words: Concrete, waste, sulfur-concrete.

Introduction

The Sulfur is residues of finding oil. Nowadays, There are lots of that countries where people are getting gas and oil. For example, in the republic of Kazakhstan there are accumulated eight millions of sulfur. Creating such materials which will durable in aggressive and other industrial natural conditions - are still actual problem [1].

Georgian black sea shore is a subtropical zone, with a moist climate of warm sea. The ground is rich with soil water. Very often, the soil is weak and it needs to make stronger with a lot of different methods with piles of ferroconcrete, changing a soil and etc). That kind of sulphate environment has a negative influence on the constructions which are located under the ground and over the ground. It causes destroying concrete and buildings, also armature is under corrosion. Resolving these problems is possible with creating sulphur concrete researchings of last years showed that for getting chemically stable materials it can be used sulphur, with supplements and reactive aggregates [1].

The sulphur concrete is a conglomerate where the astringent is hot sulphur on 150 - 170^0C.

The technology of getting sulphur concrete is a little bit different from the technology of getting asphalt concrete, that's why it's possible to use factories of asphalt concrete while creating a sulphur concrete (Fig.1).

Main part

The sulfur concrete is very popular in a different countries of the world. Rein forcing a sulfur concrete is done like asphalt concrete, cooling of sulphur in sulphur-concrete – less than 120^0C (the temperature of sulphur crystallization).

The advantages of sulphur concrete technologies are mobility of the construction of sulphur concrete mixer, also sulphur concrete can become very strong in a short period of time, and than preserve strength. Because of these distinguish features we can use sulphur concrete in extreme conditions. For example, restoration of coast – protecting constructions is very important. Elements – necessary for coast – protecting workings – blocks, tetrahedrons and others can be fabricate at the place with relevant materials. We can put the sulphur concrete in any forms which doesn't lose strength on 170 – 180^0C, and also we can leave it in that gauge as a final product.

Besides, the sulphur – concrete is characterized with water proofing, also it's very frost – resistant. According to these properties, the best area for using the sulphur – concrete is damp environment. We have to underline one more special thing – using defective sulphur – concrete again, after breaking, melting and making new forms.

According to rigidity characteristics, the sulphur concrete is on a middle loves between B25 - B30 class concrete and polymer – concrete. In dry condition it's chemical characteristics are equal to different king of polymer – concrete, and price is too law in comparison with the cheapest concrete.

Researches which were done after two years from making sulphur concrete shaued that it saves reinforcement bar from corrosion. The sulphur concrete is successfully used monolithic constructions.

For getting high solidity sulphur concrete its necessary to add fiction. Experimental researches showed that to reaching this goal it's better to use mineral dust. It gives us possibility to resolve a problem of using industrial leavings, especially while building roods and aerodromes.

It has been chosen components of sulphur concrete based on researches for future scientific experiment. Characteristics of sulphur concrete were found as the same method as for usual concrete (Table 1) [4].

There is given in the schedule components of sulphur concrete for heavy fillers, also strength of concrete probes after 40 minutes and 3 days of becoming cold. There were prepared $10 \times 10 \times$ 10cm probes of sulphur concrete.

We can make a big economical effect while building roads and aerodromes, which will be shown in economy of concrete and time [1]. Also it's very useful to use sulphur concrete in underground buildings and conducts .

It's clear that high quality covers of roads and aerodromes are contained of two layers. Each of layers thickness are 20-25 cm. changing base of concrete with sulphur – concrete we can resolve several cases:

- Economy of concrete;

Table 1

Sulphur concrete contained, strength and crack stability index of sulphur concrete

sample №		Material costs 1 M^3, kg					Volumentric mass ,1 kg/m^3	Mass correlation of concrete contains	Prismatic compressive strenght ,, Kgf/m^3	Bendingstrength , Kg f/m^3	/,	Crack stability Kic, mn/$m^{-3/2}$	Crack stabilityKic, mn/$m^{-3/2}$ according to[5]
		sulphur S	Sand 0-5	Road metal 5-10	Roadmetal 10-20	dust							
Heavy filler	1	420	515	1173	-	240	2347	S : sand : Road metal$_{5-10}$: dust = 1: 1,23 : 2,8 : 0,57	361	90,6	3,12	1,135	0,52
	2	544	543	1017	-	240	2344	S : sand : Road metal$_{5-10}$: dust = 1 : 1 : 1,87 :0,44	312	105,3	2,91	0,711	0,42
	3	504	566	352	705	252	2376	S : sand : Road metal $_{5-10}$: Road metal $_{10-20}$: dust = 1:1,12 :0,7 : 1,4 :0,5	-	51,5	6,96	-	-
Forous filler	1	737	383	480	-	-	1600	S : sand : Road metal $_{5-10}$ = = 1 : 0,52 : 0,65	95	34	3,23	0,502	0,20
	2	976	674	400	-	-	2030	S : sand : Road metal $_{5-10}$ = = 1 : 0,69 : 0,41	229	76.7	3,5	0,874	0,36
	3	875	729	187,5	250	-	2050	S : sand : Road metal $_{5-10}$: Road metal $_{10-20}$ =1 : 0,83:0,21: 0,29	228	72	3,64	-	-
	4	266	480	604	-	205	1555	S : sand : Road metal$_{5-10}$:dust = =1:1,8: 2,27: 0,77	144	48	3,4	-	-

- The sulphur-concrete is stronger than cement-concrete, so in whole, double layer cover is characterized as stronger;
- Hurrying up reinforcing of cement-concrete with hot sulphur-concrete. Also warmth of suphur concrete is enough for concrete workings in winter period;
- Guaranteeing of sulphur utilization. The sulphur is leaving of oil producing, that's why it's a big problem to be sulphur on the surface of the ground.

Considering that the sulphur-concrete is waterproof, also it's leaving of manufacture. Using sulphur concrete is advisable in underground and over the ground constructions.

Aims and problems of researches

Last years grew up high buildings on the black sea coast, they mostly are located on the weak ground Colchis lowland [6]. To build a foundation on a weak ground is very problematic case. In such kind of situations, the foundation is piled and the ground is fully changed with sand pillows. All of there are connected to big economical time problems. The alternative is to make stronger ground with using sulphur. It's known that the sulphur concrete is reinforcing in a short period, after 2-3 hours from concreting.

For using sulphur as astringent we have to find out the correlation between ground and sulphur. For getting enough firmness of foundation, we have to research the results using different kind of ground, via comparing edge of the hardness with ramming and without ramming. And also it's importantto find out how stable is sulphur ground towards to aggressive environment.

Experimental part

For experiment we take a ground from one of the constructing field in Batumi. This ground was known as weak and useless for foundation. The humidity of the ground was 20%. Different from technology of preparing sulphur-concrete, for getting sulphur-ground we take only sulphur and ground. We dried up damp ground, mixed with sulphur and then heated. We made 10X10X10 cm samples from this mixture. We made an experiment on the samples we got, experiment about firmness (Table 2). While experiment on 5, 6 sample the mixture before filing in shapes were kept in liquid form.

Because of sulphur-concrete is very waterproof, we made an experiment on the sulphur-ground (sample N:5-7). Experiment on waterproofing and water percolation during 24 hours according to standard methods [7]. After experiment compressive strength was invariable.

Table 2

Composition of sulphur-ground and strength characteristics

Sample №	Material xpenditure on a sample , gr.		Mixture composition mass correlation	R_b, Kg f/m^3	Water penetrability and water percolation
	sulphur, S	Dry ground			
1	230	420	S:dry ground=1:2,1	45,6	–
2	300	900	S:dry ground=1:3	60,8	–
3	350	1000	S:dry ground = 1:2,86	78,8	–
4	400	1000	S:dry ground = 1:2,5	159,0,	–
5	640	1700	S:dry ground = 1:2,65	200,8	0
6	1293	2400	S:dry ground = 1:1,85	380,0	0
7	1300	2402	S:dry ground = 1:1,85	380,0	0

Conclusion

All of these experiments showed that using sulphur-concrete is perspective in different construction works, especially constructions in aggressive environment. The results of experiments are very interesting. It has to be perspective to use sulphur-ground while preparing foundation of buildings. This opinion is economically feasible, because in compare with usual cement-concrete, it has double economy.

Fig.1. Modern technologies for making sulphur concrete and asphalt concrete

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CURRENT PROBLEMS AND APPLICATION PROSPECTS OF DIFFERENT TYPES OF COMPOSITE REINFORCEMENT BASALT-PLASTIC REBARS FOR CONCRETE AND FOR STRENGTHENING OF STRUCTURES

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Abstract
The progress of science and technology depends essentially on the success in the field of creation of new materials. Composite materials represent a heterogeneous structure, formed by the combination of reinforcing elements and the isotropic binder, are now widely used in various fields of technology. Therefore, this article is devoted to the problems of mass implementation of different types of composite materials including basalt reinforcement in construction practice. The advantages and disadvantages of composite reinforcement are discussed in comparison with steel reinforcement. Proposals are stated needed for actions to improve and successful wide application of composite elements for reinforcement of concrete structures.
Keywords: composite materials, basalt fiber, basalt reinforcement, deformation modulus, strength reduction.

1 Introduction

The basis of composite reinforcement is a material which is formed from different fibers: glass, basalt, aramid, carbon, zirconium, aluminborsilicate, aluminmagnezial etc. and usually with binding based on thermosetting synthetic resin which cures under the influence of the temperature or irreversible chemical reaction converted into a solid, infusible and insoluble material with the three-dimensional network structure.

Due to high cost of composite reinforcing elements as a whole, for the mass application of composite reinforcement in construction, the most suitable are glass and basalt fiber [1-8]. One of the most important features of composite materials is a significant anisotropy of mechanical properties. Therefore, the use of composite materials makes it necessary to determine the rational schemes of reinforcement and distribution of the material in the structure taking into account its loading conditions. In comparison with steel reinforcement, basalt plastic reinforcement (BPR) is more lightweight (3.3-3.5 times), greater strength (2.5-3.0 times stronger), has significantly higher corrosion resistance (12-15 times), the heat shield and dielectric properties, it is non-magnetic and radio transparent [2,3,8,11].

High specific strength of the material is implemented with short-time loading in the direction of reinforcement. Under loads that cause high levels of stress in the bonding adhesive, material loses its benefits to a large extent. Thus, the composite materials have a number of specific features that require the development of special methods of design and calculation.

2 Main part

The growth of production of continuous glass fibers is being constrained by the growing deficiency of raw materials (soda ash, boric acid, sulfate, etc.), as well as high technical requirements and others. It is well known for the fact that the limiting factor in the widespread use of fiberglass structures is a relatively high cost, low limit of fire resistance, low modulus of elasticity and others.

In this regard, in recent decades production of other fibers, instead of glass, has intensively been expanded. There are currently several technological processes of continuous fibers (roving) of the widespread eruptive rocks - basalt, which do not require any additives [1,8,11].

In Georgia there are no deposits of iron ore, therefore, steel and all metal products are mainly imported from abroad, which significantly increases the financial costs of construction and installation works. On the other hand, there are huge reserves of basalt, from which you can get high-quality fiber, that naturally causes a great interest to the basalt technologies.

Continuous basalt fiber production technology with the single-stage method has good prospects for the use and distribution, but not all of basaltic rocks may be suitable for production of continuous basalt fiber.

There are several careers on the territory of Georgia (Marneuli, Akhaltsikhe, Chiatura, Bolnisi and others), from which the raw

materials high-quality fibers are obtained. The volume of found raw materials only in these careers is estimated at many millions of cubic meters. The basalt fiber obtained from these quarries has a high initial strength and durability.

The chemical composition of the investigated basalt quarries in Georgia, which are suitable for the production of continuous super thin fiber, is given in Table 1.

Table 1 The chemical content of the investigated basalt quarries in Georgia

Studied deposits of basalt rocks in Georgia	Oxides, %										
	SiO_2	TiO_2	Al_2O_3	Fe_2O_3+Fe	CaO	MgO	Na_2O	K_2O	MnO	P_2O_5	SO_3
Chiatura	46.39	-	14.84	12.08	10.08	8.82	4.18	4.18	-	-	0.25
Akhaltsikhe	44.99	1.88	14.82	9.86	11.98	9.81	3.03	1.67	0.1	-	-
Aspindza	46.28	1..57	12.06	12.08	9.22	11.67	2.09	0.58	1.06	0.32	0.1
Bakuriani	59.06	1.16	17.54	7.26	4.86	2.94	3.7	1.62	0.71	0.93	0.12
Okami	53.41	0.63	20.38	8.29	7.85	2.56	3.75	1.07	0.73	0.32	0.1
Marneuli	51.11	0.92	15.5	10.45	10.22	6.36	4.64	4.64	0.3	0.5	-

The experimental studies of basalt and of other fibers in the concrete confirmed the decrease of fibers' strength in time.The results of experimental studies in high humidity conditions and the alkaline environment of concrete show a significant strength reduction of basalt, glass and other types of fibers (Fig. 1) [8].

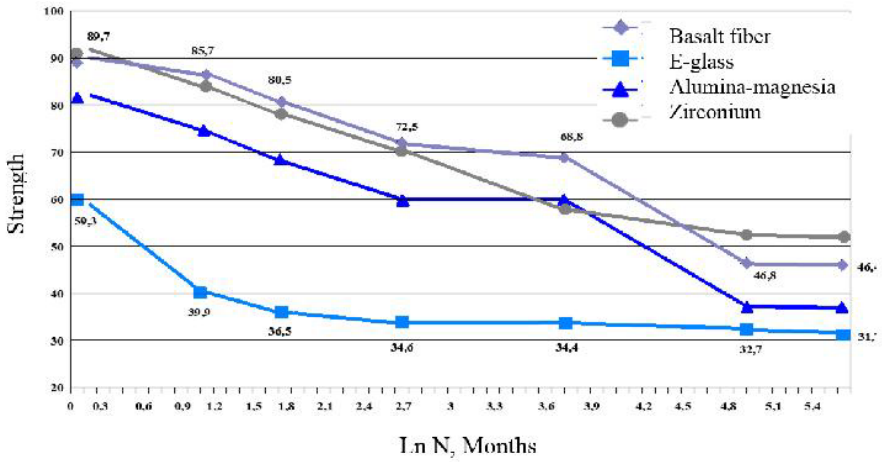

Fig. 1 Comparative time dependence of the strength loss of basalt and other fibers types in a hardening portland cement

Experiments show other features of behavior of composite reinforcement in the concrete. For example, the strength and modulus of elasticity of composite reinforcing elements are decreased with increase of the rod diameter [2,4,6]. The tensile strength of polymer composite reinforcement decreases significantly during the process of being in concrete, while the modulus of elasticity changes significantly [3].

Concrete and composite reinforcement (of glass, basalt, carbon, zirconia and other fibers) are rheologically active materials, therefore the load bearing capacity, the nature of the distribution of forces, deformability and crack resistance of concrete structures, reinforced with composite, significantly depend on the model of combined work during the long-term load action.

The tensile strength of basalt plastic reinforcement at T = 20°C is 12000-

18000 MPa and depends on the composition and percentage of binder in the product (which is typically 25-30%). Some other typical parameters:

- product density 1.95-2.2 g/cm^3;
- product elasticity modulus at 20°C ranges between 75000-90000 MPa;
- index of thermal conductivity of the product depends on many factors and can vary within 0,04-0,1Wt/m*K;
- linear expansion coefficient (0.5-0.7)x10^{-5}, at t =200°C;
- product melting point is determined on parameters of the binder and varies widely from 1800°C to 6000°C; initial softening temperature of the fibers is 6200°C, the softening temperature - 6350°C;
- basalt fiber melting temperature range from Marneuli quarries is 1150-11800° C; at a temperature of 12000°C fiber begins to char;
- special surface finishing is not required, the color of the product can vary from perfectly-black to light brown depending on the type of used binder.

Basic physical, mechanical and technical characteristics of BPR and AIII reinforcement are shown in Table 2:

Table 2 Characteristics of BPR and AIII reinforcement

Name	Value of the indicator		Note
	BPR	AIII reinforcement	
Temporary tension resistance, kg/cm^2	1.2x10^4-1,8x10^4	3650-3800	
Modulus of elasticity, kg/cm^2. Tension- Compression-	 (0.7-0.9)x10^6 (0.4-0.5)x10^6	 2x10^6 2x10^6	
Poisson's ratio	0.22-0.26	0.3	
Corrosion resistance increasing (times): In an acidic environment- In salt solutions- In hydrogen sulfide environment-	 12-15 9-12 15-20	 1 1 1	
Coefficient of linear expansion, grad^{-1}	(58-85)x10^{-7}	(111-150)x10^{-7}	
Volume weight, kg/cm^3	1.9-2.1	7.8	Lighter than steel by 3.7-4.1 times

The composite rebar in comparison with steel one has a number of disadvantages:

- low modulus of elasticity;
- low fire resistance and durability at high temperatures;
- inability to manufacture the bent reinforcing rods of composite reinforcement at the construction site;
- inefficiency and sometimes impossibility to use it in compressed zone of concrete;
- most of fibers, which form the basis of the composite reinforcement, lose strength in hardening Portland cement;
- during the long-term loading the nonlinear deformation of

glass and basalt composite reinforcement is observed even at load level 0.25-0.4 of breaking point.This makes it necessary to take it into account in calculations;

- the higher cost comparable with the cables of high-strength steel (ropes);
- compressive strength of composite reinforcement is much lower than the tensile strength;
- the virtual absence of plastic deformations in the composite reinforcement, low elongation at break - about 2%, which means an increased risk of brittle failure.

Due to the low elasticity modulus of composite reinforcement, the concrete structural elements may be destroyed in bending conditions. This character of destruction of the same section of the bent concrete structure with steel reinforcement is not possible. For this reason, high strength characteristics of composite reinforcement in most cases, remain unused.

The most effective area of application of composite reinforcement (BPR, in particular)in accordance with their features (high strength and low deformation modulus) is in prestressed concrete structures [3-6,9,10]. However, clamps used for prestressed steel may not be appliedsince a composite (and basalt plastic) rebar strength in that case is determined by the strength characteristics of the polymer binder which is significantly less than tensile strength of fiber [8, 9, 10, 11].This circumstance makes it necessary to develop a special type of clamp for realization of prestressing. The value of controlled stress in composite reinforcement should be taken not more than 50% of ultimate stress.
In view of the foregoing, before the use of composite rebar in ordinary and prestressed structures it is necessary to carry out a number of investigations to assess the efficiency of local composite materials, including:

- the influence of solar radiation and sea humid climate on the behaviour of concrete structures reinforced with composite reinforcement;
- the influence of the rheological properties of the binder polymer on the contact surface between the composite reinforcement and concrete;
- the influence of stress regime, especially under sign variable loads, which is very important in seismic zones and under dynamic effects;
- the influence of creep and shrinkage of concrete and interaction features on the contact surface, taking into account long-term strength of the composite reinforcement and different types of concrete.

One of the most important features of composite materials is a significant anisotropy of their mechanical properties. Therefore, the use of composite materials makes it necessary to determine the rational schemes of reinforcement and the material distribution in the cross section taking into account the relevant loading conditions.
Bond strength of composite reinforcing thin rod with the concrete depends on the structural features of the production process, the mechanical characteristics of the rod and the environment,however, when designing should take into account that the interlaminar shear strength of the polymer is provided with a relatively weak component. The strength of the connection between a composite reinforcing thin rod and concrete depends on the structural features, of the production process, mechanical characteristics of core and environment. And during design it should be considered that the interlayer shear strength of the polymer is ensured by a relatively weak binder component.

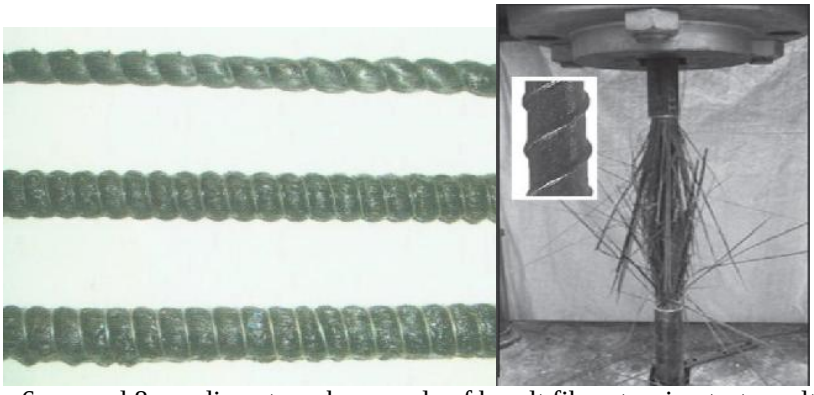

Fig. 2 4mm, 6mm and 8mm diameter rebars made of basalt fiber; tension test results of rebars made of basalt fiber

Figure 3 shows the experiment results- distribution of stresses in concrete in the contact zone with BPR (obtained experimentally by the author - J.Gigineishvili).

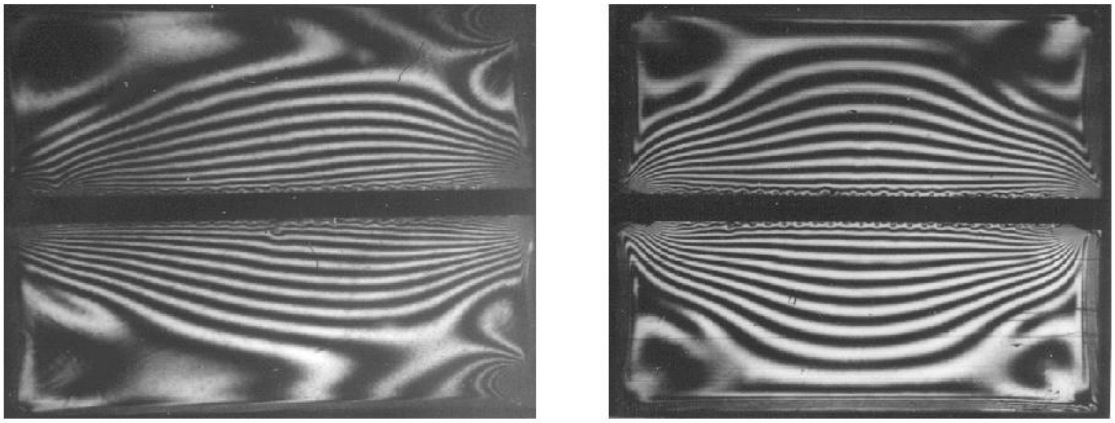

Fig. 3 Distribution of stresses in concrete in contact zone with BPR: in normal concrete (left); in expanding concrete (right)

The tension elasticity modulus of the composite reinforcement $E_{r.sp}$ (including basalt plastic) can be calculated by the formula: $E_{r.sp} = E_a F_{st} + E_s F_s$, where E_a and E_s – the tension elasticity modulus of the fiber and a binder, accordingly; F_{st} and F_s - volume content of glass (basalt) and binder in the composite.

The elastic stresses in composite reinforcement: $\sigma_{r.sp} = V_1 E_1 \varepsilon_1 + V_2 E_2 \varepsilon_2$, where V_1 and V_2 - volume content of glass and binder in the composite, accordingly; E_1 and E_2 – the elasticity modulus of fiber and binder; ε_1 и ε_2 –deformation of fiber and binder.

The Poisson ratio **m** of the composite reinforcement in longitudinal and transverse directions are different. For BPR **m** varies in longitudinal direction within 0.25-0.35 depending on type of binder.

Prestressed concrete structures reinforced with BPR, should be calculate for both groups of limit states: strength, cracking and deflections. Calculations of members with BPR are based on the following provisions: sections remain plane; stress diagram in the compressed zone has a parabolic shape at extreme edge of the compression stresses that are equal to calculated resistance at compression R_c. If the edge compression stress is $0.7R_c$ or less, the diagram is simplified to the linear one.

The chosen diagram form of compression stresses and the magnitude of the boundary stresses depends on technological and

structural factors. Reinforcement of the compressed concrete zone using any type of composite reinforcement is not economically justified. From our opinion, an obstacle in the practical use of all the positive features of BPR is the insufficient study and reflection in the existing regulations [12-27] and, most important, the absence of studies and an appropriate normative basis for the effective application of basalt plastic reinforcement in conditions of Georgia, the local deposits of basalt, its properties for the local realization in construction, especially under seismic loads, as well in offshore structures, under solar radiation, low temperatures and in other conditions.

3 Conclusion

It's completely impossible to replace the steel reinforcement by the composite one in the modern concrete products. But it's expedient and effective the use of composite reinforcement in certain conditions instead of steel.Primarily we are talking about chemical resistance, radio transparency and dielectric properties of the composite reinforcement.

Thus, it's appropriate to apply the composite reinforcement to the following construction sites: in structures exposed to aggressive environments, causing corrosion of steel reinforcement; in road-transport infrastructure; in concrete prefabricated and monolithic structures on elastic foundation (foundations, roads, airfield slabs etc.);in water treatment plants; for reinforcing the masonry; in structures operating under conditions of high electromagnetic fields and the potential difference; in tunnels, soil and bank revetment structures; in prestressed structures; during reconstruction, restoration of buildings and structures.

In our opinion, in order to expand the area of the composite reinforcement application in the construction activity, it is necessary to perform the following:

- develop standards and norms with calculation requirements of composite reinforcement taking into account its mechanical properties and methods of control, based on local materials and conditions including extreme conditions;
- prepare specifications on different types of periodic profile of composite reinforcement taking into account types and classes of concrete;
- work out standard solutions to ensure the required level of fire resistance of concrete structures reinforced with composite reinforcement;
- compile an inventory of composite construction materials and technologies necessary for preparation of normative documents;
- standardize the typical solutions of the bent rebar elements and work out the rules for their manufacture, quality control etc.

What will be the new technical solutions in the direction of wider application of composite reinforcement for concrete reinforcement, unfortunately, still has no clear answer. However, practice requires that they should be studied in greater detail, which offers vast scope for theoretical work and creative research.Success largely will depend on the creation and discovery of new types of fibers, binders and their joint application for conventional or special types of concrete.

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STRUCTURAL AND ARCHITECTURAL DESIGN IN CONDITIONS OF THE COMPLEX TOPOGRAPHY OF GEORGIA

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Abstract

The aim of this work is to evaluate and summarize the influence of topography in seismic conditions on some buildings recently designed in Georgia. In the paper the design methodology is proposed for adjustment of the of topography effect on seismic input parameters for design of buildings. The lack of code requirements and experimental data regarding this issue determine the importance of the problem especially for countries with complex terrain like Georgia.

In order to evaluate the influence of topography and to adjust the input seismic parameters a numerical methodology has been used including the following principle components: calculation of site-specific seismic response of the construction site at foundation level of a structure considering the real topography and geological conditions (2D FEM time history analyses); the numerical analyses of a structure using the site-specific input data; evaluation of stability of construction area in static and dynamic conditions and considered loads from the structure. The numerical investigations were carried out based on detailed computer modeling using software GeoStudioand LIRA-SAPR. The accelerograms at baserock level recorded in Georgia and in Japan (K-NET network) were used as input data for calculation of time history response and response spectra in key points and for the construction of site-specific design response spectrum curve.

On the recently designed structures the effect of topography is demonstrated, the quantitative results are presented– seismic soil response, relief amplification factor – significantly higher than specified in seismic codes.

It was concluded that the effect of complex terrain including its geometry, soil conditions should be considered in seismic design and the related requirements should be developed and included in seismic code.

Keywords:topography effect, seismic influence, time history, equivalent linear model, relief amplification factor

1 Introduction

The rational development of territories with complex terrain has great artistic advantages in comparison with construction on flat surface. But one should also take into account the negative aspects of the location of the building on steep slopes. These include the construction cost increasing due to the use of special types of buildings, more complex excavation, construction and reinforcement technology. On the characteristics of the relief, the upcoming costs of construction and the choice of the future house project are largely dependent. The relief is determined by the slope of the surface, which is calculated as the ratio of the difference in height of two points of the terrain to the distance between them horizontally, which is the tangent of the inclination angle of slope. The slope is measured in fractions or percentages. Usually, the area is flat with slope not more than 3%, small slope - from 3 to 8%, medium - to 20% and steep - over 20%.

With a slope of more than 15-20%, it is necessary to develop a special house project. The disadvantages of the slope can be turned into advantages if you design a multi-story dwelling with separate blocks. On the slope you can also arrange a terrace for rest, with a wonderful view.

Currently, the housing resources for the perspective and general development of Tbilisi intend the intensive development of mountain slopes and gorges. As a rule, the

terrain is not favorable for construction. In most cases, the slope angle is within 12-15°, and in some cases it 30° and more - see studied multistory building, Panaskerteli str., Tbilisi (Fig. 1).

With such deviations as the norms of the existing in our country and in other countries, the construction of buildings and structures on a complex terrain (with a slope of 12-15 ° and more) is not recommended (due to frequent collapses caused by gravitational forces, damage to buildings and structures, etc.).

In Eurocode the topography amplification factorST is presented for slope stability issues as informative annex[1] to be considered independent of the fundamental period of vibration - multiply as a constant scaling factor the ordinates of the elastic design response spectrum. The values of this factors are in range of 1-1.4. For average slope angles of less than 15° the topography effects may beneglected, while a specific study is recommended in the case of strongly irregular local topography.

In Italian seismic code [2] there are four topographic categories depending on inclination angle ***i*** of slope with factor from 1.0 (***i***<=15^o, flat surface, slope and isolated ridge))to 1.4(***i***>30^o, ridges with crest much smaller than the base). The hight of slope is not defined in this classification.

In French seismic code [3] the topographic factor τ (from 1.0 to 1.4) is taken into account for structures situated at the crest edge, the height **H**of the slope and its angle ***a***mainly define the value of τ.The maximum value of τis near the crest edge at the distance **b**which isminimum of two values: **b**= min{20tg***a*;**(**H** +10)/4} (Fig. 2).

Fig. 1 The multistory building on the slope

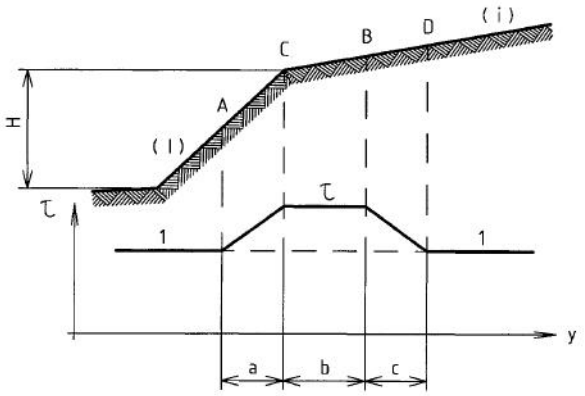


Fig. 2 The topography factor definition (from [3])

In Georgian seismic code [4] construction sites with slopes more than 15^0 require special design measures on foundation, structural strengthening and adjustment of seismic hazard with microzonation. The reason - frequent collapses caused by gravitational forces, damages of buildings and structures etc.

Thus, for the complex terrain the seismic codes propose the topographic factor depending on slope geometry and the building location on the slope. The local construction practice shows, that required in [4] seismic microzonation is not usually carried out and does not give the necessary data for structural analyses, e.g. site-specific accelerogram package. According to the analyses results of earthquake consequences, the real behavior of structures on the complex terrain is underestimated using the topography factor proposed in codes, e.g. [5].It is natural to assume that other factors can also significantly increase the dynamic response on the construction site - soil conditions, dynamic parameters of the construction site topography.

2 **Methodology**

The proposed numerical methodology of the seismic response assessment on construction sitesin complex terrainconditions includes the consideration of the relief geometry, geological data and the dynamic featuresof the construction site

area. In our case, the GeoStudio computer software was used for site-specific soil dynamic analyses [6] and LIRA SAPR software [7] was used for detailed calculations of the structural bearing system. The equivalent linear dynamic approach was used for analyses of soil response [8]. A dynamic analysis starts with the specified soil stiffness. The soil stiffness *G* is modified in each element in response to computed strains at each iteration according to specified *G r*eduction function. This iterative procedure continues until the required *G* modifications are within a specified tolerance *(G* is a constant during one iterative pass through the earthquake record). The static soil initial stress-strain condition is calculated before the dynamic analyses. The *G r*eduction function (G function vs. strain level and vs. stress) is specified in initial conditions for different soils compiled from existing experimental data [9, 10]. E.g. for clay material it is shown on Fig.3.

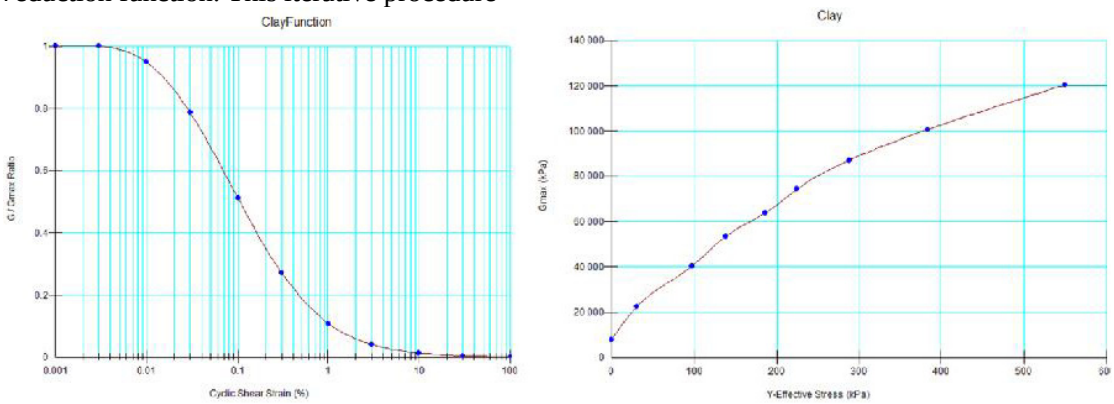


Fig. 3The *G r*eduction function vs. strain (left) level and vs. stress (right) for clay

In calculations a staged approach with the load history has been used and it is the following. The initial stress-strain state was defined in statical conditions for the soil massif without a building. The second stage represents the stress-strain state calculation with constructed building in statical conditions. At the next stage the time history equivalent linear (iterative) analyses are performed with recorded data in key points – at the base level where the input accelerogram as applied; on the surface near the bottom level of the slope; at the several points of the construction site on the slope top level. The last stage – the stability assessment of slope performed both in time domain and in limit equilibrium conditions. Then, for structural calculations of the building the response accelerograms at the foundation level are usedas input data for structural calculations of the building. Another approach – a package of generated accelerograms can be used for the building structural analyses on the basis of calculated site-specific response spectra. Below some results are presented demonstrated the significance of the complex relief consideration in design.

With the Equivalent Linear model a dynamic analysis starts with the specified soil stiffness, then steps through the entire earthquake record and identifies the peak shear strains at each Gauss numerical integration point in each element. The shear modulus is then modified according a specified *G r*eduction function and the process is repeated. This iterative procedure continues until the required *G* modifications are within a specified tolerance. The *G* is a constant while stepping through the earthquake record andmay be modified for each pass through the record, but remains constant during one pass.

3 **Numerical Modelling & Results**

The design model (Fig. 4) includes the relief geometry, geological data, designed building at the top of slope of 15m height. Key points (base, surface, edge of slope) where the necessary calculated parameters

are recorded are specified as well. The second model with rocky material (Rock1) instead of clay is the same, except the lithology;it is used for the comparison of results.

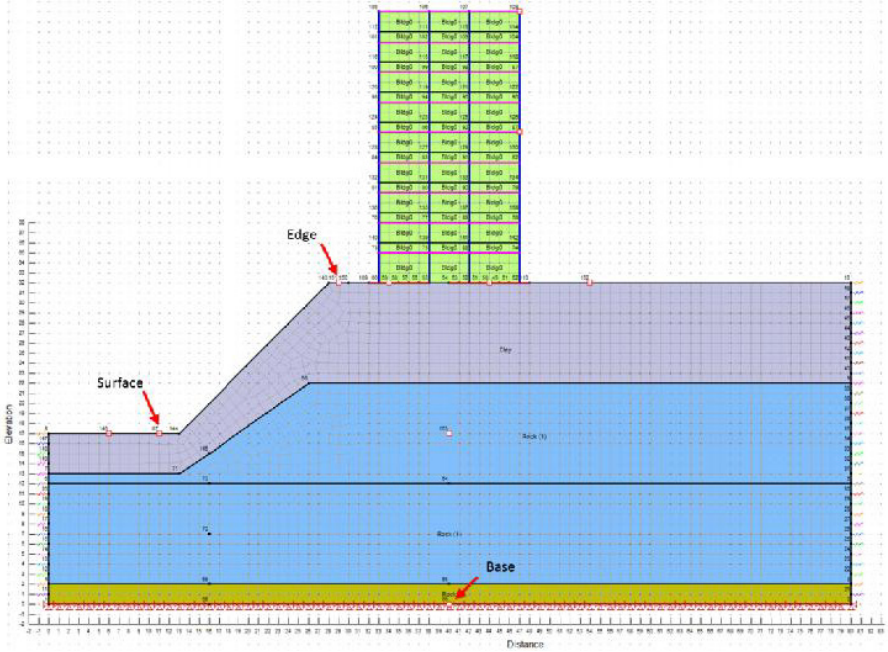


Fig. 4 Design model with ground massif, building, specified key points (Base, Surface, Edge)

For the numerical simulations the following accelerograms were used, see Table 1. The presented below results are obtained with El Centro NS input record which may not represent the high frequency components of the ground motion[11].

The soil initial parameters- Clay sand: **r**=18kN/m^3, **c**=25kPa, **f**=15^0,**m**=0.4; Rock(1): **r**=21kN/m^3, **c**=200kPa, **f**=35^0,**m**=0.25.

Table 1 Earthquake records used in calculations

#	Title	Comp.	pga [g]	Date	Site	Note
1	Tbilisi	NS	0.10	25.04.2002	Nutsubidze str.	On rock
2	El Centro	NS	0.35	18.05.1940	Term. Substation Bldg	RC slab
3	HKD087	EW	0.24	26.09.2003	Futamata	On rock
4	HKD087	NS	0.14	02.02.2013	Futamata	On rock
5	ISK003	NS	0.56	25.03.2007	Wajima	On rock
6	MYG011	NS	0.25	02.12.2001	Oshika	On rock
7	SAG001	EW	0.35	20.03.2005	Chinzei	On rock
8	MYG011	NS	0.38	04.08.2013	Oshika	On rock
9	MYG011	EW	0.24	04.08.2013	Oshika	On rock
10	AKT017	NS	0.22	14.06.2008	Yokote	On rock

All accelerograms were recorded on rocky foundation except #2. ##3-10 records are from the perfectly organized Japan K-NET strong-motion seismograph network [12]. Accelerograms recorded on the baserock are the most useful tool in our case for the further computation of the soil seismic response at any construction site with the known geological data. In numerical simulation all records were scaled to pga=0.2g corresponding to the normative seismicity level in the Tbilisi territory.

Fig. 5 Shows the Y-stresses at first stage of calculations, without a building; Fig. 6-

after the second stage, with the erected building.

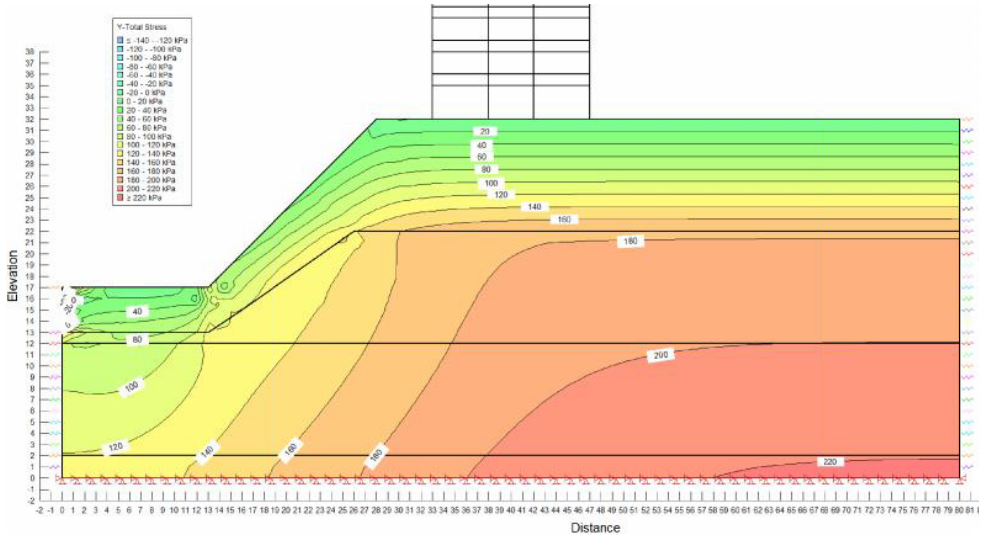


Fig. 5 Y-stresses in soil massif at initial stage of calculations

The dynamic response – accelerations in key points are presented in Fig. 7 for both cases of geological conditions.

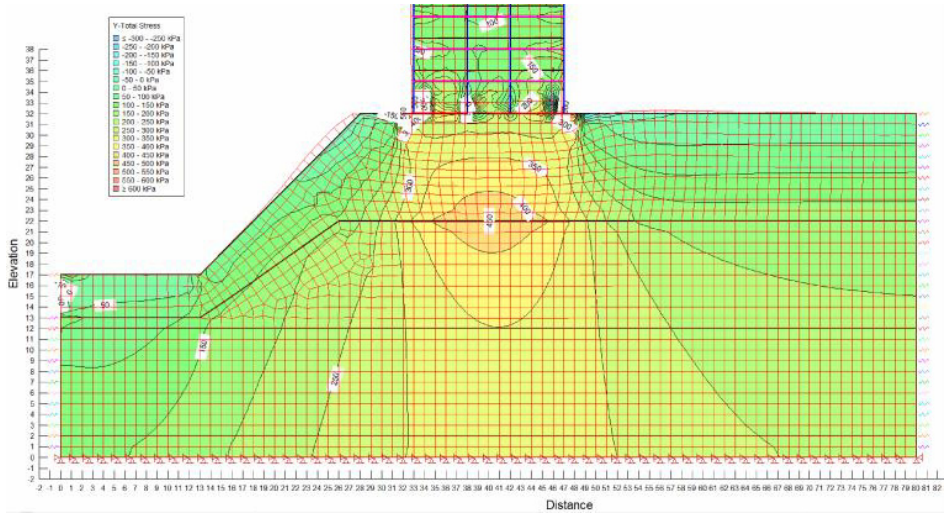
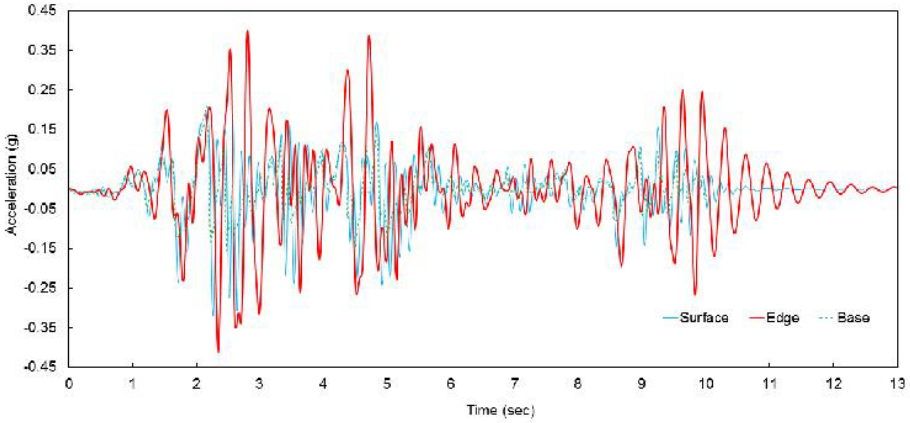


Fig. 6 Y-stresses in soil massif at second stage of calculations – with erected multystory building (the fragment of the building is shown)



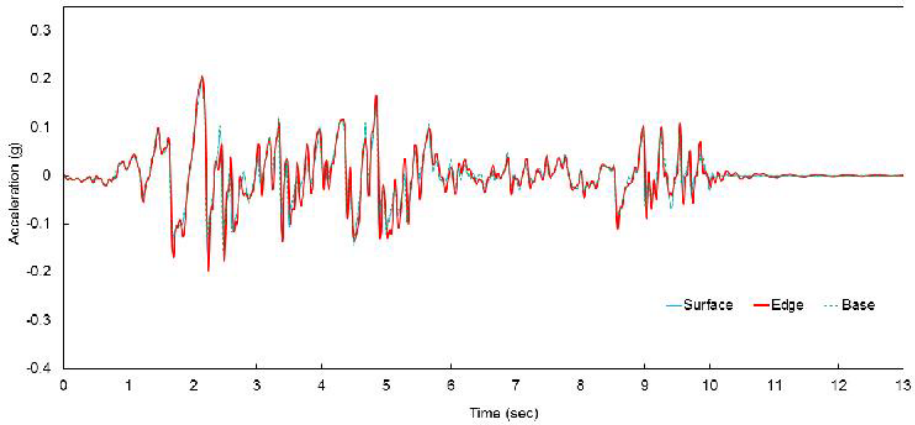


Fig. 7 Response accelerations in key points: the massif with clay-sand upper layer (above); the massif with rocky soil (below)

The acceleration response spectra in key points are presented in Fig. 8, 9 for both cases of geological conditions. The spectral amplification factor in the Edge key point relative to the Surface key point are shown in Fig. 10.

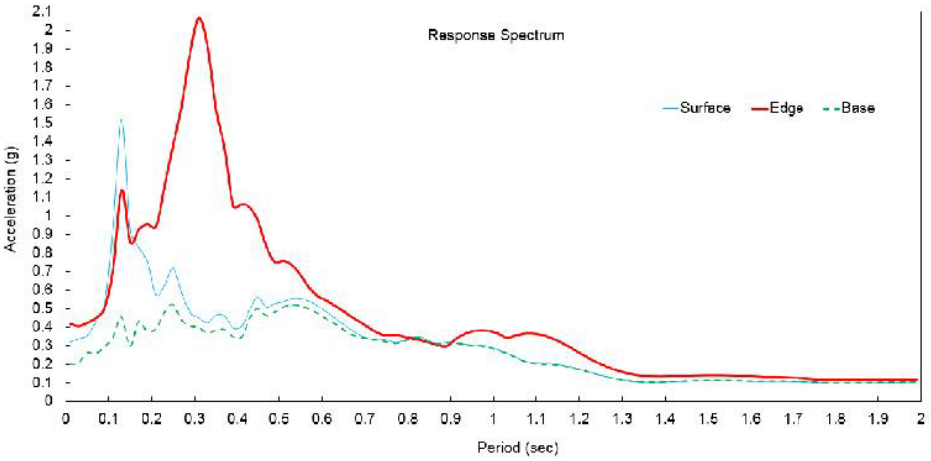


Fig. 8. The acceleration response spectra in key points of massif with clay-sand upper layer

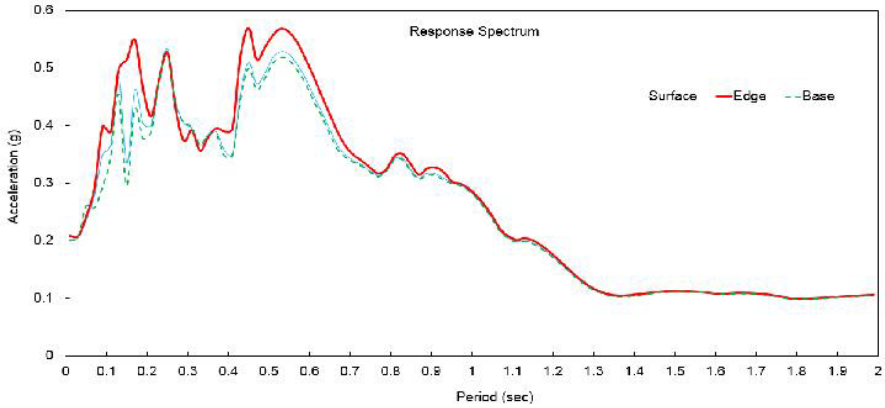


Fig. 9. The acceleration response spectra in key points of massif with rocky soil

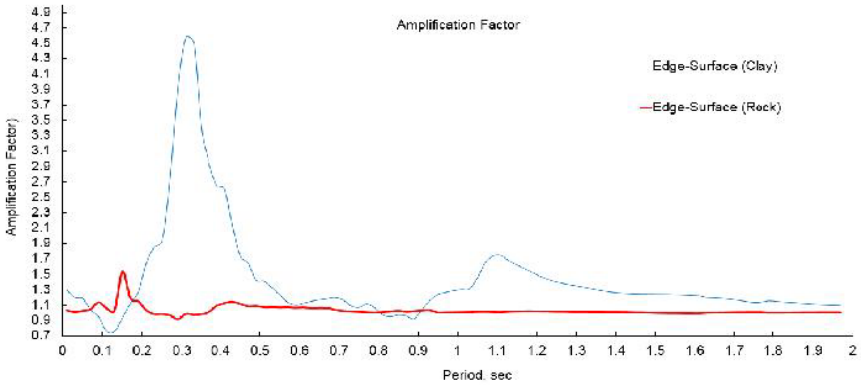


Fig. 10. The spectral amplification factor in the Edge key point relative to the Surface key point

In our case, the maximum value of amplification factor is 4.6 at T=3.2s; for the rocky massif this value is more than 1.5 at T=0.15s that also exceeds the maximum normative value 1.4 of the topography factor.

4 Conclusions

Based on the obtained results of numerical modelling we can conclude the following:

1. The design regulations for construction in complex terrain conditions should be developed in details, including not only the topography factor (multiplier for seismic forces depending on the relief geometry) but the other factors – soil parameters, dynamic behavior of soil massif with its own natural modes of vibrations which may significantly increase the soil seismic response on construction site.
2. The results of numerical simulation show that the soil seismic response amplification on a construction site situated on a complex terrain depends not only on the relief geometric characteristics but more significantly on soil parameters of slope and shape of relief that should be considered in design.
3. In case of a relief composed of homogeneous rock formations, some results of numerical simulation are generally consistent with the normative data [2, 3] related to the values of topographic factor but in most cases they significantly exceed the normative values that requires a careful interpretation and development of normative regulations and procedures especially for design of multistory buildings in complex terrain conditions.
4. The slope stability assessment should be, as a rule, an essential, inseparable part of design in a complex terrain preferablyperformed in time domain which gives, on our opinion, more conservative results comparing to the results of limit equilibrium methods.

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Influence of lattice substrate longitude acoustic oscillation energy localisation on deep centres on island growth into epitaxial process

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Abstract

The model of growth of the islands, taking into account effect of localisation of longitudinal acoustic oscillations of the host atoms of a lattice on deep centres is developed. Localization of oscillations: takes off degeneration from surface deep centres; defines partial pressure over the deep centres, which generates high frequency longitude acoustic wave; occurrence and magnitude of instant dipole moment of quantum transitions of vapor-gas phase molecules; adsorption; a deviation from stoichiometry; attracting of molecules and atoms of vapor-gas phase in epitaxial layer and concentration charged impurity; the form of growth islands, their size, and dependence of their density on temperature.

Key words: energylocalization, quantum dipole moment, acoustic wave generation, island growth.

Introduction

At the initial stage of GaAs so InP epitaxial layers growth by chlorid-hydrid method arise a growth islands. These islands can be used as quantum dots. These data prove to be true the researches, executed with use of microscopes by TEM (transmis sin-electron microscop), XSTM (cross-sectional scaning tunneling microscope), technicians XRD (X-ray difraction) and AFM (atomic - force microscopy). However, the model of growth of islands is not offered. In the present article the model constructed on the basis of phenomenon of localization of energy of longitudinal acoustic oscillations of the host atoms of a lattice (**LELAO)** on the deep centres (DC) in volume and on a surface of a substrate is stated.

Influence of DC of a surface on modulation of pressure and disperse interaction in vapor-gas phase.

Amplitude of oscillations of atoms of a surface more than atoms of volume. Localisation of energy of oscillations of the host atoms of a lattice on DC of a growth surface **compels** them to make oscillations on higher frequencies of a branch of longitudinal acoustic oscillations. They extend in vapor-gas phase (VGP) over a growth surface. This running acoustic wave harmoniously changes partialpressure over DC, causes modulation of growth rate and distribution of concentration of the basic components and an impurity, for example Si, in VGP. In VGP in places of crossing of fronts of the acoustic waves generated by DC of a surface, wave energy of the basic atoms of a substrate is raised. These places can become: the centres of excitation of atoms and molecules in VGP, places of the raised electronic and chemical activity in VGP.

High-frequency acoustic wave of compression and rare, collisions, concentration and temperature local fluctuations, quantum intermolecular interactions excites atoms of VGP by transfer an energy share by it. In the excited condition neutral atoms and molecules of VGP create electric dipole moment of quantum transitions. Stereochemistry of phosphorus and arsenic compositions depends on oxidation degree. Them d-orbital are energetically accessible, that allows to increase their valency to five or six and allows to participate in connecting orbital. At sp^3 hybridization, positively charged phosphorus or arsenic form simple tetrahedron structures. sp^3d hybridization is characterized by two basic configurations: trigonal bipyramidal and a square pyramid. Stereochemistry transitive elements also it is difficult enough. The

general property of particles participating in reaction is possibility to form instant dipole moments and, as consequence, participation in disperse interaction. The condition of balance of molecules VGP depends: on intensity of an ultrasonic wave, its long, expense VGP, laminarity of a gas stream, a constant of time of the relaxation defined by the mechanism of a thermal relaxation, properties of the oscillatory and rotary scheme of terms of molecules. The strong requirement of laminarity is a consequence of disperse forces very strong dependence on distance. Therefore kinetics of processes in VGP and in adsorption layer it is defined also by disperse interaction.

Disperse interaction is impotent for: chlorid, chlorid-hidrid and metalorganic vapor-phase epitaxy (MOVPE) methods.

Initial growth of an island on the excited DC

Complexes of type $Fe_{Ga}O_{As}$ ($Fe_{In}O_{P}$) or (CrO) settle down, basically, along [111] direction of substrates. In locations Ga and As there are atoms Fe and O, accordingly. On a polar plane (111) A there are atoms Fe_{Ga} or Fe_{In}. At temperature of growth **LELAO** leads to splitting of levels of a skeleton of atom Fe [1]. If the DC has more difficult structure, split off levels coupling, bonding and antibonding, loosening orbital (states). The part from them is located below initial levels, whereas another - above.

Located on gallium (indium) surfaces of a substrate excited DC with splitting levels by **LELAO** catch electrons. Electrons suppliers are atoms of arsenic (As_2, As_4) or phosphorus (P_2, P_4), being in VGP over the excited DC, an elementary impurity, complexes of type $SiAs, SiP$ and others [2].

As the electrons weight is small, the acoustic wave in VGP deforms an external electronic **cover** of atoms of arsenic (phosphorus). They lose the tetrahedron symmetry that promotes increase of number of arsenic atoms (phosphorus) catched by DC.

As a result, the first monoatomic layer of arsenic (phosphorus) atoms is not according with structure of an arrangement of atoms of a substrate, leading to infringement of growth microstructure and a deviation in it stoicheometry. Capture of an elementary impurity, complexes SiAs (SiP) and others brings additional infringements. In the subsequent atomic layers this infringement is partially inherited, but with distancing of a growing layer from a substrate it grown over because of decreasing of intensity of an acoustic wave.

Process of capture of arsenic (phosphorus) by the excited DC of a substrate surface defines growth of island on DC of a side (111)A. Because concentration of atoms Fe is great, also they are located mainly in a plane (111)A so on this plane the density of growth islands is maximum

If the growth surface is not a facet (111)A then atom of the impurity, for example-Fe, localizing longitudinal acoustic oscillation of host atoms of a lattice and located among row atoms Ga (case GaAs) or In (case InP), arranges settles down near to a growth surface. Roughly it is possible to consider this distance equal to half of lattice constant $a_O/2$. On a growth surface there are no deep centres with abnormal capture. As a result of excitation, the deep centre, arranged near a growth surface, catch electrons from semiconductor volume, leading to occurrence negatively charged area near to a growth surface. The local charge on a growth surface of the elliptic form appear also as a result of acoustoelectric effect. The acoustic wave in the form of a cone, generated by the excited deep centre of volume, carries away quasifree electrons to a surface. The section of a cone by growth surface creates an ellipse. The charge on a growth surface attracts molecules $GaCl(GaCl_2)$, influences on massflow masscarry factor and further growth according to model based on defining participation of GaCl is realized [3]. The surface of growth does not contain the centres with abnormal capture. Molecules VGP rather freely move on more homogeneous surface of growth. For this reason, on a plane of growth (111) In density of islands is

lowered. The shape of these islands is elliptical –like.

The islands quantum well

The island structure is nonuniform across the growth direction. The growth island contains the lattice mismatched layer as interface with dangling bonds. The island contains deep symmetrical quantum well of DC and rectangular quantum well ordered as island. His sides is not equal each other and depends on: growth temperature, elastic stress near the island, acoustic wave frequency, its intensity, quantum interaction with nearest neighbor DC. Excited surface DC is islands nuclei.

Influence of excited DC of volume on growth

It is established, that the thickness of epitaxy buffer layer of an order of micron essentially improves epitaxy layers quality. On this basis it is possible to consider, that the main contribution to growth processes bring DC located in this layer. In somebody approach the deep centres form flat two-dimensional periodic structure. On it diffracts the acoustic waves generated by excited DC of substrate volume. Such structure radiates secondary waves with length λ. Coherence defines collective movement of the host atoms of a lattice. Disperse resonant interaction promotes. At a multibeam interference in points of the main maxima (a difference of a course of beams $\Delta = m\lambda$) all oscillations come in identical to a phase, and the resultant amplitude of interference beams arise in N-times more amplitudes of separate oscillation. N-number of identical intensity beams coming to a point of the main maximum. Roughly, it is possible to consider, that $N = \sqrt[3]{N_s}$, where N_s-density of thescatteringdeep centres.

In the areas nearest neighbor to the main maxima the amplitude of oscillations, on the contrary, sharply decreases, reducing disperse interaction. The corner between axes of diffraction lattices defines the form of diffraction maxima. The island arrangement is rectangular or parallelogram. Strong disperse interaction in stains of the main maxima and almost zero in the neighbor causes growth of islands. The island interface is abrupt. The growth surface is an interferences field of acoustic waves of excited DC. Surface lattice of islands is well oriented by interference, depends on temperature, substrate and bulk crystal technology and DC concentration. Island is one dimensional quantum dot. It consist DC of growth surface and low ordered initial sublayer as the result of strong disperce forces on the growth surface. Through the initial deposition surface remains generally planar. With the addition of VGP the surface transforms from planar to three-dimensional islands. The changes in the islands size is insignificant.

The form of the surface of island

The density of the localized energy in stains of the main maxima, as well as nonequilibrium electrons density in the area near to a growth surface (subsurface layer), carried away by a stream of elastic energy to a surface, is maximum in the ellipse centre. For this reason the excited complexes of the centre of an ellipse are in conditions with the greatest quantum numbers. They most strongly attract VGP particles, therefore concentration of VGP particles is maximum in the elliptical centre, causing maximum growth rate in it. The surface of an growth island is convex [4]. On the other hand, energy localization create locally strained surface region. The strain field on the growth surface leads to an interaction between neighboring islands.

Model of an estimation of width of a growth island

In the elementary case of a two-beam interference of coherent waves distance ΔX between two nearest neighbor maxima of intensity it is estimated with under the formula $\Delta X = \frac{l}{d}\lambda$, where d-distance between sources of waves, *l*-distance to a growth surface, λ-length of a wave. Spatial coherence imposes restrictions onthe angularsizes of a source of radiation $\varphi : \varphi < \frac{\lambda}{d}$. In process of distancing from a wave source spatial coherence increases. We will admit, that $\varphi \approx 6 \cdot 10^{-2}$ radian, then the condition

$d < \frac{\lambda}{\varphi}$ gives $\mathbf{d} \le 17\lambda$. For example, for d $\approx 15\lambda$ then $\Delta X = \frac{l}{d}\ \lambda = \frac{l}{15}$, $15\lambda \le l$, $\Delta X \ge \frac{15\lambda}{15}$ or $\Delta X \ge \lambda$. Thus, width of an island is of an order of wave length. Length of a wave on the brink of Brilluin zone of an order $2a_0$, where a_0-lattice parameter. Hence, the width of a growth island can be estimated by means of expression $\Delta X \ge n \cdot 2a_0$, where n=1,2,3...., decreases with temperature growth. The received value of the expected diameter of a growth island corresponds to experimental data [4].

Dependence of islands density on temperature

We assume, that interference the stain of two-beam system defines diameter of a growth island: $\Delta X = \frac{l}{d}\lambda$. We will admit, that λ is defined by the first Wine law: $\lambda = \frac{b_1}{T}$, where T - absolute temperature of a surface of growth. The distance - d is equal to distance between two generating DC: $d = \frac{1}{\sqrt[3]{n_s}}$, where density $n_s = N_s e^{-\frac{\varepsilon}{KT}}$ of the excited DC possessing energy ε, sufficient for occurrence interference pictures. Hence, growth temperature rise causes disproportionate rise of islands density [4].

Dependence of islands diameter on growth temperature

Using earlier used formulas, we will receive**:** $\Delta X = \frac{l \cdot b_1 \cdot e^{-\frac{\varepsilon}{KT}}}{T}$. Hence, diameter of islands decreases with rising growth temperature because of reduction of length of a wave radiated by the excited DC**.** Structureof DC in i-type materials are difficult and actively absorb acoustic waves. Because of smaller density of the similar centres density of islands on n^+ type substrates is more low, than on i-type substrates.

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The Calculation of the Retaining Wall Lying on the Pressure Base

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Abstract

The originality of this work is in Basic part

Let's consider a retaining wall (fig.1). It is required to construct diagrams of ground reaction "η", shear forces „Q" and bending moments „M" for its base plate, as well as to calculate the wall shear and overturning. The angle of friction between the ground and the wall $\varphi_0 = 0^0$, internal ground friction angle $\varphi = 30^0$; the coefficient of friction between the ground and the plate material f = 0,58; specific ground weight γ = 18 kN / m^3; specific ferroconcrete weight Δ = 24 kN / m^3; ground deformation modulus E_0 = 41000 kN / m^2; plate material elasticity modulus E_0 = 2 · 107 kN / m^2. Height of retaining wall H_2 = 4m.

considering and analyzing the retaining wall as a construction with his baseplate located on the line-deformed base. Given are desing variables of the retaining wall. Forces acted on 1meter length and also the intensity of ground pressure on the baseplate, the weight of the vertical part of the wall the loads on the plate are taken into account. On the basis of the results, the wall according to the displacement and turning over is calculated. Given is the analysis of the baseplate in accordance with method proposed by the author with the use of the prof. I. Simvulidi's statements. Given are the diagrams of the ground's reactions "η", the cutting forces "Q" and the bending moment "M" which give the visual representation of the real work of the baseplate of the retaining wall.

Keywords: Retaining wall, baseplate, pressure base

Introduction

In most cases, at the time of calculation of the retaining walls,the combined action of thelower part of the retaining walls, the base plate and a collapsible base are not considered, which leads to the fact that the calculation results are not quite correct. Based on calculation of a specific example, in this article we propose to consider the work of the base plate as a structure lying on a linearly deformable foundation.

Preliminary dimensions of the wall structure: thickness of the vertical wall above δ_1 = 0,12 m; thickness of the vertical wall below δ_2 = 0,25 m; thickness of the base plate h = 0,20 m; base plate width L = 2,5 m;

The following forces act on the length unit of the wall: Active ground pressures (in kN):

$$R_1 = \frac{1 \cdot 1^2}{2} \cdot \text{tg}^2(45^0 - \frac{3}{2}^\circ) = 3$$

$$R_2 = \frac{1 \cdot 0{,}2(0{,}2+2\times1)}{2} \cdot \text{tg}^2(45^0 - \frac{3}{2}^\circ) = 1{,}32$$

$$R_3 = \frac{1 \cdot 4{,}0^2}{2} \cdot \text{tg}^2(45^0 - \frac{3}{2}^\circ) = 48$$

$$R_4 = \frac{1 \cdot 0{,}2(0{,}2+2 \cdot 4)}{2} \cdot \text{tg}^2(45^0 - \frac{3}{2}^\circ) = 4{,}92$$

Base plate weight: (kN/м)

$$q = 24 \cdot 0{,}2 \cdot 1{,}0 = 4{,}8$$

Intensity of ground pressure on the base plate (kN/m):

Left $q' = 18{,}0 \cdot 1{,}0 = 18$

Right $q'' = 18{,}0 \cdot 4{,}0 = 72$

Full distributed load on the plate (kN/м):

Left $q_1 = q + q' = 22{,}8$

Right $q_2 = q + q'' = 76{,}8$

Weight of the wall's vertical part: $P_c = 24\frac{(0{,}1 + 0{,}2)}{2} \cdot 4 = 17{,}76\ k$

By distributing the wall weight by thickness, we get: $q_c'' = 17{,}76/1 \cdot 0{,}25 = 71{,}04\ k\ /m$;

As quantity q_c'' is close to q_2, we accept the load to the plate on the right side as follows:

$$q_c'' = q_2 = 76{,}8 k\text{N}/m;$$

Checking the wall shear: Full vertical load on the base plate:

$$p_{n.b.} = 1 \cdot q_1 + 1{,}5 q_2 = 138\ k$$

The force of friction occurring between the base plate and its base:

$$T = f \cdot p_{n.b.} = 0{,}58 \cdot 138 = 80{,}1\ k$$

Shear force $R=R_3+R_4-R_1-R_2$; R=48+4,92-3-1,32=48,6 (kN)

Shear stability conditions:

$$\frac{T}{R}>1{,}62$$

$$\frac{8\ ,1}{4\ ,6}\approx 1{,}65>1{,}62$$

Checking the wall's overturning: $K=\frac{3\left(0{,}2+\frac{1}{3}\right)+2\ ,8\cdot 1\cdot 0{,}5+7\ ,8\cdot 1{,}5\left(1+\frac{1{,}5}{8}\right)}{4\ \ (0{,}2+\frac{4}{3})}=2{,}1>1{,}62$

Terms and conditions are observed.

Let's consider the base plate as the beam having width b = 1 m and a length L = 2,5 m, lying on a linearly deformable foundation and loaded with two types of load and a moment of Mc (Fig.1b).

For calculation of the base plate the index of flexibility must be determined preliminarily:

$$\alpha=\frac{1-\mu^2}{1-\mu_0^2}\cdot\frac{\pi\cdot E_0\cdot b\cdot L^3}{E\cdot I}=\frac{3{,}14\cdot 41000\cdot 1\cdot 2{,}5^3}{2\cdot 10^7\,\frac{1\cdot 0{,}2^3}{1}}\approx 150$$

By accepting conventionally distributed load q_1, acting over the entire length of the beam and the load (q_2 - q_1) on its right part and by using the tables and the formulae [3], we find the ordinate values η_1, Q_1, M_1 and η_2, Q_2, M_2 from q_1 loads and (q_2-q_1) and the ordinates η_3, Q_3, M_3 from the moment M_c being equal to:

$$M_c=48\left(0{,}1+\frac{4}{3}\right)-3\left(0{,}1+\frac{1}{3}\right)=66\ k\ \cdot m$$

Here $M_c$is a moment from the horizontal forces applied to the vertical wall, relative to the neutral axis of the plate section.

Diagrams of η, QandMare built onFig. 1 c, d, e.

Values of M are given in - kNm; Qв – kNm, ηв – kNm/m^2

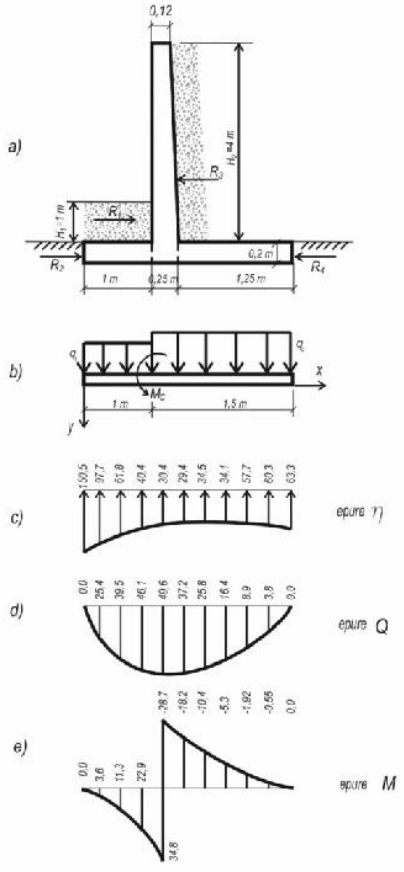

Fig. 1

Conclusions

The methods for analysis of the retaining wall are proposed in the present work. The baseplate is considered and analysed as a construction situated on linearly-deformed basis. Here is given an example of calculation.

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UNIVERSITY AS A DRIVING FORCE FOR IMPLEMENTATION OF THE TERRITORIES' DEVELOPMENT STRATEGY

(in the case of Ekaterinburg and the Ural Federal University)

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Abstract: Ekaterinburg is one of the largest and most significant administrative centers of Russia, which is dynamically developing, strengthening its positions in the world ratings, acquiring features of the capital. At present, the efforts of the authorities and society are aimed at forming the strategy of spatial development of Ekaterinburg. The goal of the strategy is to develop a concept of the urban space that contributes to improving the life quality of the population, provides a balanced growth of the city's economy and the social sphere development. The authors analyze the project "100 thoughts about Ekaterinburg" and strategic foresight sessions, in which representatives of business, science, culture and the authority took part; new projects presented at the international forums "100+ FORUM RUSSIA" and "Cities of Russia: 2030", where the prospects for the city development have been discussed.

The authors turn to the historical past of Ekaterinburg, reveal traditions and modern values of the present city, form a vision of the future metropolis on the basis of ideas and opinions of its inhabitants. The article shows that joint discussions on the urgent problems of urban development and collective project activities within the framework of public hearings of the spatial development concept of Ekaterinburg are a powerful tool for involving wide range of population for implementation of the proposed projects.

As an applied aspect of the research, the authors consider the implementation of the spatial development strategy of Ekaterinburg through the prism of the influence of the science and education on the formation of a person capable for implementing these ideas of the future. Universities should become centers of competence and social elevators for creating equal opportunities for participation of various population categories in the urban development projects. The Ural Federal University, as one of the largest engineering universities in Russia, has broad competencies and achievements in various fields of science and technology. It offers several pilot projects for implementation of the urban development concept. Projects are interdisciplinary in nature and cover various areas of life; they involve different age groups and representatives of various sectors of the economy. The proposed experience can be used in other cities and towns, taking into account their characteristics features and needs.

Keywords: spatial development strategy, university, science.

A city has been a centre of intellectual activity, culture, trade, science and production for a long time.

This process is irreversible due to the growth of the cities. A human is developing himself and evolving in different directions in the city. Of course appearing problems cannot be dismissed. The mail problem behind the human is degradation of the environment and water. Overpopulation, lack of the resources for providing basic services, problems with the property, infrastructure degradation, energy providing problems etc. can be also distinguished.

The future wanted for everyone presumes the reduction of the problems listed above by progressive development and growth, more effective resources usage, decreasing of environmental pollution level, poverty level etc. As it has been mentioned, ecology goes to the first place. The development of the population identity, its intellect level growth with the education, science, sport (for example, the city of Yekaterinburg has always been remaining one of the cultural, scientific and sport centers) is highly important here. Nowadays the formation of different organizations contributing to the future development of this directions is happening. Basically the accenting of the attention is on the problem of the formation, development and uniting of the city communities for cities movement in different aspects.

So, as it has been mentioned before, the active creation and development of different city communities is happening on the modern step. A group of people united by the common interests and performing common activity (the Internet communities united by interest: forums, blogs, social network groups; the associations of people united by hobbies, goals, religious or political views, geographical position, ethnicity) can be understood under the communities [1, 2].

Discusses the movement of urban asset, that is, the development of the territory at the expense of individual subjects or communities of subjects [3, 4, 5]. Identification of active communities – the potential carriers of new meanings, joint activities, can become the most important subject of development of the city. An example would be the Strategy of spatial development of Yekaterinburg, which is included in "100 thoughts about Ekaterinburg" [6], strategic session, the project at "100+ FORUM RUSSIA", "Russian Cities: 2030" and the other "100 thoughts about Yekaterinburg", strategic session, the project at "100+ FORUM RUSSIA", "Russian Cities: 2030" and others [7, 8].

A specific example is the social responsibility of universities as part of the successful development of the country. Universities today are social launch pad in the Russian society, giving young people regardless of financial well-being of their families, the opportunity to make a career, and the economy is the best human resource.

The stimulation of the scientific and entrepreneurial activity of students is the growing trend. This innovation will push the development of any territory in the future.

Let's consider a student's social projects in the Ural Federal University (UrFU) [9].

University acts like a model of an open social structure in which the customer is not only the student and employer but the surrounding social space. Integrating the University through architecture in urban space is a trend of development of the university as a modern model of socially responsible and open institutions. It is important not only the openness of the University, but also the openness of government, business and culture. The university's mission will be successful, if there is synergy between the government and the university.

Let's consider the model by the example of the organization of the students at the Higher School of Economics and Management of the Ural Federal University. This is a constantly functioning student's platform – Project Bureau «Management of the territories».

The Project Focus is the development of social initiative and responsibility, leadership and professional competencies, skills, teamwork and project work of the students.

The current situation on the labor market is quite tough. It's rather difficult to find a job for a graduate student because many employers require a job experience, but most students do not have that. In this situation, something intermediate is needed, the «bridge» between the university and work which will allow student to move smoothly from one to another.

The project is interdisciplinary, meaning students of all majors can participate. The project involves the future builders, power engineers, economists, managers, physics, radio operators, information technology specialists, future employees of Ministry of Emergency Situations and others.

Teachers play the role of the experts, mentors of the students in the project. Technology project activities are based on the elements of games and practice of project management.

The essence of the school is to develop technologies to educate students on programs of applied bachelor degree of teamwork, the use of theoretical knowledge in practice by modeling the activities of real estate companies.

Each team is headed by the current real estate developer, whose mission is to help participants. Teams perform common tasks of the company, interact with real customers, face the problem and try to solve the non-standard situations.

OBJECTIVE OF THE PROJECT:

Creating the conditions for the practice of project's activity, based on the elements of applications and management.

TASKS:

- Create a comfortable working platform operating on the basis of the university to work on projects and to adjust them for the further activities with experts.
- Provide the long-term partnership between business and universities.
- Transit to the new generation of qualitative intellectual resources.
- Improve the efficiency of business investments in high-tech projects.
- Update of the educational programs with the direct participation of business.

THE TARGET AUDIENCE:

- The project is interdisciplinary in nature.

- Students of all majors can participate.

The project involves the future builders, power engineers, economists, managers, physics, radio operators, information technology specialists, future employees of MES (Ministry of Emergency Situations) and others.

ESSENCE OF THE PROJECT:

The essence of the school lies in the technology development of teamwork, application of knowledge in practice by modeling the activities of real estate companies.

Teams perform the application tasks of the company, simulating the interaction with real clients, work with real problems, try to solve the non-standard situations.

The project will be implemented on the basis of chair "Economy and management of construction and real estate market".

STAGES:

First stage: Fall (September/October 2016) nomination of the project "Management of development of the territory" through the UrFU competition website. The launch of the project is scheduled on 1 September.

The prospective participants will be required to upload the self-presentations through the website. Mentors (5-10 representatives of customers) together with tutors (teachers and the best students of the department, who will be directly involved in the project) select teams of 5-8 people, each team will have a certain practical problem.

The second stage: October 1, according to the selection by the team leaders, the list of the team members with the entry criteria will be published on the website. Participants will register and begin to interact with each other remotely (online) while preparing the team presentation. The team needs to examine the problem and understand it, the initial stages of the solution will be welcomed.

The third stage: on October 20, all the participants will gather in the auditorium I-154 for the face-to-face meeting with the tutors. The same day the trainings and quests will be conducted aimed at teambuilding and getting familiar with the Ural Federal University.

Fourth stage: October 21 presentation of the teams and issuing assignments, case studies from the practice of real companies. Start of lectures and master classes.

Fifth stage: October 25 presentation of case studies and awards.

A similar platform is scheduled for the spring.

PROJECT RESULTS:

- The creation of the mutually beneficial cooperation of several generations through series of lectures, workshops and training programs;
- Integrated effect through the conversion of knowledge for the benefit of the territory, namely, district, city, region, country;
- The development of the basic professional skills by participants meaning the first working experience and gaining specific knowledge in the field of development, including the management of territories and related spheres;
- Opportunity to create a network with potential employers.

This project will enable participants to develop the basic professional skills, get a first work experience and specific knowledge, including the management of territories and related spheres. In addition, participants have the opportunity to create a network with potential employers.

PROJECT meets the following performance indicators:

- Development of social initiatives and responsibility, leadership and professional competencies, team and project work skills of the students.
- Strengthening the cooperation of the university with its partners.
- Development of the scientific community and stimulation of the research activity of students.
- Mutually beneficial cooperation among several generations.
- Creation of the base of the intellectual potential of the best students.
- Increasing the attraction of students from leading foreign universities to study in Russian universities, including partnership of educational programs with foreign universities and university associations.

The project includes a comfortable working area and conducting the presentations or workshops with experts at least once a month.

In addition, the joint efforts of the multi-disciplinary teams provide a long-term partnership between business and universities. Both sides will move to a qualitatively new generation of intellectual resources for businesses and the university. The efficiency of business investment in high-tech projects will increase. University actualizes the educational programs with the direct participation of Business.

To assume, it’s a good way to identify gifted young people in the early stages. The alignment of individual tracks of intelligence is

based on immersion in the work environment, initiated with the specific tasks of an actual customer. Development of social initiative and responsibility, leadership and professional competencies, skills, team and project work of the students will be performed. Moreover, the mutually beneficial cooperation of several generations, cycles of lectures, workshops, training programs are included.

The integrated effect will be achieved through the conversion of knowledge for the benefit of the native land (territory), namely, district, city, region, country.

The synergy while achieving the objectives by the joint efforts entails the formation of stable developing communities to manage the development of the territory.

Thus, we need the generator of new communities in the form of a specific subject of urban life in the cities.

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Rational technology for collecting spent regeneration solution of cation exchangers and its reuse.

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Abstract

With the view of defending the environment and rational use of a reagent the technology for collecting waste regeneration solution during cation exchanger regeneration with sodium sulfate was considered. After collecting the thermo-chemical processing and recovering of the strength, the regeneration solution is used again. Given are dependences which give possibility to determine the time of each stage both for regeneration process, washing off and termination of spent regeneration solution and rinse water collection. Given is the dependence for determining the additional amount of reagents needed to restore the strength of the regeneration solution. Shown is the collection of spent regeneration solution mix and rinse water where the reagent consumption is minimal.

Key words: cation, exchanger, regeneration.

Introduction

For improving the environment and rational use of natural resources the technology for regenerating cation exchanger with sodium sulphate and reuse of regenerant solutions (fig.1) [1] was developed. The research was carried out according to the project ISTCG-725.

General

The reuse of salt waste water of Na – cation exchangers requires examination of the matter for collecting spent regenerant solutions.The technology of cation exchanger regeneration which is used today causes the dilution of regenerant solutions by water cushion being above the charge of the exchanger. The collection for reclamation of such waste water is irrational, as the necessity in additional discharge of the reagent for the reduction of regenerant solution concentration appears. Besides, the discharge into waste-water disposal system of the first, the most concentrated part of washed off water, also causes the loss of a reagent. For preventing these shortages and reducing the discharge of a reagent it is necessary:

- to make the discharge of water cushion;
- to define the volume of washed off water, which is expedient to treat jointly with the spent regenerant solution.

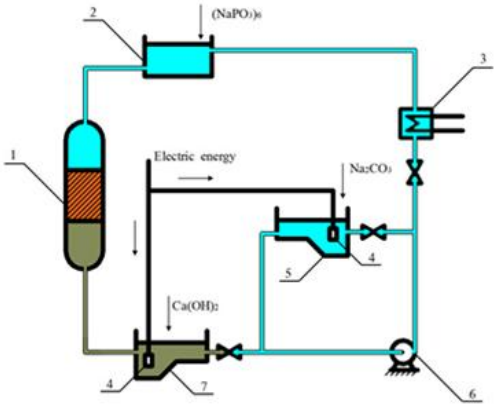

Fig.1. Technological scheme of regenerant solution reuse: 1 – filter; 2 – service tank; 3 – refrigerator; 4 - electric heater; 5 – settling basin for precipitating calcium carbonate; 6 – a pump; 7 - settling basin for precipitating gypsum and magnium hydroxide.

The calculated scheme for the determination of mixture concentration of the spent regenerant solution and the washed off water is given in fig 2. The regenrant solution with the discharge of q_1^p ,preliminarily freed from the water cushion and then the washed off water with the discharge of q_2^p are fed into cation exchanger. The washed off water presses out the part of the regenrant solution from the exchanger and the last portions dilutes.

The following marking is applied:

The duration for feeding regenerant solution into the exchanger

$$t = \frac{W_p}{q_1^p}\ , \qquad (1)$$

where W_p - the volume of regenerant solution, m^3.

The time from the beginning of the regeneration till the appearance of the regenerant solution at the outlet of the exchanger: $t_1 = \frac{\rho H_0 f}{q_1^p}$, (2)

where ρ - porosity of the charge;

H_0 - the height of the charge, m;

f - the square of the cross section of the exchanger, m^2.

The time interval from the beginning of washing off till the finishing the collection of the spent regenerant solution mixture and washed off water is designated as t_2. Neglecting the volume of additional (supporting) solution of regenerating substance, the following can be applied:

$$q_1^p \cdot t_1 = q_2^p \cdot t_2 . \qquad (3)$$

From here

$$t_2 = \frac{q_1^p}{q_2^p} \cdot t_1 , \qquad (4)$$

The time during whichthe regenerant solution with the maximum concentration at the discharge of q_1^p flows out of the exchanger: $t_3 = t - t_1$. (5)

t_4 - the time during which, the regenerant solution with the maximum concentration at the discharge of q_2^p flows out of the exchanger.

The time during which, the concentration of the regenerant solution changes from maximum quantity up to salt content of washed off water: $t_5 = \frac{mH_{б.п}}{v_1}$, (6)

where, $H_{б.п}$ - the height of liquid over the charge, m;

m - a part of water cushion volume participating in mixing [2];

v_1 - the speed of front movement with the concentration washed off water ([2],formula (2)).

Thetimeintervalfromthebeginningofregenerantsolutionconcentrationchangetill the final collection of mixture of spent solution and washed off water.

$$t_x = t_2 - t_4 . \qquad (7)$$

With the account of taken markings, the quantity of reagent not participating in mixing with washed off water is determined according to the dependence:

$$G_1 = C_p \cdot t_3 \cdot q_1^p + C_p \cdot t_4 \cdot q_2^p , \qquad (8)$$

where C_p - the concentration of regenerant solution, g/m^3.

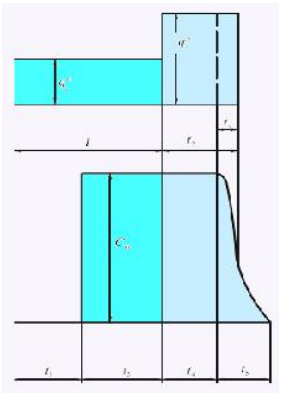

Fig.2. The calculation scheme for defining the concentration of the mixture of spent regenerant solution and washing off water.

The quantity of the regent participating in mixing with washed off water is determined as an integral of a tail part of the diagram (fig.2.) equal to approximation

$$G_2 = 0{,}4 \cdot C_p \cdot t_5 \cdot q_2^p . \qquad (9)$$

The quantity of substance passed through the charge of the exchange is equal:

$$G = G_1 + G_2 \qquad (10)$$

or

$$G = W_p \cdot C_p . \qquad (11)$$

Solving jointly the equations (8), (9), (11) relatively to t_4, after manipulation yields:

$$t_4 = \frac{W_p}{q_2^p} - 0{,}4 \cdot t_5 - t_3 \cdot \frac{q_1^p}{q_2^p} . \qquad (12)$$

Having determined t_4 according to the equation (12) we find t_x. Then according to the equation

$$\frac{C}{C_p} = \frac{C_в}{C_p} + \left(1 - \frac{C_в}{C_p}\right) \cdot e^{-\frac{vt}{mH_{в.п} - vt(1-\alpha)}} \qquad (13)$$

the concentration value under which it is necessary to stop colleting the mixture of spent regenerant solution and washed off water is determined. In the equation (13) $C_в$ - the concentration of salts in the water cushion, g/м^3; v-the speed of the solution movement above ion exchanger, м/h; α - a part of regenerant solution participating in the mixing process. The same concentration can be determined according to figure 1[2].

For industrial exchanger with the diameter of 3.0 m, and with the height of the charge of 2.0m., with the discharge of regenerant solutionandwashedoffwaterwhichcorrespondsto 14.0 and 28.0 m^3/hour, theconcentrationvalueunderwhichitisnecessarytostopcollectingthe spent solution is $C_x = (0{,}4 - 0{,}5) \cdot C_p$. The additional quantity of the reagent necessary for filling the losses is determined according to the dependence:

$$G_д = 0{,}4 \cdot C_x \cdot q_2^p \cdot (t_5 - t_x). \qquad (14)$$

Thelossesoccurduetoexpedientcollectionofthelastpart ofthemixtureoftheregenerantsolutionandwashedoffwater.

Conclusion

With the account of above mentioned the collection of spent regenerant solutions after semi-industrial installation was carried out according to the following technology. First discharged was water volume equal to the volume of the exchanger above the charge. Then the regenerant solution was fed to the exchanger.

Afterhavingdischargedthevolumeofwaterintoseweragеequaltothevolumeofintergrainspaceofthecharge ,thespentregenerantsolutionwascollectedintothetankwhichwasusedatthesametime as a settling basin.The collection was stopped after the regenerant solution had reached the half concentration. Under such kind of technology for collecting spent regenerant solutions for restoring the balance of natriumsulphate, the minimum quantity of the reagent is discharged.

Acknowledgements

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MONITORING OF BUILDING STRUCTURES BY FIBER-OPTICAL SENSORS

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Keywords: fiber-optical sensor, monitoring, construction, signal, contr.

Abstract

The modern construction is characterized by the high rate of introduction of new materials, analysis methods, constructive solutions, and methods of works performance. Experimental studies of building structures play an important role in such conditions. Recently is revealed the tendency to use more automated systems to evaluate the current technical state of building structures and buildings itself. These systems are called as monitoring systems and are mainly used for the evaluation of the technical state of the facilities for the field of aerospace and aviation equipment, railway, building [1,2,3].

The research aims is to develop complex system of monitoring for evaluation of building structures state based on the experimental-theoretical analysis and its implementation during the construction or operation of buildings and its implementation at construction or to provide safety of buildings during the operational process.

5. BASIC PART

Statistical analysis of earthquake frequency indicates on increase in their number and power.

If increases the number of earthquakes also increases the losses caused by them, because now more expansive is interior arrangement of buildings, materials, as well as more expensive is industrial equipment.

That is why are directed the efforts of scientists, seismologists and builders of advanced countries worldwide to develop and invent new seismic resistance measures.

It is logical that in this situation should be developed that field of science and design, which is directly involved in the protection disaster of the population and created by them prosperity from catastrophic impacts of the earthquake.

Number of such research, scientific-experimental institutes were functioning in Georgia: the well-known in Europe and worldwide institutions: Kirik Zavriev Institute of Structural Mechanics and Seismology, Tbilisi Zonal Research and Experimental Design Institute, Institute of Hydraulic Engineering and Building Materials, etc.

Each institution had its own scientific-experimental research bases, which were equipped with modern test equipment, devices and specialists.

That was why our institutions were performing the duty of the initiators and leaders in the former Soviet Union, including its seismically active 11 republics.

By our specialists were designed and built for that period most high 272 m arch dam on the Enguri River.

Was created the universal frame-panel system for seismic areas that lasted for almost 30 years when millions of buildings were built, which save a lot of money for the country.

The unique were series for the industrial multi-storey buildings for the 2500 kg/m² load; multi-storey of buildings with presstressed coverings, built accordingly of Prof. Zhezhelev system.

But time traveled and abroad scientists have developed new seismic resistance seismic insulation systems.

Observations have shown that in modern buildings at design earthquakes, are originated various size cracks, and at repeatedly shocks the number of cracks and damages are increasing, they are summarized that reduces the seismic resistance of building. An obvious example is Japan where earthquakes are frequent and buildings are easier to observe.

At the same time decreases the stiffness of the building and seismic load magnitude.

Thus, the reduction of the building's stiffness caused by the cracks reduces the acting on it seismic impact.

Scientists have used this circumstance to save the buildings.

Thus were created the seismic insulation systems, on which are now issued more than 100 patents.

What is the seismic insulation of buildings?

The reduction of seismic forces impact during a strong earthquake leads to reduction of

inertial forces in the load bearing structures of building, resulting in a decrease in the level and magnitude of structural elements fracture.

Reduction of inertia forces causes the reduction of the movement of the building's constructions; the reduction of displacement is associated with reduction of forces in load bearing structural elements. The reductions in forces cause reduction of cross-sections in the structural elements and consumption of reinforcement bars.

The roofing displacement is restricted that excludes the damage of non-structural elements and gives the possibility to apply the building immediately after a strong earthquake.

Summing up all of the above mentioned gives us a reduction in the seismic loading effect at strong earthquake due the application of seismic insulation facility up to 1.5-2.0 magnitudes.

In the case of application of considered by us rubber-metal isolators, their arrangement occurs mainly in the basement, on the level of columns and basement covering, where will be maintained obtained due proper calculation value of clearance.

We carried out appropriate computer calculation and analysis at having different height and purpose objects.

It was found that the equipped with seismic insulation systems buildings, besides protecting the main building from destruction and damaging, are characterized by certain economies in construction of multi-storey buildings and are diseconomy at low-floor buildings.

In addition, some impact on the diseconomy of low-floor buildings makes current acting construction rules and regulations PN 01.01-09.

Here, acting in the K point of the given direction appropriate seismic loading Sik is determined by the formula.

$Sik= K_1. K_2. K_3 S_{oik}$, where the K_3 is the coefficient of the building importance and K_3 = 1,4, or the accepted actual seismic load increases up to 40%.

The name of these buildings is given in Table ℓ 5: large railway stations, theaters, cinemas, airports, covered stadiums, schools, kindergartens, hospitals, fire, telephone and television systems, police, banks, museums buildings, etc.

Here the economic effect is less and it is known. But after the earthquake we have a colossal gain: to save people, children, the lives of patients, and this is a richer wealth. We will be able to liquidate earthquake results in a continuous mode.

We save the existing in the buildings and factories expensive infrastructure.

From the nature of the construction in our capital, in that were defined to be constructed 16-30 storey building and from the fact that seismic insulation systems provide a reduction up to 1,5-2,0 magnitude seismic impacts in the case of catastrophic earthquakes, the proposal is to apply all known seismic insulation systems (does not matter that!).

But as we have mentioned above, science develops in our field and has been fulfilled over the centuries the dream of the world's people: it is possible to predict the earthquake.

Because of the article format, we will not begin to characterize them and description of the results, we will only list our famous authors and methods.

In Baku, Azerbaijan scientists have developed a geophysical device-station "Tropotena", which operates in Baku, Islamabad, Jakarta, Istanbul.

We know that in the Siberian department of the Russian Academy of Sciences There is a geochemist registrar in Tomsk, who used at the construction of Vietnamese nuclear plants. Specialists of Saint Andrew the First Called Institute of the Patriarchate of Georgia are working on predicting earthquake in Georgia. According to them, they can identify earthquake space, intensity and its aftershockes for a few minutes.

Mr. Davit Chachanidze, a scientist and enthusiast is working in this field in Georgia.

There are still many scientists around the world who are attended this problem, but it is enough for us.

6. Conclusion

So what recommendations did our builders make?

1. At the design and construction of buildings should be taking into account all currently existing theoretical and already practically tested systems created by the structural elements, such as implementing for decades the unified frame units.

2. All responsible facilities (kindergartens, schools, hospitals, government, museum, police, fire, water supply, etc.) and high-rise buildings should be built using seismic insulation systems.

3. In seismically active cities shall be implemented available means to predict earthquake, for that in the conditions of Georgia will allocate adequate funds by government for scientific research and practical implementation.

4. Similarly to the European and American countries will be arranged wide networks for accelerograms registration on the ground as well as on multi-storey residential and public buildings, bridges, dams.

5. Similarly to the European countries must be established in Georgia training courses of engineering and technical personnel for at least once in 5 years the specialists will able to acquire new knowledge and experience in science, design and construction.

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ANALYSIS OF SANDWICH ORTHOTROP PLATES ON LOCAL LOADINGS

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Keywords:plate, loading, stability, convergence.

Abstract

Are considered new task of strength and stability analysis of having irregularities sandwich panels in condition of different types of supporting on edges and various shape of loading. Is developed the practical methodology of solution of non-linear differential equations with variable, in particular discontinuous coefficients. This method is based on linearization of initial equations by successive loading method and introduction in desired solutions of having unknown coefficients discontinuous functions that leads to rapid convergence of series and compact algorithm of calculation.

1. INTRODUCTION

Thin-walled shell structures are applied in the various fields of engineering: in machine-, aircraft- vessel-building. In the construction they widely are distributed as rational solution of industrial, agricultural, trade, cultural and athletics buildings.

The construction of large-span buildings and implementation in practice of low-modulus materials with high strength characteristics leads to necessity of taking into account larger in comparison with thickness deflections at analysis of thin-walled structures.

The development of various fields of industry and construction is related with improvement of existing and creation of new thin-walled structures, consisting from shells, plates, rods, having reinforcements, breaks, cuts, point supports. The group of above mentioned singularities is called as discontinuous parameters.

Most natural method to increase the stiffness of shell is the arrangement of ribs. On borders of holes, at impact of local loadings is advisable to reinforce the spatial structures by finite length ribs. The methods of ribs attachment to skin make impact on behavior of structure at deformation.

The singularities of geometrical and physical parameters in thin-walled structures cause significant concentrations of stress and makes dangerous areas of cracks or plastic deformations origination. Their load bearing capability in most cases is determined sue strength or buckling conditions in areas of stress concentration. Other kinds of violation of regularity include breaks occurring in corrugated and multi-wave coverings. By their impact on stress state they are similar to ribs.

Various rigid inclinations as reinforcement rods also violate the regularity.

The areas of stress concentration in places of violation of regularity (end of rib, discrete constraints) make significant impact on load bearing capability and buckling of thin-walled structures. At this the known traditional analytical and numerical methods for research of mode of deformation of ribbed thin-walled structure are less effective.

Due this makes the necessity to develop the new effective methods of solution of mentioned class of tasks. Intensively developing in last year's theory of generalized functions, in particular, discontinuous impulse functions significantly extend possibilities of calculation of thin-walled structure with various violations of regularities.

The multi-layered systems also would considered as systems with violation of regularity on thickness.

Sandwich panel with lightweight filler and two external load bearing layers would be widely applied in construction of residential and public buildings as prefabricated typical component, in that accordingly of its destination would be various structural features and additions as additional constraints, holes for openings, windows and doors [8-10].

The method of analysis of sandwich planar elements with cuts and holes on strength and stability is not sufficiently developed. Practically are not exist relatively simple and convenience for engineering calculation formulae for definition of critical compressive force with taking into account various structural features, including the values and arrangement of mentioned above weakening.

The development of basics of sandwich panels theory was originated from 30-ies of XX century. The scientific literature related to sandwich plates and shells includes several thousand papers. The detail information on this isssue are stated in articles [1, 2, 3].

In lot of papers [4, 5, 6, 7, 8, 9, 10, 11, 12, 13, 14] are stated approximation and refined

methods of analysis of weakening by arbitrary arranged on contour cuts and apertures stability of sandwich plates and shells. Based on tehm is possible to develop practical recommendations for analysis and design of widely implemented in construction practice sandwich precast panels.

Rather this is reason for creation of versatility of variants of sandwich plates and shells theory. Number of published in this field papers is increasing every year.

Are proposed the new theories that are grounded on various approaches of their creation. Considered by us task also is based on general dependencies of theory of elasticity and structural mechanics.

2. Methodology

Let's consider the mode of deformation of sandwich plate under action of concentrated force *P*.

In the case of lightweight filler the decisive differential equation of bended plate with respect of deflection function will be as [15]

$$2B\left(h+\frac{t}{2}\right)^2\Delta^2 W+\left(1-\frac{Bh}{G\Delta}\right)(2D\Delta^2 W-P)=0. \quad (1)$$

Then if we assume the external loading function $P(x,y)$ as

$$=P\ (x-x_i)\delta(y-y_i), \quad (2)$$

where x_i, y_i–are the coordinates of point of force application, the equation (1) lets represent as

$$2B\left(h+\frac{t}{2}\right)^2\Delta^2 W+\left(1-\frac{Bh}{G\Delta}\right)(2D\Delta^2 W-P)=$$

$$=P\left[\delta(x-x_i)\delta(y-y_i)\frac{Bh}{G}\delta''(x-x_i)\delta(y-y_i)-\right.$$

$$\left.-\frac{Bh}{G}\delta(x-x_i)\delta(y-y_i)\right]. \quad (3)$$

As result of sinus transformation on coordinate of *y* this equation will be as

$$C\left(\frac{d^2}{dx^2}-\beta_k^2\right)^2\psi_k+F\left(\frac{d^2}{dx^2}-\beta_k^2\right)^2\psi_k=$$

$$=A_k\delta(x-x_i)+B_k\delta''(x-x_i), \quad (4)$$

where are introduced the designations

$$C=2\left[B\left(h+\tfrac{t}{2}\right)^2+D\right].$$

$$F=-\frac{Bh}{G}\cdot 2D;$$

$$A_k=\frac{2P}{b}\left(1-\beta_k^2\frac{Bh}{G\Delta}\right)\sin\beta_k y_i;$$

$$B_k=-\frac{Bh}{G}\sin\beta_k y_i;$$

$$\beta_k=k\frac{\pi}{b}.$$

The characteristic equation

$$C(z^2-\beta_k^2)^2+F(z^2-\beta_k^2)^2=0 \quad (5)$$

has the real and multiple roots

$$z_1=-z_2=\sqrt{\beta^2-\frac{C}{F}}$$

$$=\sqrt{\frac{\beta^2 G}{h}\left[\frac{1}{D}\left(h+\frac{t}{2}\right)^2+\frac{1}{B}\right]};$$

$$z_3=-z_4=\beta;\ z_5=-z_6=-\beta. \quad (6)$$

Due this solution of equation (4) will be written down as

$$\psi=C_1 chz_1 x+C_2 shz_2 x+C_3\beta\quad h\beta+C_4 ch\beta\ +$$

$$+C_5\beta\quad h\beta\ +C_6 sh\beta\ +\psi^*. \quad (7)$$

Let's apply for determination of partial solution ψ* the method of variation of integration constants and represent the ψ* as convolution

$$\psi^*=\frac{1}{d}\int_0^x\Delta(x-\eta)[A\ (\eta-x_1)+$$

$$+B\ ''(\eta-x_1)]d\ . \quad (8)$$

The determinant d is reduced to expression $d=32z\beta^4(z^2-\beta^2)^4$; the determinant $\Delta(x-\eta)$ will be as

$$\Delta(x-\eta)=$$

$$=\begin{vmatrix} 1 & 1 & 1 & 1 & 0 & 0\\ z & -z & \beta & -\beta & 1 & 1\\ z^2 & z^2 & \beta^2 & \beta^2 & 2\beta & 2\beta\\ z^3 & -z^3 & \beta^3 & -\beta^3 & 3\beta^2 & 3\beta^2\\ z^4 & z^4 & \beta^4 & \beta^4 & 4\beta^4 & 4\beta^3\\ e^{z(x-\eta)} & e^{-z(x-\eta)} & e^{\beta(x-\eta)} & e^{-\beta(x-\eta)} & (x-\eta)e^{\beta(x-\eta)} & (x-\eta)e^{\beta(x-\eta)} \end{vmatrix}$$

and after some transformations let's represent it as:

$$\Delta(x-\eta)=\frac{1}{2}z\beta^2(\beta^2-z^2)^2[2\beta^3 shz(x-\eta)$$

$$-z(3\beta^2-z^2)sh\beta(x-\eta)+$$

$$+z(\beta^2-z^2)\beta\quad h\beta\ ].$$

Then the expression ψ* after some transformations let's write down as:

$$\psi^*=\frac{1}{2}z\beta^2(\beta^2-z^2)^2\int_0^x[2\beta^3 shz(x-\eta)-$$

$$-z(3\beta^2-z^2)sh(x-\eta)$$

$$+z(\beta^2-z^2)\beta(x-\eta)\cdot$$

$$\cdot ch\beta(x-\eta)][A\ (\eta-x_1)+B\ ''(\eta-x_1)]d\ , \quad (9)$$

or with taking into account of filter property of δ function

$$\psi^*=\frac{1}{2}z\beta^2(\beta^2-z^2)^2[2\beta^3 shz(x-x_1)-$$

$$-z(3\beta^2-z^2)sh\beta(x-x_1)+$$

$$+z(\beta^2-z^2)(B\beta^2 2sh\beta(x-x_1)+$$

$$+\beta(x-x_1)ch\beta(x-x_1)]H(x-x_1). \quad (10)$$

It is clear that substitution of (10) and (6) in the equation (4) transform it in the equality. The diagrams of function ψ and it's derivatives for squared hinged supported on contour pate are presented on the Fig. 1. The function ψ itself represents the rather smooth, regular, but the second derivative from it has the break, and the third – the jump that corresponds to character of distribution of bending moment and shear force.

Due this the trigonometric series for moments and shear forces have the rather rapid convergence in all considered domain, including the adjacent of point of application of concentrated force (practically are enough 2-3 terms of series).

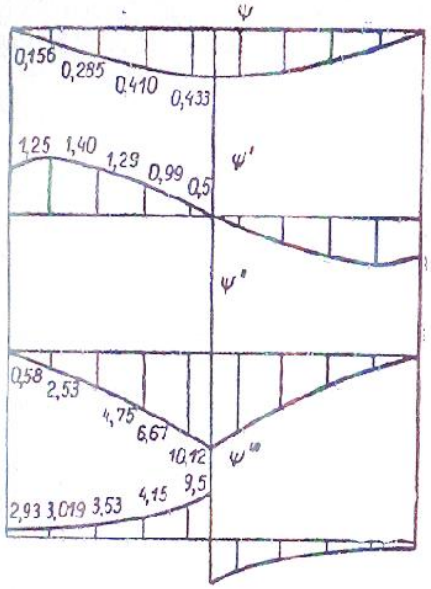

Fig. 1. Diagrams of function ψ and its derivatives corresponding to character of distribution of deflections, angles of turn, moments and shear forces in sandwich plate from concentrated force

It is necessary to mention that derivatives from ψ have the less outlined singularities then according derivatives from deflection function in continuous pate at the same loading. This would be explained due obviously compliance of middle lawyer.

At implementation of applied by us sinus transformation on coordinate *y* the constructed function ψ reflects the discontinuous character of forces and moments in the direction of coordinate x, and in cross-section, in parallel to axis y, the stressed states is more smooth. For obtaining of functions, having similar singularities in two perpendicular directions that corresponds to physical sense of task, would be generated the function accordingly of method [16] as series

$$\varphi = \sum \varphi'_k \psi_k \ \psi_k \ , \quad (11)$$

by determination of coefficients φ_k from equation (4) by variation method of Bubnov-Galerkin. The formula (110 is advisable to apply for study of stressed state in the minor domain of application of concentrated force

3. Conclusion

Is developed the algorithm of finding of discontinuous functions determining the mode of deformation of bending sandwich plate under the action of concentrated force.

The series for determination of forces and moments compiled from discontinuous functions, have rather rapid convergence in the all considered domain, including in adjacent of singular point and gives the possibility to effectively carry out the strength and deformability analysis of elements of machines and devices as sandwich plates with point supporting.

Stated in the article method would be successively developed on analysis of multilayer orthotropic plates and depressed shells

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ERVAND KOCHAR'S RELATIONS WITH GEORGIAN ARTISTS AS REFLECTED IN THE CONTEXT OF ARCHITECTURAL MODERNIZATION OF ERVAND KOCHAR MUSEUM

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Keywords:museum, museum architecture, cultural heritage.

Abstract

Under current socio-political conditions when globalisation influences the human activities, i.e. economy, politics, education and other sectors on a daily basis, endangering the cultural and ethnic frontiers, the existence of mother tongues and individual identities, impacting the spiritual and moral shaping and development of a person, the issues of awareness and valuation of national characteristics gain higher importance in public relations. Museums, being centres for preserving and introducing the memory of humanity – cultural heritage – are a special instrument for highlighting these issues.

The article presents the Museum of the artist and sculptor Ervand Kochar (founded in Yerevan in 1984) which is among the most important research centres in the region on the historical *Avant-garde*.

The Museum, like most museums in Armenia, needs architectural modernisation.

The Museum has developed an architectural modernisation project which will enable to match the Museum with modern requirements: have a screening and lecture hall, rooms for eduactional activities and master classes, repositories with controlled storage environment, a souvenir shop, and a recreation and leisure zone, including a snack bar.

The current area of the Museum covers 249 sq m, the usable area comprising 208 sq m with six display halls where, in addition to the permanent exposition, events of various nature are being organised for schoolchildren, students, tourists, people with restricted abilities and pensioners.

With expansion and development work in the yard, the second floor of the Museum will be joined with the second floor of the storage located in the yard. As a result the Museum will obtain an addition area of 83 sq m, passages with modern architectural solutions connecting the old and new buildings and an additional area with alleys for thematic display of exhibits matching the nature of the Museum will be created.

Expanding the area of the Museum will enable to present to the public all of the creative periods of Kochar in a broader and comprehensive manner, with original exhibits, which is currently performed through modern architectural, design and technological solutions.

Mobile panels with new types of design solutions display Kochar's first creative period (Tiflis, birthplace of Kochar), his exhibition activities, relations with the prominent Georgian artists Lado Gudiashvili, David Kakabadze, Elena Akhvlediani and others, as well as the Georgian-Armenian cultural environment. These relations were never discontinued: on the contrary, they grew even deeper and continued both in Europe and after Kochar's return to Armenia (1936).

Popularising the creative heritage of the great artist, the Museum expects to continue shaping intercultural communications and relations and the further development of co-operation.

1. Introduction

Urban architecture greatly influences the shaping of people's aesthetic perceptions. The interaction between man and architecture starts well before the visitor finds himself on the museum grounds. Before he walks in, he is directly impacted by the architectural structure, its exterior and the adjacent area which create new criteria for perception, submerging man in the world of culture. Thus, architectural art becomes an instrument for pan-cultural communication [1].

With time, museum-related activities acquire a greater socio-cultural significance: the role of the museums grows in terms of enhancing the educational level of the public, preservation and interpretation of the cultural heritage, cultural identification and organising leisure.

Today major museums manoeuvre between culture and attraction: marketing strategies are being developed to attract visitors, hit exhibition are being organised, with the tickets bought months before the event [2].

2. Case History

The establishment of Soviet power in Armenia marked a true rise of museology, accounted for by

the new principles of organisation of the sector, adoption of theoretical bases for designing dedicated museum buildings, shaping of memorial museums, adaptation of some structures to museums, etc. The buildings of most, if not all, museums currently operational in Armenia are non-typical historical and architectural heritage from the Soviet times. In view of this, it must be admitted that, despite partial reconstructions, the nature of architectural solutions of the majority of these buildings, as well as their furnishings do not enable a full-scale activity of the museum.

The Muesum of the the artist and sculptor Ervand Kochar (1899-1979) is located in Yerevan, Armenia. He was born in Tiflis (now Tbilisi), finished Nersissyanschool, studied at O. Schmerling Academy of Sculpture and Fine Arts. Kochar continued his education in Moscow, and lived and created in Paris between 1923 and 1936. He grew into an artist who had a significant influence on the development of world fine arts by creating a new plastic form of expression (*Peinture dans l'espace*) which includes time as an extra fourth dimension.

The European art critics considered Kochar as one of those "pioneers who, within a few years, transformed the perceptions about modern art and overturned the world..."»[3].

In 1936 Kochar immigrated into the USSR and was never again entitled to travel abroad.

Ervand Kochar Museum was founded in 1984 on the location of Maestro's studio, and today is among the most important research centres in the region for the popularisation of the historical avant-garde (**Fig. 1**).

Fig.1Ervand Kochar Museum – 5th Hall

3. Methodology

Currently all of the museums in the RoA are attempting to combine the architectural objectives with the development of cultural tourism, application of modern technologies and the shaping of the concept of the exposition plan of museums through the criteria of modern museum design.

Modern museum programmes help the visitors to transition from the state of a passive observer to the active state of creative potential and associative thinking [4].

Through the application of state-of-the-art technologies, as well as by carrying out open-air cultural and leisure events, Armenia's museums, including Ervand Kochar's Museum, are attemptingto comply with the international criteria (**Fig. 2**).

Fig.2 Open-air events, organized by E. KocharMuseum

4. Results

The current area of the Museum covers 249 sq m, the usable area comprising 208 sq m with six display halls where, in addition to the permanent exposition, events of various nature are being organised for schoolchildren, students, tourists, people with restricted abilities and pensioners(**Fig.3**).

Fig. 3 Events of various nature, organized in E. Kochar Museum

The exposition of the Museum is a complete system of space-and-objects where the museum values, objects and other exhibits are united around the complete scientific and artistic idea and concept.

The artist's Tiflis creative period and his exhibition-related activities are presented in a special hall. Kochar held abundant communications with remarkable Georgian artists. Letters from Gudiashvili, D. Kakabadze and E. Akhvlediani, as well as Kochar's correspondence with the Georgian-Armenian cultural environment, photos and other rare evidence are kept in the archives of the Museum(**Fig.4**).

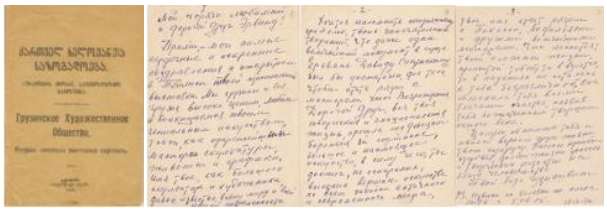

Fig.4 Documents on relations with Georgian artists.

Here is an excerpt from the letter of Lado Gudiashvili to Kochar [5]:

...We, the Georgians, and all of Georgia hold your art of a genius and you as a highly gifted master of sculpture, painting and graphics, in high esteem, love and worship. Your name as a great sculptor and artist has been known to the world for a long time... I wish to remind those who are watching your wonderful compositions today, that the marvellos monument to David of Sassoun alone would suffice to put you among the Masters of the Renaissance...

Lado Gudiashvili

High-value quality prints are featured in a special showcase through mobile panels: their originals are kept in other national museums and private collections **(Fig.4)**.

Films interpreting Kochar's biography and works are regularly shown on the screens in the display halls of the Museum; virtual expositions are organised,"inviting" the visitors to an interactive cooperation **(Fig.5)**.

Fig.5 Interactive programs

The Museum has developed an architectural modernisation project which will enable to match the Museum with modern standards. With expansion and development work in the yard, the second floor of the Museum will be joined with the second floor of the storage located in the yard. As a result the Museum will obtain an addition area of 83 sq m, passages with modern architectural solutions connecting the old and new buildings and an additional area with alleys for thematic display of exhibits matching the nature of the Museum will be created. As a result, the Museum will have a screening and lecture hall, rooms for educational activities and master classes, larger repositories, a souvenir shop, and a recreation or leisure zone.

5. Conclusion

The architectural, design and technological approaches applied for the organisation of the Museum's premises will enable to present in detail all of the creative periods of the artist, his exhibition-related activities and relations with artists of world renown.

In this era of globalisation when the idea of preservation of cultural diversity becomes primary, and globalisation is perceived a s a danger for national traditions, local customs, beliefs and values, Ervand Kochar Museum attaches importance to the formation and development of cross-cultural communications and relations with the expectation of further co-operation, which will result in that museums will become centresfor education, cultural information, cultural heritage and popularisation of the country, thus promoting the dissemination of creative novelties and expansion of communications between the international communities [6].

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TRADITIONS AND INNOVATION IN THE ARCHITECTURE OF VASILI MIRZOYAN'S RELIGIOUS STRUCTURES

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Abstract

The list of built structures of one of the most outstanding representatives of Armenian architectural elite of modern time, civil engineer Vasily Mirzoyan in the territory of the former Yerevan province includes the entire catalogue of architectural and engineering objects. This registry includes and religious buildings, a variety of which are characterized not only by the sense of architectural shaping, but religious directions of the current in the region: Apostolic, Orthodox and Islamic. In addition to construction of new facilities, the architect executed several reconstructions, which provided modern architectural sound to old buildings.
The article is an attempt to identify the characteristics of the creative style of the architect on the basis of archival and bibliographic materials and direct survey. In this connection, the specifics of religious buildings in the context of the preservation of traditional structures, the impact of the artistic world outlook epoch and new style-forming motives are analyzed.
The study of the architect's projects allows us to conclude, that principles of all religious buildings the architectonic of traditional forms is preserved and modifications are correlated with certain ideological and artistic presuppositions left its mark on the image of the structures.

1. Introduction

Vasiliy Mirzoyan (1863, Tiflis - 1925, Yerevan) is one of the greatest architects of 19 - early 20 century, he is the Knight of the Order of St. Stanislav 3rd and 2nd degree, St. Anna 3rd degree, "Lion and Sun" 3rd degree. He began his professional career in 1892, the year after graduating from the St. Petersburg Institute of Civil Engineers. A list of his works includes all types of public, residential, industrial and engineering structures. He is the author of the administrative and religious buildings, schools, banks, mansions, apartment houses, factories, bridges, roads in Yerevan, the towns and the villages of the province [1].
Strength, Utility and Beauty (firmitas, utilitas, venustas) - the basic laws of architecture, are combined in the form-building principles of all the works of V. Mirzoyan. The indisputable presence of traditional architectural forms, substantiated by regional factors and reflecting national artistic images, their harmonious combination with modern methods of form creation, expressed in historical styles and Art Nouveau, and rational solution of the problem - so generalized it is possible to characterize the priorities of the architect's worldview.
Among the religious buildings designed by V. Mirzoyan are apostolic churches, chapels, bell towers, orthodox churches, mosques. Despite the unconditional difference between the form-building concepts in the architecture of different denominations, the works of the architect are united by the expressed correctness in relation to the construction, traditional forms and artistic decorations.
The purpose of the study is to identify the main features of the creative handwriting of the architect used in architectural design of the apostolic church, and analyze the interaction of author's innovation and traditional national forms of Armenian church architecture.

2. Methodology

The article is written on the base of architectural analyze of archival and literary graphic and written materials andfield studies.

3. Results

Substantiation of methods of analysis and key initial data that determined the view of the research direction were two documents of the 1900s: Mirzoyan's written technical expertise of

the church project in the village Ortakilis of the Alexanderapol region (1905) and his own project for the reconstruction of the Simeon Tseruni(St. Simeon the Elder, 1901) church in Nork district of Yerevan.

The first document contains quite definite theses of the architect concerning the criteria, which, in his opinion, should be followed in the construction of the apostolic church. The dating of the document in 1905 clearly shows that by that time, Mirzoyan certainly had accumulated practical experience in the construction of confessional buildings: he completed the projects of the Cemetery Church (1900) and the belfryat the St. Grigor Lusavorich Church (1902) in Yerevan, including the above-mentioned reconstruction, a number of churches in the former Nor Bayazet district - in the cities of Gavar (1902), Vardenis (1900), in the villages of Shorzha Verin (1903), Averak (1904), etc.

The originality of the second document, from the point of view of studying a specific design methodology, is that plans, facades and sections of old and new churches are combined in the drawing, and thus all the changes made are graphically illustrated. Comparative analysis of the two solutions makes it possible to clarify the world view of the Mirzoyan concept of style, which, in his professional notation, corresponds to the architectural image of the apostolic church.

After studying the project of the Church in the official capacity of the Provincial Architect of the Construction Department of the Provincial Government, he noted the following in the protocol: "... the mentioned project cannot be considered satisfactory in mind next: plan form (correct rectangular) very unsuccessful for Church architecture. Neither the facades nor the holes correspond to the style of the Armenian-church architecture, in which style (judging by the details), suppose so designed the building. Frames of the windows, albeit stylish, but in this case a very cumbersome. Also it should be noted concerning the entrances... Rafters for a five-sazhen baywith one headstock, despite the ingenious reduction of span by brackets, cannot be considered sufficiently strong. It should be noted that with a building volume of up to 250 cubic sazhens - a useful area of only 35 square sazhens. From all this we must conclude that with a more rational distribution of the same quantity of materials, it is possible to obtain a construction according to plans and facades more appropriate to its purpose [2, file 393, sheet 5].

Thus, proceeding from the verbal description (the project archive was not detected, possibly because of the seizure of the Construction Department, and only corrected, which was approved), the main requirements are as follows:

A. Construction, the question of the strength of structure;

B. Economics, rational use of building materials, matching the volume of construction usable area;

C. Plan, the choice of the outline drawing, in which the functional divisions inherent in the church architecture are expressed;

D. Style, artistic design, the correspondence of architectural elements and decorations to the peculiarities of the Armenian church architecture, in which harmony in the ratio of fragments to the whole is always preserved.

The new version of the church (K. Leontiev, 1905) [2, file 408, sheet 4] is a three-nave basilica, with four cross-shaped in plan pylons, with a middle nave twice as wide as the lateral ones. The apse with a five-sided volume protrudes on the eastern facade, a semicircle is drawn inside, passing at the top in the conch. The square in the plan belfry is placed along the main longitudinal axis in front of the western facade, between the bell tower and the portal there is a covered passage to the church. In the volumetric-spatial composition, the gable roofs of the central nave and the waste from the median square two transversal spans form a cross. The vertical dominant of the composition is the eight-column rotunda crowned with a dome towering above the main body of the church. The analysis of the revised project allows us to conclude that, first, the author took Mirzoyan's recommendations into account, and, secondly, it can be seen that parallels with the Myzoyans' churches of Nor Bayazet region, which will be discussed below, are visible in the structure of the plan and separate fragments of the new church.

The graphic material of our research is the old stone church of Surb Simeon [2, file 123, sheet 2], which was destroyed by an earthquake in 1679. According to Karo Ghafadaryan, from the bishop of Hovhannes Shahatunyants, who saw this building in 1840, the neglected church was left without a dome, but functioned. In the middle of the 19th century the church was demolished and a new one was built in its place K.Ghafadaryan notes the two inscriptions found on the church: the first on the south wall of the northeastern aisle, dated 1885 and indicating the construction of the bell tower, the second, in 1886, an epitaph on the western outer wall, dedicated to Shoghakat

Harutyunyants (the church was demolished in the 1930s) [3].

The old plan is a rectangle with a 4/3 sides ratio, with an apse inscribed in it and two lateral eastern side altars. The eastern part is compositionally separated from the central hall with four cross-shaped pylons, forming a visual square by their arrangement. The central nave faces the eastern and western façades with a triangular gable common to the whole wall; the gables of the southern and northern facades marks the position of the transept and continues with horizontal cornices. Two six-columned dome rotundas are placed on a longitudinal ridge. Windows and portals are arched.

In confirmation of what was said in the text of the report described above, Mirzoyan refuses the laconic silhouette of the rectangle in the outline of the plan and with minimal means, adding avant-corps, animates the smooth walls of exteriors. As a result, this emphasizes the position of the apse and the cross-forming volumes. Unlike the previous through small rotundas, a new massive drum and dome subordinate to itself the entire composition of the voluminous sections of the building. The height of the drum with the dome is determined by the golden section with relation to the main massif, the point of division of the two lengths is the ridge. Window openings are reduced to one width and are greatly extended, in proportion 1/6, instead of the old 1/2. Three facades, except for the east, are decorated with expressive three-arched windows under a common three-blinded arcade (Fig. 1).

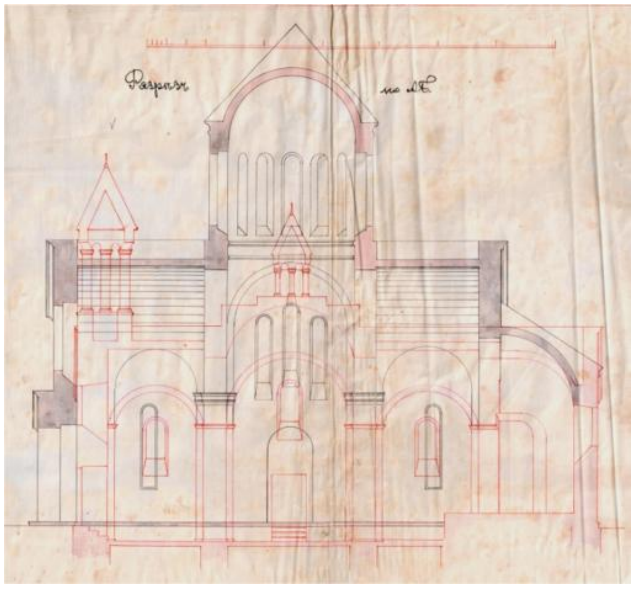

Fig. 1.The Church of Surb Simeon in Yerevan

It cannot be said that the choice for the study of the next type of church was justified by a detailed classification of all Mirzoyan's church works or their indisputably recognized special status in the list of these buildings of 19 and the beginning of the 20th century in Armenia - in fact, special information about them is limited. But their type is original, the structure that determines the monumentality of the architectural image, is of great interest, besides the existence of old projects allows analyzing on the basis of author's, objective materials.

Drawings and documents of the Construction Department of the Yerevan Provincial Government tell about the state of affairs connected with the construction of churches in one of the region of the province - Nor Bayazet, occupying the coastal area around Lake Sevan, in the north-eastern part of the province. Many documents of the 1900s contain petitions for permission to build a church, for the money of parishioners, since there is no such church in the village or the existing one is in a dilapidated state. The different in their architectural characteristics projects are attached to the cases, including Mirzoyan's works that we are interested in. Among the latter, one type deserves attention, the definition of which corresponds to the description of the Echmiadzin temple (4-19 centuries) by H. Khalpakhchyan: "The cupola`s abutment, cross-shaped in plan, are connected with each other and with the walls by arches underlying the walls... the apses are crowed with conchs. The arrangement of the ceilings at various levels causes the interior to taper off to the central dome". In addition, a specific fact is a "three-tiered belfry were built, the latter in front of the western entrance to the Cathedral" [4].

The architectural images of churches in the cities of Gavar, Vardenis [2, file 96, sheet 239], in the village of Averak [2, file 171, sheet 2], which are suitable for this description, are identical in their general design. Their differences determine the drawing of forms, elements, proportions of plans, dependent on the factors of a particular situation, the dimensions of the building and its position in the environment, respectively, affecting the emotional perception. Projects of the last two churches, for example, differ only in size and proportions in the composition of plans.

All churches are cross-shaped with a central dome. The drawing of the plan is extended along the longitudinal axis. Four cross-shaped in plan columns divide the interior space of the

rectangular hall into three naves, the middle of which is two times wider than the side. But this proportion is present only in the four corners of the hall, because the central nave and the transept are elongated and extend beyond the main rectangle. Due to this reception with protruding ends along the longitudinal and transverse axes of the building, the volume cross, characteristic of Armenian and Christian architecture, is also expressed in terms of.

To the east there is a pentahedral apse, semi-circular in the interior and completed by a conch, with two rectangular side-chapels adjoining it. The southern and northern avant-corps, on the width of the transept, continue in the voluminous portals. Square in plan, the belfryat the bottom has the form of a portico, one-arched on the facades, which is open on four sides: it communicates with the western central portal by a covered passage.

The top of the dome of the church - the highest point of the building, coupled with the monumental forms of the decorated with an arcade of the rotunda of the drum, determines the massive dominant of the composition. The second dominant is a graceful vertical of a three-tiered, tapering bell tower, also ending in a arched rotunda with a dome.

The favorite motif of the architect is a three-arched window with an overstated aperture of the middle, with a trefoil arch at the top. They are diverse in construction, as, for example, the position of the lower horizontal of the central aperture and the decorative design vary from simple holes on a clean wall to artistic composition in an arched niche.

Fig. 2. The Church of Surb Astvatsatsin in Vardenis

All the structural elements, functionally justified division of the internal space, horizontally and vertically protruding fragments are reflected in the volumetric construction of the structure by the variety of constituent forms. The richness of architectural forms, which are united by hierarchical subordination, create a virtuoso in its perfection aesthetic image of the building, which reflects the professional skills of the architect and the traditions and quality inherent in the architecture of Armenian churches (Fig. 2).

Identical methods of creating the form Mirzoyan uses in the projects of chapels and small churches, but the ability of the function to dictate the nature of the structure has determined a different, intimate image of such buildings.

These include the Cemetery Church (the length of the side is 12.1 m, the height of the building to the base of the cross is slightly more than 18 m) [5]. The Greek cross expressed in the planning composition rises above the reduced other masses of the chapel. The facades of the wings continue the roof with two sloping surfaces to triangular gable with profiled cornices. Western, southern and northern facades with the entrances to the church, designed the same: porch, semicircular portals, framed by perspective arches, relief crosses. The eastern wall decorates the composition of the three-arched window. Other windows are also narrow but low.

The conjugations of planes in outer corners are fixed by engaged columns, different in height, but similar in their interpretation - with identical globular bases and capitals. Dominant of the central-axial composition of the building is a 12-bays rotunda, formed by a perimeter arcade on which the pyramidal dome rests. In the proportions of the arcade uniform metric division with equal dimensions of bay and pier, a ratio of the height of the column to the diameter is 8/1.

4. Conclusion

Thus, on the basis of the analysis of the projects of the church buildings of Vasily Mirzoyan and their architectural and planning decisions, one can make a description of the architect's creative outlook regarding the architectural features of the apostolic churches:

A. Traditional methods of Armenian art;

B. The rational interrelation of the plan and the volumetric-spatial composition, the desire for dismemberment of the architectural masses;

C. Harmonious balance, the conformity of the architectural image of a function, grandeur of monumentality or intimacy;
D. The combination of the two components, the church and the belfry into a single whole and the complexity of the composition;
E. In the plan of small churches, chapels, belfries, the desire for a square with fixed angles;
F. Hierarchical structure of the composition, organized by different levels and different sizes collected around the vertical dominant of the dome rotunda;
G. Vertically elongated arched windows, solitary, decorated with an blind arch and three-part, with an average overstated arch and a trefoiled blind arcadeon the top;
H. The use of proportions of the golden section and simple mathematical ratio;
I. Constructive system of arches and pendentives;
J. Symbol of the cross in the volumetric-planning composition.

The number of churches built by Vasily Mirzoyan is difficult to count, and their architecture, despite the general concepts of construction, is diverse. The article revealed only some of the questions contained in the title, so the need for further research is obvious.

5. Acknowledgements

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DEVELOPMENT OF ENGINEERING METHODS FOR CALCULATING COMBINED PILE-RAFT FOUNDATION (CPRF)

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Abstract

This paper reviews performance of combined pile-raft foundations (CPRF). The goal of the study is obtaining the ability to design solid, reliable and cost-effective foundations allowing for minimum settlements. The task is to develop methods for calculating CPRF for optimum choice of the foundation raft size, dimensions and number of piles. To achieve these results, an analytical method was used, curves of the ratio of CPRF settlement to raft foundation settlement vs. total load *N* on the piles were plotted.

The scientific significance of the study is in the method of engineering calculation of CPRF, which is better correlated to the experimental data and allows optimal distribution of loads between the raft and piles, ensuring reliability and operational integrity of buildings, reduction of settlements and the number of piles, which would lead to cost savings and reduced consumption of materials for foundations. A new coefficient showing the residual stress has been introduced, and a method for its calculation has been proposed. A new equation for determination of the share of total load *N* on the piles has been obtained. This equation is more general in comparison with the previously generated equations. It allows excluding the assumption that the stresses arising under the raft completely dissipate at the level of the pile end.

Application of the developed methods will increase reliability and reduce settlements and the number of piles, which ultimately will lead to cost savings and reduced consumption of materials for foundations.

1. Introduction

Combined pile-raft foundation (CPRF) is a structure comprising a reinforced-concrete raft foundation and piles. The raft can be either constant or variable in thickness, and the piles can be either constant or variable in length, diameter and spacing [1, 2]. Methods of connection between the piles and the raft can also be different, e.g. rigid or hinged connection or one-sided contact support [3, 4, 5].

The structure is called combined**,** since the work of its elements is mutually complementary to each other through the foundation soil medium. This is manifested in partial transfer of the building weight at the upper level (under the raft bottom), allowing for increase in friction forces in the upper part of the piles [6], which, in its turn, leads to increase in the maximum load on a pile, while the raft allows the foundation remaining reliable even after the "failure" of a pile. This agrees with the emergence principle of the systems theory, stating that it is typical of a system to acquire properties of constituent components, as well as new properties.

The grounds stated in the article [7] are based on two basic provisions for comparison of pile foundations (PF) and raft foundations (RF):

a) Pile foundations are more expensive than raft foundations.

b) Pile foundations are more rigid.

Combined pile-raft foundation (CPRF) is assumed to be an intermediate option between PF and RF in terms of settlements, rigidity and costs.

Regulatory documents adopted in Russia, containing basic rules for design of bases and foundations [8, 9, 5, 10] do not consider the possibility of using piles in the breakdown zone, i.e. after their "failure". For this reason, the existing documents allow for transfer of no more than 15% of the total weight of the building via the raft to the base [8, 10], which rather corresponds to the classic pile foundation according to the classification [11], which properly accounts for the actual deformability of friction piles and the ability of the raft foundation to take up some load. The EU regulatory document on regulation of geotechnical design [12] classifies combined pile-raft

foundations as foundations of Category 3. This means that standard provisions stated in this Eurocode are not applicable to them, and the design procedure should include alternative and specially developed rules and measures.

2. Methods

The task is to develop methods for calculating CPRF for optimum choice of the foundation raft sizes, dimensions and number of piles, so that settlements would not exceed the ultimate values. The most cost effective and safe number of piles is the number which would provide settlements under the ultimate values. Load and settlement values of PF and RF have been determined for high-rise buildings under load *N*, with the assumption that the first settlement s_{pf} is below and the second settlement s_{rf} is above the ultimate value for the building. Thus, three foundation models are considered —pile (PF), raft (RF) and combined pile-raft (CPRF) — under the same conditions and the same static load *N*.

The following assumptions were introduced in the work [7]:

1. Bending and torque moments and transverse forces acting on foundation structures are considered negligible.

2. Settlement of piles under load is bilinear, i.e. the value of settlement is directly proportional to the corresponding load before achievement of the design load-bearing capacity; subsequent deformations develop without changes in internal forces of piles.

3. The raft foundation does not resists unless some ultimate load is achieved; all loads from the structure on the base are distributed among the piles. Developing settlement involves overlying soil layers (including the near-pile soil body and the structure as a whole).

4. When the ultimate load-bearing capacity of the piles is achieved, the piles slip along the shaft in relation to soil, and raft foundation resistance appears. It also takes up subsequent additional loads.

5. Deformability of the pile shaft is considered negligible.

Apart from clearly stated assumptions, there is a number of issues that are implicitly omitted: reliability issues (reserve factors), variability in characteristics of construction materials, soils and loads. This is due to the fact that the proposed method [7] does not imply final calculations for such a complex system, but it makes it possible to estimate the required number of piles in the first approximation with the help of a simple engineering calculation.

Analytical calculations performed for three models, taking into account the above assumptions, allowed obtaining an equation of the optimum share of loads from the structure, distributed by piles in CPRF, depending on the total load from the structure, in the work [7]:

$$\alpha_{cprf} = \frac{s_{rf} - s_u}{s_{rf} - s_{pf}} = \frac{\delta_{rf} - 1}{\delta_{rf} - \delta_{pf}} \qquad (1)$$

where α_{cprf} is the share of total load *N* on piles; s_u is ultimate settlement; s_{pf}, s_{rf} are total settlements of pile and raft foundations, respectively; δ_{pf}, δ_{rf} are relative settlements of pile and raft foundations.

Equation (1) was obtained after mathematical transformation, during which total settlement of pile and raft foundations s_{pf} and s_{rf} were divided by the ultimate settlement s_u, and relative settlements of pile and raft foundations were obtained. This conclusion implied an implicit assumption that the vertical stresses due to pressure transmitted along the raft bottom dissipated at the level of the pile bottom end. This assumption can be valid if the length of piles is much larger than the size of the foundation in plan, and the vertical stresses are significantly reduced.

It would be reasonable to assume that there is an opposite extreme assumption that the additional vertical stresses under the raft and at the level of the pile ends are equal. This assumption most closely describes the case when the length of piles is significant in comparison with the characteristic size of the raft foundation. As is the case in [7], taking into account the opposite assumption, we obtain the following:

$$k_{pf} = \frac{N}{s_{pf}} \qquad (2)$$

where k_{pf} is rigidity of pile foundation; *N* is the load on CPRF.

$$k_{rf} = \frac{N}{s_{rf}} \qquad (3)$$

where k_{rf} is rigidity of raft foundation.

The total load on CPRF can be determined when summing up the loads transmitted onto the piles and the raft:

$$N = N_{pf} + N_{rf},$$

where N_{pf} is the load applied to the piles; N_{pf} is the load applied to the raft.

The load N_{pf} is determined as follows:

$$N_{pf} = \alpha_{cprf} N \quad (4)$$

This being the case, N_{rf} is determined as follows:

$$N_{rf} = (1 - \alpha_{cprf}) N \quad (5)$$

If the additional vertical pressure due to the load at the level of the pile bottom ends does not decrease due to pressure transmitted along the raft bottom, then the soil under the piles is loaded with the sum of forces N_{pf} and N_{rf}. Thus, the pile component of settlement $s_{cprf,1}$ (due to soil shrinkage below the pile ends) is equal to:

$$s_{cprf,1} = \frac{N_{pf} + N_{rf}}{k_{pf}} = s_{pf} \quad (6)$$

The second component of settlement (due to soil shrinkage under the raft, following the "failure" of piles) is equal to:

$$s_{cprf,2} = \frac{(1 - \alpha_{cprf}) N}{k_{rf}} = (1 - \alpha_{cprf}) s_{rf} \quad (7)$$

The total settlement of CPRF equals to the sum of the following components:

$$s_{cprf} = s_{pf} + (1 - \alpha_{cprf}) s_{rf} \quad (8)$$

If we assume that CPRF settlement equals to the ultimate settlement s_u, then α_{cprf} can be calculated from equation (8):

$$\alpha_{cprf} = \frac{s_{rf} + s_{pf} - s_u}{s_{rf}} = 1 - \frac{s_u - s_{pf}}{s_{rf}} = 1 - \frac{1 - \delta_{pf}}{\delta_{rf}} \quad (9)$$

Equation (9) allows determination of the load share taken up by the piles.

2.1. Comparison with experimental observations

In the work [2], materials on systematization of the monitoring results with regard to various objects constructed with CPRF are presented. Regularity of changes in the ratio of CPRF settlement to RF settlement $\frac{s_{cprf}}{s_{rf}} = \frac{\delta_{cprf}}{\delta_{rf}} = \frac{1}{\delta_{rf}}$ (assuming that design CPRF settlement is equal to ultimate settlement, i.e. $s_{cprf} = s_u$ and $\delta_{cprf} = \frac{s_{cprf}}{s_u} = 1$), depending on α_{cprf}, was determined in the course of the study. The values α_{cprf} obtained according to equation (1) in the work [7] were compared to the obtained results, accepting the value $0{,}18 \le \frac{s_{pf}}{s_{rf}} = \frac{\delta_{pf}}{\delta_{rf}} \le 0{,}47$ from experience (this ratio depends on physical and mechanical characteristics of soil layers composing the base, and their strata as well). This comparison is shown in Figure 1.

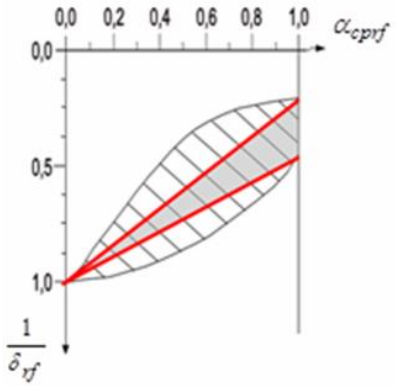

Figure 1. Dependence of the ratio of CPRF settlement to RF settlement vs. α_{cprf} calculated according to equation (1)

The results obtained according to equation (9) are presented in Figure 2.

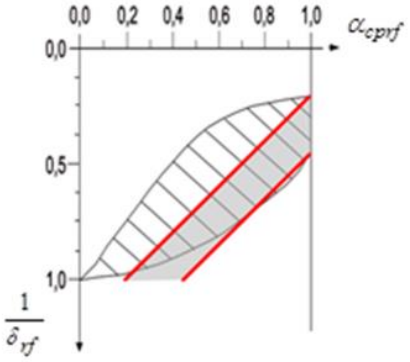

Figure 2. Dependence of the ratio of CPRF settlement to RF settlement vs. α_{cprf} calculated according to equation (9)

It can be seen in Figures 1 and 2 that two extreme hypotheses provide for distributions close to the limits of actual measurements, which is consistent with the assumption that actual foundation behavior lies between two abstract assumptions: the vertical stresses from the raft completely dissipate or do not dissipate at all between the levels of the raft bottom and pile bottom ends.

3. Results

In order to account for a certain actual value of decrease in the vertical stress at the section from the raft bottom to the pile bottom ends, it is suggested to introduce value α representing the ratio of additional vertical stress in the soil body, occurred due to pressure below the raft at the level of the pile bottom ends, to vertical stress which is directly below the raft. In other words, α is the share of undissipated pressure, the value of which is known (for example, its tabulated values are given in Table 5.8 [9], depending on the ratio of the raft sides and the pile length as the depth parameter). In this case, equation (6) takes the following form:

$$s_{cprf,1} = \frac{\alpha_{cprf} N + \alpha(1-\alpha_{cprf})N}{k_{pf}} = \\ = \alpha_{cprf} s_{pf} + \alpha(1-\alpha_{cprf}) s_{pf} \qquad (10)$$

Considering the residual stress α ($0 < \alpha < 1$), we will obtain α_{cprf} in the following form:

$$\alpha_{cprf} = \frac{\delta_{rf} + \alpha\delta_{pf} - 1}{\delta_{rf} - \delta_{pf}(1-\alpha)} \qquad (11)$$

It should be noted that under limit values $\alpha = 0$ and $\alpha = 1$, equation (11) turns into equations (1) and (9), respectively, confirming once again that the mathematical calculations are correct.

Figure 3 shows distribution at the arbitrary value $\alpha = 0.5$ (continuous line), and $\alpha = 0$ and $\alpha = 1$ represented for comparison (dashed lines). Analyzing the results obtained, we can conclude that equation (11) represents a more general expression, which coincides practically with the whole area of distribution drawn on the basis of observations and monitoring [1].

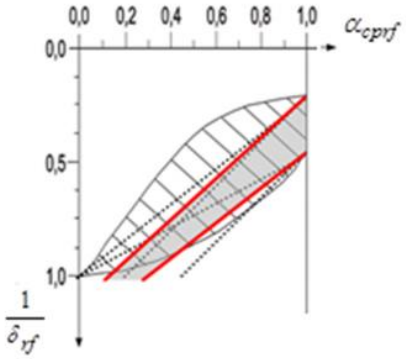

Figure 3. Dependence of the ratio of CPRF settlement to RF settlement vs. α_{cprf} calculated according to equation (11) at the arbitrary value $\alpha = 0.5$.

Figure 4 shows the area of possible location of all points if the value α constantly changes from 0 to 1, reflecting, at each α, the area of possible location of points in the graph in the same axes.

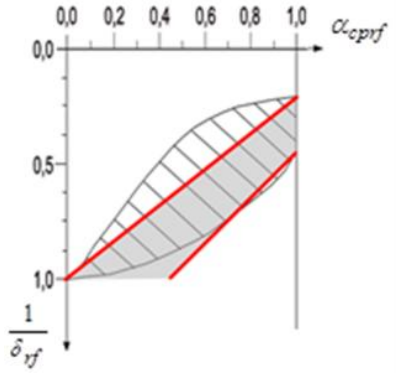

Figure 4. Dependence of the ratio of CPRF settlement to RF settlement vs. α_{cprf} calculated according to equation (11) under possible values of $0 < \alpha < 1$

It should be noted that although only purely raft and pile foundations correspond to the limit values $\alpha_{cprf} = 0$ and $\alpha_{cprf} = 1$, and there is matching of distribution zones according to the proposed method and those obtained in the result of monitoring, significant deviation in the middle zone of the experimental distribution towards smaller actual settlements of CPRF is observed. This can be explained by the fact that internal interaction in the pile–raft–soil system is the most pronounced in the middle zone ($0.3 < \alpha_{cprf} < 0.7$), and behavior of this system has been studied to a less degree than behavior of separate components (raft or piles).

This fact implies that the proposed method for determining the optimum number of piles is conservative, that is, it can only understate the

predicted settlement, which is consistent with the principles of reliability. Nevertheless, this also shows a lack of knowledge on properties of CPRF as a system with effective interaction of its elements.

4. Conclusions

The method of engineering calculation of CPRF has been proposed, which is better correlated to the experimental data.

The new equation for determination of α_{cprf} — the share of total load N on the piles — has been obtained. Proposed equation (11) is more general in comparison with equations (1) and (9), and excludes the assumption that the stresses arising under the raft completely dissipate at the level of the pile bottom ends.

The new coefficient α has been introduced, showing the residual stress, and the method for its calculation has been proposed.

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FABRICS STRUCTURE AND ITS REINFORCING EFFICIENCY IN COMPOSITE "TEXTILE-CONCRETE"

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Kew words: untwisted yarn, composites, concrete.

Abstract

In construction works, there is often need for the creation of light-weight constructions and decorative elements. Due to increasing reliability and durability, as well as because of the great weight of the concrete structures, one of the ways to reduce the increased loads is reinforcement of various construction structures with different textile materials. Producing of concrete composites reinforced with textile fabrics must ensure the guaranteed penetrability of particles into the fabric.

The possibilities of using the untwisted bonded yarn and fabrics formed from them, in the concrete matrix was studied. The bonded yarn is a new-structure material, in which the fibers are bonded by adhesive, instead of twisting. Untwisted yarn, produced from the short staple fibers, which by its structure is a composite, in which the parallel fibers are connected with point-like adhesive bonds. After finishing of crude fabric or knitted materials, the adhesive substance removed and fabric is formed, in which the parallel, twist-less yarns are kept due to weave.

Microscopic tests have shown that: the parallel arrangement of fibers in the yarn and fabric structures after finishing, ensures an increase in the porosity of product by 15%. The absence of twisting and porosity of yarn after finishing, provide a high degree of penetrability of the concrete micro-particles into the space between the threads of fabric and between the fibers of the threads, by increasing the adhesive forces on the boundary surface 'fiber-matrix".

1.Introduction

One of the ways to improve reliability and longevity, as well as to decrease significantly loads due to the own weight when using concrete designs, is the application of textiles for reinforcement of various engineering structures. Also, in construction work, it is often necessary to create and restore the lightweight structures and a number of decorative elements - borders, reliefs, etc. These concrete composites can be produced with thickness of 10 mm that might not always be achieved with the use of the standard steel fittings.

The main advantages of the textile reinforced concrete are as follows:
- lack of corrosion;
- more thin and lightweight structures;
- a possibility of creation of the irregular shapes due to the excellent draping property;
- easy handling of fabrics
- the increased longevity of a structure.

A suitable fiber for use in textile reinforcement should fulfill some basic requirements [1]: high fiber/yarn tensile strength at break (i.e. tenacity), high ultimate elongation at break, a modulus of elasticity superior to that of the cementitious matrix, as well as being resistant to an alkaline environment. Without these basic properties, the textile reinforcement cannot adequately take tensile stresses and allow for sufficient crack development in a concrete structure. Long-term properties, which are important to consider consist of, e.g. consistent bond between the reinforcement and matrix and small relaxation under permanent load. Furthermore, the initial production cost, material availability and the ease of production are critical factors that also come into consideration.

Mechanical properties of reinforced concrete depend on the properties of reinforcing textiles, such as strength, extensibility at break, stiffness to yield point, geometry (length, diameter, shape), the volume ratio of fibers, adhesion to a concrete matrix, etc. When reinforcing concrete, there can be used both natural, and high-modulus chemical fibers, various types yarns and cloths. At the same time, it is necessary to take into account adhesion of textile fittings to concrete.

For the most part, there are applied the reinforced yarns whose structures are similar to the steel fittings of a periodic profile.

For the proper realization of the properties of yarns in concrete, it is necessary to focus them in the direction of action of loading. Therefore, there are applied textile fabrics, which are produced from both natural and synthetic fibers. Both woven, and knitted products can be used. Recently the main advantage is given to knitted

fabrics. The deformability inherent in knitted structures can be perfectly used during consolidation of a concrete matrix, and when giving the necessary frame form to the product or structure, and also to receive necessary mechanical characteristics of the end product.

Manufacture of lightweight Manufacture of lightweight decorative elements of the concrete composites reinforced by textile fabrics has to guarantee high penetrating quality of particles of concrete in the fabric. Penetrability of a matrix into the fabric, especially between the filaments, depends on the nature of bonds in the fabric and the resultant tightening effect, structure of fabric, quantity of filament in the yarn, and technological process.

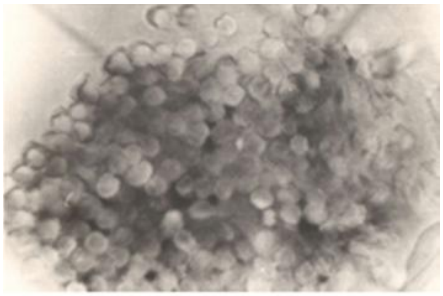
The use of textiles with a new structure will broaden the scope of application of textile-reinforced concrete, and will increase the effectiveness of the implementation technological process.

2. Methodology

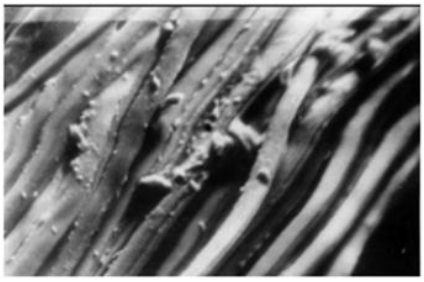
The paper offers the use of fabrics produced from untwisted yarn for reinforcing concrete. Untwisted yarn is a textile material in which the bundles of fibers are located in parallel and are connected to each other by an adhesive. After finishing of crude fabric or knitted materials the adhesive is washed out and the bundles of parallel fibers remain in the fabric's structure, which are retained by interweaving of fabric [2].

There were studied the structure of yarn with the line density of 25th tex. produced from the high-modulus viscose fibers with a staple length of 40 mm. As an adhesive, there was used a water-soluble polyvinyl alcohol in the amount of 8-10%.

Microphotographs (Fig.1) represent the arrangement of fibers in the transverse and longitudinal sections of untwisted yarn. As can be seen the binder not completely fills the space between the fibers, but forms the point-like adhesive couplings between them. Unlike the yarn of traditional spinning the cross-section is not round, but closer to the ellipse, but the longitudinal arrangement points to the porosity of yarn.



a.

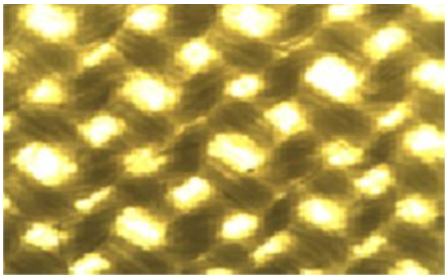


b.

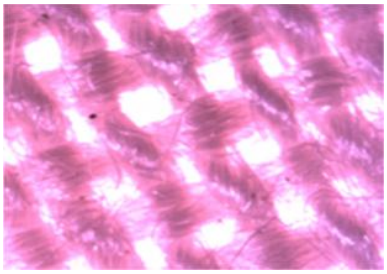
Fig.1 The arrangement of fibers in the transverse (a) and longitudinal sections (b) of untwisted yarn

There has been studied the structure of the tested fabric of plain structure. After finishing of crude fabric the adhesive is washed out and only the yarn remains in the structure - fibers located in parallel.

Figure 2 illustrates microphotographs of fabric before and after finishing,



a.



b.

Fig.2. Microphotographs of fabric
a- crude fabric ; b-fabric after finishing,

3. Results

Ideally, the aim is to activate the entire yarn/roving to achieve a constant stress distribution across the yarn cross-section.

Unlike the polymeric matrix, the mechanism for reinforcing fibers and yarns in the concrete matrix is different. This is conditioned by splitting property of concrete, that is because of low extensibility of the concrete matrix, which significantly lower than extensibility of fiber. The matrix of polymeric composites is more plastic than the matrix of fibers . But destruction of the destruction of the matrix in the concrete composites occurs before the fiber strength is completely determined. So, the effect of reinforcing of fibers is determined after the crack initiation in the matrix, that is the fibers are capable of load carrying after the matrix destruction.

The reinforcement yarns are composed of a large number of filaments or fibers, which are densely packed. The penetration of fine-grained concretes matrix into the fibers is possible if no additional barriers are on the surface of yarns, which prevent the intrusion of matrix particles. The small diameters of fibers (about 10-20 μm) and the distances between them lead to filtration effects. It seems that due to its relatively high viscosity, the cement particles intrude only into an outer layer of yarns.

A shape and geometrical parameters of yarn influence on penetrability of matrix. There are several models of yarns. Yarn has two major bond zones. The outer ring zone of yarn, is called fill-in zone and the central part of yarn is named the core zone. In the fill-in zone, the matrix has strong adhesive contact with fibers. The fill-in zone is the depth at which adhesive load transfer can take place between the filaments and the matrix. The inner zone, i.e. the core zone, is defined as the filaments having less contact with the matrix but assuming that frictional load transfer between the filaments remains possible [3].

In fill-in zone the matrix has continuous contact with threads, there exists a relatively strong adhesive bond between a matrix and threads. At will, the coatings are the connecting intermediate links between the matrix and filament. Adhesion is caused mainly by chemical bonds between the cement paste and filaments, but also it can be resulted from mechanical reciprocal interlocking. The matrix barely penetrates into the yarn core. This leads to primary transfer of frictional loading between threads, once the initial resistance of weak transverse bonds is exceeded. However, if you apply additional coatings, which penetrate into the core of threads, the bond can be assessed as significantly stronger because of the larger number of adhesive transverse bonds and the adapted properties of bond between the coating, thread and a matrix.

Transfer of frictional load depends on surface roughness and transverse pressure acting in a contact zone. In the mechanisms of transfer of frictional load. The transferred forces in the mechanisms of transfer of frictional load, are significantly lower in comparison with the case of the adhesion in a filling zone. Unlike the steel reinforcing cores with ribs, no shearing bond exist in the direction of fibers , because of mechanical interlocking between the fittings and concrete. However, in textile products, transversal threads, which are connected to longitudinal threads canal so cause shearing bond.

The size and ratio between filling and the core zone depend on various properties:

1.The form of yarn or ratio between the circumference and cross-sectional area. For example, circular-shaped yarn with a small circumference has a small number of filaments in the fill-in zone and a large number of filaments in the core zone. In contrast, very flat yarn, for instance with an elliptical shape, might have almost all filaments in the fill-in zone and might have essentially no core zone. As is seen, in our case, the cross –section of untwisted yarn is closer to the ellipse (Fig.1)

2. The packing density of threads, which, in turn, depends on the production technology of yarn and interweaving technique in the fabric. While the low packing density enables the best penetrability of threads, the high packing density can raise frictional loading of transfer due to the increased side pressure. As is seen, in our case, after the removal of the adhesive, a free space in the fabric increases that allows the micro-particles of concrete for penetrating into yarn fibers. (Fig.3)

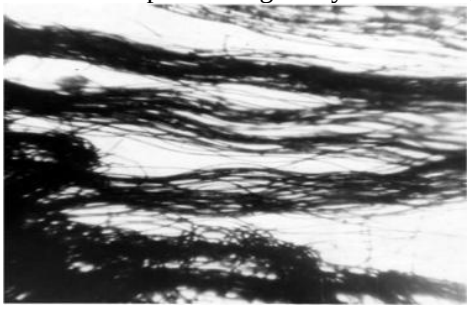


Fig.3 Microphotographs of yarn after finishing

To some extent, penetration of micro-particles of concrete into yarn is influenced also by viscosity of a fresh matrix, since putting in of fittings is carried out before concrete hardens.

4. Conclusion

Studying the structure of untwisted yarn and fabrics, and comparing them with the existing models, and also based on the descriptions of the mechanisms of penetration of concrete micro-particles into the structure of reinforcing textile materials, it is possible to make the following conclusions:

- the cross section of untwisted yarn it is close to an ellipse;
- in the longitudinal direction, fiber are located in parallel to each other, the space between them short is not completely filled with binding material;
- after finishing of crude fabrics, the adhesive is washed out and a free space remains between fibers.

The structure of such reinforcing textile ensures the maximum penetration of particles of concrete into the space between threads and the space between fibers in yarn.

The use of fabrics from untwisted yarn allows for forming the thin concrete composites for restoration works.

5. Acknowledgments

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INTERACTIVE ACCULTURATION AND POST-COLONIAL CONSTRUCTION OF URBAN IMMIGRANTS' CULTURAL IDENTITY: VANUATU AS A CASE STUDY

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Key words: Interactive acculturation; post-colonial; urban; cultural identity; Vanuatu

Abstract

A continuum of ideological premises is proposed that seeks to account for the broad range of Vanuatu's immigrant integration policies adopted by the immigrants from European and Asian countries. A review of Urban-Social Psychological models of immigrant acculturation strategies demonstrates the necessity to explain the interactive nature of urban immigrant-Vanuatu (as a host community) relations. The Interactive Acculturation Model (IAM) presented proposes that urban immigrant-Vanuatu (UIV) relational outcomes are the product of the acculturation orientations of both Vanuatu (host) majority and immigrant groups as influenced by integration policies. IAM makes predictions regarding the acculturation combinations most likely to produce consensual, problematic, and conflictual relational outcomes between urban immigrants and members of Vanuatu (host) community.

1 Introduction

Against the backdrop of Vanuatu Multiculturalism or Culturalism Pluralism, the Interactive Acculturation Model (IAM) in Vanuatu is always seeking to combine the components of immigrants and Vanuatu as a host community relations within a common theoretical framework, that is, acculturation orientations adopted by immigrant groups in Vanuatu as the host community and acculturation orientations adopted by Vanuatu towards special groups of immigrants. And the goal of IAM is to present a more dynamic account of immigrant and Vanuatu as a host community acculturation in multicultural Vanuatu social settings. The highlight that is running through the whole discussion turns out to be: is it acceptable that immigrants maintain their cultural heritage or that immigrants adopt the culture of the host community? IAM essentially categorizes Vanuatu population in favor of cultural integration, assimilation or separation. In terms of the accommodation level of Vanuatu as the host society, IAM predicts whether the immigrant population will become fully assimilated, marginalized or even isolated from Vanuatu as the host community.

Post-colonial ethnic groups in Vanuatu turn out to be a combination of otherness and contingency and it is the two factors that arouse reconsideration as to wholesome urban planning in Vanuatu. From the perspective of Post-colonialist, to illustrate, Edward Said, Post-colonial ethnic groups dwelling in a confined zone or district must be of the same attribute, and this perspective results in such a traditional conviction that traditional planning mode or Science of architecture takes urban interior space as a homogeneous, static and balanced space. Nonetheless this conviction loses sight of different urban ethnic groups in Vanuatu: they enjoy their own cultural identity, they cherish the same historical background and cultural type.

In ***The Location of Culture*** (Homi Bhabha, 1994), the liminality of hybridity is of colonial anxiety. The hybridity of post-colonial Vanuatu cultural identity, to be specific, urban community group cultural identity as a cultural form, altered the authority of power. Via the theory of Hybridity it is made clear how urban community group cultures in Vanuatu come to be

represented by processes of iteration and translation through which their meanings are vicariously addressed to or through an "Other". The colonial subject takes place, its subaltern position inscribed in that space of iteration. The colonial subject is located in a place of hybridity, its identity formed in a space of iteration and translation by the colonizer. "The discriminatory effects of the discourse of cultural colonialism, for instance, do not simply or singly refer to a 'person'… or to a discrimination between mother culture and alien culture … the reference of discrimination is always to a process of splitting as the condition of subjection: a discrimination between the mother and its bastards, the self and its doubles, where the trace of what is disavowed is not repressed but repeated as something different---a mutation" (Homi Bhabha, 1994). Hybridity in urban community groups opens up a space, figuratively speaking, where the construction of a political object that is new, neither the colonizer nor the "Other", properly defies our political expectations. And for another, hybridity is a doubling, dissembling image of being as well in at least two places at once. This turn, beyond any doubt, makes the presence of colonist authority no longer immediately visible.

2 Interactive Acculturation and Urban Cultural Identity in Port Vila

Since Greco-Roman Culture took shape, city or polis has been rated as the core of human civilization. City is "the material token of human civilization" (From Gorgias, Plato, 1986). European urbanization in the Middle Ages eulogizes that the urban cultural token derives from the urban planning of urban basic facilities and recreational places. Urban multi-community-group culture also embraces regulation of urban space, rational cultivation of land and cultural policy to distribute urban living, the outcome of urbanization. With regard to urban planning of Port Vila, the fundamental factor which really counts proves to be its urban culture. Urban multi-community-group culture is the decisive factor which lays a solid foundation for the urban image of Port Vila embodied by its substantial tokens. Urban planning turns out to be the spatial layout of urban substantial tokens. Accordingly, port Vila's urban culture determines its wholesome layout and distribution of its urban facilities.

With Vanuatu's independence of the British and French colonization in 1981 as well as the European – Bislama cultural diversity in Vanuatu existing for 106 years, Port Vila commenced to enjoy four great cultural divisions:

Northern Cultural Zone:

"Pedro-Fernandez-de-Queiros". Urban ethnic group is a combination of British-Anglo urban culture and Bislama Aboriginal Culture.

The Middle Cultural Zone: Polynesian Culture and Anglo- British urban culture shape the cultural type of the Middle Cultural Zone. Local chiefs and tribal people co-exist with the urban British Anglos in the urban area.

Southern Cultural Zone: "Island City" is a striking characteristic to urban ethnic groups in the Zone. Tana is such island city: Tana people accepted Anglo-British Culture. In their alleged "group culture", they, though maintaining their own ethnic culture, take English as their native language, receive British education and mingle British Culture into their own. Hence their culture is a typical example for Homi Bhabha's "hybridity".

Ambea and Shepherd Cultural Zone:

"hybridity" and "liminality" are more striking and of great sincerity, compared with the other cultural zones. Urban ethnic groups in Ambea and Shepherd are indigenous Bislama, speaking different local tribal tongues, as an illustration, Aronga and Araki. Their own national cultural is well-maintained and in the meantime Anglo-British Culture silently, even without knowing, merges into their daily life.

2.1 Intra-generational Equity and colocation as the core

Port Vila, the capital and largest city of Vanuatu, is located on the island of Efate. Its population in the last census (2009) was 44,040, an increase of 50% on the previous census result (29,356 in 1999). In 2009, the population of Port Vila formed 18.8% of the country's population, and 66.9% of the population of Efate with various community groups co-existing.

The multi-community-group harmony and coexistence go between "modernity and nature" in Port Vila and demonstrate the philosophical thought of

"Environmental Intergenerational Equity" (Page, 1986). "Environmental intergenerational equity", also known as "environmental equity of inter-generations" in philosophy, refers to the fact that the development of human society shall follow through on a sustainable mode and current and future members of the society shall form an entirety of virtuous cycle to share natural and cultural resources of the planet and enjoy the livable ecological environment. People now living on the planet are not just administrators of future environment, but also the inheritors and beneficiaries of resources and fruits of the society left by the former generation. As such, they are obliged to protect the existing environment and resources and also entitled to enjoy earth resources and environment. "Environmental intergenerational equity" principle provides insights into the issue of how to rationally and justly allocate resources and environment between current and future people who enjoy equal rights in three aspects, namely equitable use of natural resources, resources meeting personal demands, and seeking personal survival and development. People living at the moment shall leave necessary environmental and natural resources to future generations.

Collocation as the core

Page put forward the concept of "Environmental Intra-generational Equity" in 1988 at the earliest time. The concept underscores "collocation", which means that every generation is the trustee of future generation and under the entrustment of future generation, this generation is responsible for protecting the environment and handing it over to the future generation in good condition. Page's principle has been applied in Port Vila, which focuses on three significant rules. The first one is the principle of protecting selection. Current people shall preserve diverse natural and cultural resources for the future generation to avoid their limited rights and ensure they could enjoy diverse resources which are similar and made available to today's people. The second one is the principle of protecting quality. Every generation is obligated to protect the "quality" of the earth and when handing it over to the later generation, the earth isn't damaged or earth "quality" isn't exhausted. The third one is the principle of protecting contact and usage. Intergenerational members have the right to contact and use the heritage left by the former generation on an equal footing and preserve the right to contact and use for the future generation. In other words, current people are entitled to benefit from the heritage left by the former generation and future generation could also benefit from various heritages left by their former generation.

What is the best way?

Urban environmental form embraces the senses in both time and space, connoting the strategies or solutions to taking advantage of space in "the best way". "The best way" is applied to denote the value system and judgment cherished by various urban community groups towards life quality form both personal sphere and collective sphere. And meanwhile the urban dwellers make their judgments and evaluations towards the forthcoming environment policies to urban life. In the 19th-century Port Vila, its urban transformations made its new urban culture: to begin with, Port Voila challenged the traditional villages and towns of South Pacific Area, and ultimately new urban multi-community-group culture took the place of the original urban cultural mode. The perception to new urban culture derived from a new attitude towards the relation between urban ecological form and the physical conditions, spirit and virtue of urban dwellers. Accordingly, between the 1960's and the 1980's, which corresponded with the starting period and accelerating period of modern urban planning, the advancement of urban culture was confronted with quantitative problems of urban environmental form, and there should be coming up with new techniques to wipe out these problems. It was during the two periods that great urban transformations took place in Port Vila, to illustrate, the planning and construction of new transportation facilities, the new layout of residential district, public space, health-care organizations, partition of land use and recreational facilities. These transformations mark the increasing living conditions resulting from the material and virtue carrier, i.e. city.

Beyond any doubt, urban environmental form proves to be the fundamental token of urban culture. Port Vila, as the carrier of the life and work of its urban

dwellers, bears the citizen identity and cultural identity of its urban dwellers, which comprises the urban culture of Port Vila. From Port Vila as a case study, urban environmental form depends on the planning and construction of urban material space and substantial space. And urban dwellers' awareness is expected to go to the planning of humanistic idea of urban dwelling. The goal to plan urban environmental form is to provide urban dwellers with positive ecological circulation and intergeneration-equity urban environment. Urban environmental form consists of urban substantial environment, urban spiritual environment and urban policy-making environment, which are complementing and enrich one another. How to make urban environmental form substantial attributes to "the best way" and the token of urban culture.

2.2 Cultural Identity and Urban Public Space

Urban public space is all-round open to all urban community groups and rated as the core of urban environmental form in which all the cultural activities are going on. Diachronically and synchronically, in Port Vila, the formation of its urban public space is, first and foremost, motivated by its urban economic growth and cultural mode. Urban public space, as the substantial environment to host life, work, communication and recreation for urban dwellers, represents urban historical legacy and landscape feature.

In Port Vila, urban public space is compared to "urban multi-community-group sitting room". "Urban public space with striking characteristics always arouses evocative resonances of its urban dwellers. When approaching the city, you will soon develop your senses of belonging and identity." (Susan S. Fainstein, 2011)

3 Intercultural Acculturation and Urban Spatial Justice

3.1 Urban Space

Urban space is, to be specific, composed of physical, institutional and cultural dimensions and subdivided into physical sphere, social sphere and spiritual sphere. Perth is divided based on these three dimensions. "Contemporary urban space is the outcome of capital production and spatial logic is subject to capital logic".(Henry Lefebvre, 1996) City turns out to be the result of spatial production and goes against the backdrop of modern urban development, and urbanization advancement is defined by the characteristic of capital property. As the combination carrier of both modern civilization and global capitalist system, Vanuatu has evolved into a spatial focus point of capital concentration and circulation.

3.2 Void of Urban Spatial Injustice

Space of urban community groups, as a matter of physical existence, objectively proves to be independent of human consciousness. As an object of science, space of urban community groups is labeled with "neutral", "innocent" and "non-political"; when it comes to its nature or "intuition", urban space is, no wonder, a political space, and its political and economic traces left by urban space in particular. Spatial possession and division is referred to as the embodiment of political and economic position, and in succession the cultural identity and social formation unique to urban space take shape. Space, in essence, is by no means an object of science which keeps a remote distance between ideology and politics. On the contrary, space is permanently stamped with political identity and strategic identity. Being "neutral and non-profitable", what's more, space of urban community groups functions as a microcosm of pure latitudinarian and rational abstractness. The process for space of urban community groups to develop turns out to be political and accordingly, the space in discuss is dependent of politics and ideology. Space of urban community groups is treated as the relation between work force and means of production as well as the social relation of production and part of reproduction.

3.2.1 Void of the crisis of urban sustainable development

Vanuatu Government has started its low-carbon urban development since the 1960's and adopted the urban development strategy of "controlling urban development size, rationally developing living quarters in city outskirts and actively developing ecological urbanization" in the 1990s. Since the early 21st century, Port Vila, Capital of Vanuatu, has witnessed the rapid entrance of low-carbon track in terms of urban spatial development, green coverage, energy consumption reduction and low-carbon work. By the early 2015, its green urbanization rate has been increased by 62%. Thanks to the dual role of urban expansion and internal spatial reorganization, suburbanization of Port Vila has become ever evident. Rational urban management and "apple-pie ordered" urban spatial organization guarantee the rational extension of Port Vila, prevents Port Vila

from loss of arable land, and narrowed the internal and suburban difference of urban space.

Port Vila and other South Pacific Island cities, though defined by different humanity history, geology and environment, urban scale and function, under the joint dynamic mechanism of low-carbon city development, demonstrate similarities in urban space: ①stable urban form, ②balanced spatial aggregation, ③coexistence of horizontal and vertical urban expansion, ④positive ecological circulation, and ⑤harmony among history, convention and urbanization. While ensuring environmental quality and construction well-underway in the city, Port Vila retains historical style and traditional features, strong urban recognition of urban dwellers and diversification of city identity. As such, Port Vila has achieved sustainable development and built a low-carbon urban form amid rapid urbanization process.

3.2.2 Void of Spatial Mismatch

Professor Kain at Harvard University put forward the complete "Spatial Mismatch Hypothesis" or "Mismatching Space" for the first time in 1964. "Spatial Mismatch" originally referred to the fact that Ghetto had fewer job opportunities than white society and Black people were faced with greater difficulties in employment and gained lower salary, or they had to spend more time on commuting than white people, making the unemployment rate higher. Space of urban community groups in Port Vila: ①reduces the investment scale of infrastructure construction, ②cuts back on the rapidly inflating urban space, ③puts a lid on floating population, ④protects arable land resources and reduces the consumption of coal, electricity and oil as transportation fuel, advocating “Pedestrian Movement” and ⑤reduces the expansion of industrial zone. Supposed that it evolves into the deprivation of spatial interest by some, urbanization will surely lead to intensified contradiction and social instability. Urbanization as such is unsustainable and the urbanized development of Port Vila proves to be void of the occurrence of such scenario.

3.2.3 Void of “city feature crisis”

Alongside with urbanization, there springs up modern buildings as an intruder towards traditional national buildings. It is beyond any doubt that urbanization will definitely contribute to the emergence of "urban forecast" composed of rebar and cement. Despite of the birth of modern buildings in space of urban community groups, Port Vila has perfectly preserved its historical icons coexisting with the modern buildings, forming integrity between the tradition and the modern. Port Vila has been void of the “city feature crisis” resulting from blind urban planning manifested by ①unrealistic "destructive enthusiasm", that is, unrealistically elevating urban orientation and blindly building international metropolis or cosmopolitan city; ②unrealistic urban renewal and vanity project; and ③particularly irrational planning of transportation system, blindly chopping trees of sidewalks and broadening carriageways.

4 Conclusion

Interactive Acculturation and cultural identity prove to be integrity of a cultural system with the combination of the substantial, spiritual and ideological factors of urban environmental form. The harmonious development is a striking focus between cultural identity and urban justice, Intercultural Acculturation and immigrant identity. The cultural mode of urban community groups in Vanuatu--- harmony between the traditional and the modern is still acting as a microcosm of urban spatial justice in Vanuatu.

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ON CONTEMPORARY CONNOTATION OF BEIJING TRADITIONAL ARCHITECTURAL CULTURE

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Key words: Beijing; traditional architecture; architecture culture; connotationAbstract

Being the concrete manifestation of culture, architecture not only reflects the level of productivity and technology of an era, but also suggests people's spiritual pursuit and values, and records people's production mode and lifestyle at that time. Chinese traditional architecture is an important part of Chinese traditional culture as well as the fruit of wisdom of the Chinese nation over the five thousand years. We could get to know the lifestyle and thinking pattern of ancient people in China, understand the spiritual life and cultural essence of the Chinese nation and establish the Chinese culture with time and ethnic characteristics, by studying on Chinese traditional architecture. As the capital of China, Beijing is the national political, economic and cultural center and the ancient capital in Yuan, Ming and Qing Dynasty, and therefore embraces rich and brilliant Chinese traditional culture. By researching on traditional architecture in Beijing, we can acquire a full understanding of the history and culture of Beijing and the rich connotation of the Chinese culture. The study on Beijing traditional architecture culture is to study on the history and culture of the thousand-year-old city, Beijing, through the research on traditional architecture in Beijing. It mainly consists of such aspects as study on palace architecture culture, study on religious architecture culture, study on landscape architecture culture, study on names of streets and lanes and Hutong culture, study on folk houses and folk culture, study on mausoleum architecture and *Fengshui* culture, study on origin and changes of Beijing city, study on old town and historical sites of Beijing and study on local chronicles of the city, counties and districts of Beijing.

1 Introduction

Being the concrete manifestation of culture, architecture not only reflects the level of productivity and technology of an era, but also suggests people's spiritual pursuit and values and records people's production mode and lifestyle at that time. Therefore, architecture is the integration of technology and art and the reflection of material civilization and spiritual civilization of human. The best way to understand the culture of a state is to understand the architecture of the state, meanwhile the best way to get to know the cultural characteristics of an ethnic group is to understand its architecture. Architecture, the special cultural manifestation, enables us to get a full understanding about the cultural features and spiritual characteristics of a state or an ethnic group. Ancient Chinese architecture is an important part of Chinese traditional culture as well as the fruit of wisdom of the Chinese nation over the five thousand years. We could get to know the lifestyle and thinking pattern of ancient people in China, understand the spiritual life and cultural essence of the Chinese nation and establish the socialist new culture with national characteristics, by studying on ancient Chinese architecture.

As the capital of China, Beijing is the national political, economic and cultural center and the ancient capital in Yuan, Ming and Qing Dynasty, and therefore embraces rich and brilliant Chinese traditional culture. By researching on traditional architecture in Beijing, we can acquire a full understanding of the history and

culture of Beijing and the rich connotation of the Chinese culture. The study on Beijing traditional architecture culture is to study on the history and culture of the thousand-year-old city, Beijing, through the research on traditional architecture in Beijing. It mainly consists of such aspects as study on palace architecture culture, study on religious architecture culture, study on landscape architecture culture, study on names of streets and lanes and Hutong culture, study on folk houses and folk culture, study on mausoleum architecture and *Fengshui* culture, study on origin and changes of Beijing city, study on old town and historical sites of Beijing and study on local chronicles of the city, counties and districts of Beijing etc.

1 Architectural Culture of Royal Palace

As Beijing is the ancient capital of Yuan, Ming and Qing Dynasty, the palace architecture constitutes the most important and distinctive traditional architecture of Beijing. The palace architecture and the palace culture reflected by the architecture make one major characteristic of Beijing traditional architecture culture.

The study on palace architecture culture mainly focuses the characteristics of royal palace architecture (mainly of the Imperial Palace), and the cultural connotation. The whole architectural art of the Imperial Palace architectural complex reflects the special style and outstanding achievements of Chinese traditional architectural art, which is considered one of the most excellent architectural complex worldwide. Since the construction in Yongle years of Ming Dynasty, this magnificent building has been exposed to constant reconstruction and transformation in over five hundred years, representing the collective wisdom of the laboring people in ancient China. The Imperial Palace, also known as the Forbidden City, is the imperial palace in Ming and Qing Dynasty. Its construction was initiated in the year of 1406 and basically completed in 1420. At present, it has approximately 9,000 rooms of different types, which apply the wooden structure and adopt the yellow glazed tiles as the ceiling, greenish white marble as the base and color paintings as decorations. The Imperial Palace is divided into the Outer Palace and the Inner Court by the Gate of Heavenly Purity. The Outer Palace centers on three main halls, i.e. Hall of Supreme Harmony, Hall of Central Harmony and Hall of Preserving Harmony, and has the Hall of Literary Glory and Hall of Martial Valor respectively located at the east and west, where the emperor met officials, dealt with national affairs and held ceremonies. The three major halls, exactly situated on the Central Axis of Beijing, are the tallest buildings in the Forbidden City.

The study on architecture culture of prince's mansion plays a supplementary part in the palace architecture culture. Mansions of princes and princesses in Beijing refer to the dwelling places of princes, officials, princesses and emperors' sons-in-law in the past dynasties, which are the architecture form one level lower than the palace architecture. By investigating on the architecture style and features of mansions of princes and princesses in Beijing, it is possible to understand the lifestyle of the nobility and discover the historical and cultural connotation. Numerous mansions of princes and princesses are distributed in Beijing, mainly including Prince Gong's Mansion, Prince Yan's Mansion, Prince Fu's Mansion, Prince Na's Mansion, Prince Chun Mansion, Prince Yu's Mansion, Prince An's Mansion, Prince Zheng's Mansion, Prince Li's Mansion, Prince Qing's Mansion, Prince Sengzhong's Mansion, Prince Jingjin's Mansion, Prince Xunjun's Mansion, Princess Shou-en's Mansion and Princess Hejing's Mansion, etc.

3 Religious Architectural Culture

Being the embodiment of religious consciousness, aesthetic sense, traditional customs and architecture technology of human, the religious architecture plays an important role in the architecture history and cultural history of ancient China. The religious architecture in ancient China reflects the brilliant ancient civilization over the five thousand years of the Chinese nation and reveals the outstanding wisdom and excellent technology of laboring people and architecture craftsmen in ancient China. These architecture works witness the blend between Chinese traditional culture and foreign culture and their development and thus feature extremely high scientific and artistic value as well as historical and cultural value. The study on the traditional religious architecture in Beijing helps us grasp a deep understanding of religious and philosophic ideas in ancient China and analyze the profound connotation of the traditional culture of Chinese nation.

Many religious architecture works are distributed in Beijing, which mainly consist of the following five types of works:

3.1 Buddhist architecture.

Temples, pagodas and grottoes mainly represent the Chinese Buddhist architecture. The Buddhist architecture works in Beijing and the suburb mainly include the Lama Temple of Peace and Harmony, Tanzhe Temple, Jietai Temple, Biyun Temple, Fayuan Temple, Yunju Temple, Tianning Temple, Huguo Temple and Baita Temple etc.

3.2 Taoist architecture.

The major representative of Chinese Taoist architecture is the Taoist temple, where the Taoists cultivate themselves, offer sacrifices and pray. Main Taoist architecture works in Beijing include the Imperial Ancestral Temple, Altar of Land and Grain, Temple of Heaven, Temple of Earth, Altar of the Sun, Altar of the Moon, Baiyun Taoist Temple, Dongyue Temple and Zaojun Temple etc.

3.3 Confucius architecture.

The Confucius Temple and the Imperial College are the masterpieces of traditional architecture that reflect Confucius thoughts in Beijing. Beijing Confucius Temple is a place to worship Confucius, the famous thinker, philosopher and educator of China. Beijing Imperial College is the top institution of learning established by the imperial court in Yuan, Ming and Qing Dynasty and the only government-run institute of higher learning in ancient China, aiming to cultivate officials for feudal governments.

3.4 Christian and Catholic architecture.

Generally, major representatives of Catholic architecture should be churches. There are four famous Catholic churches in Beijing, respectively including the southern Xuanwumen Church, the northern Xishiku Church, the eastern Wangfujing Church and the western Xizhimen Church. Famous Christian churches in Beijing include the Chongwenmen Church, Kuanjie Church, Zhushikou Church and Gangwashi Church etc.

3.5 Muslim architecture.

Mosques (Qing Zhen Temple literally in Chinese) are the major representatives of Muslim architecture. “Qing” indicates the pureness of Allah; “Zhen” suggests the uniqueness and eternity of Allah; and Temple follows the appellation of Buddhism and Taoism. Niujie Mosque is considered the masterpiece of Muslim architecture in Beijing.

4.Architectural Culture of Landscape

The classical gardens of China have been recognized as a pearl in heritages of Chinese architecture culture due to unique artistic style. Though classical gardens of China are mainly distributed south to the Yangtze River, including the most well-known Suzhou Gardens, some famous classical gardens are also preserved in northern area of China by now, such as the Summer Palace, the Old Summer Palace and Beihai Park in Beijing.

Distinguished from private gardens in the area south to the Yangtze River, the classical gardens in Beijing are mostly royal gardens. In Ming and Qing Dynasty, famous landscape architecture works in Beijing mainly consist of “Three Hills and Five Gardens”. As the collective term for the imperials palace and gardens at the western suburb of Beijing, the “Three Hills and Five Gardens” were firstly constructed under the reign of Emperor Kangxi in Qing Dynasty and flourished under the reign of Qianlong, which were mostly burnt and destroyed in the Second Opium War in 1860. The “Three Hills” refer to the Longevity Hill, Fragrance Hill and Jade Spring Hill; and the Garden of Tranquility and Pleasure, Garden of Tranquility and Brightness and Garden of Clear Ripples respectively built on the three hills and the neighboring Garden of Everlasting Spring and the Old Summer Palace are collectively known as the “Five Gardens”. Only the Garden of Clear Ripples, namely the Summer Palace, has been preserved by now out of the five gardens. The Old Summer Palace only has ruins left, while the Garden of Everlasting Spring outside the western gate of Beijing University has been transformed to be restaurant and dormitory.

The emphasis of the study on classical gardens in Beijing should be placed on study on their style and features, exploration of their historical and cultural connotation and protection of current relics parks.

5. Names of Street, Lane and Hutong Culture

Beijing is currently positioned as the national capital, international metropolis, famous cultural city and livable city. It is an important part in protection of the famous historic and cultural city of Beijing to protect the place names within the old town of Beijing. When studying on names of streets and lanes in Beijing, the focus should be placed on exploring the historical evolution and cultural connotation of names of these streets and lanes, preparing the list of classified protections of place names and protecting the cultural heritages of place names. To be specific, it requires giving priority to use of historic place names, reserving the name of Hutong, respecting the habits of the public, restoring the cultural heritage and revealing the historic landscape.

Names of streets and lanes in Beijing are mainly divided into two types. One was assigned by the royal, such as Chang’an Street, Dongdan and Xidan etc.; and the other was gradually formed in people’s daily life, representing the mode of production and living of citizens in a certain period and containing profound historical culture and folk traditions, which are mostly named as XX Hutong. Hutong constitutes the feature of names of traditional streets and lanes in Beijing. The word “Hutong” firstly emerged in Yuan-Dynasty zaju plays. For example, Guan Hanqing wrote “fighting a way in the bloody Hutong” in *Single-knife Meeting*. Xiong Mengxiang of Yuan Dynasty once mentioned in *Xijinzhi* that “Hutong was originally a word from the dialect”. In Ming Dynasty, Shen Bang further explained in *Wanshuzaji* that “The word Hutong was derived from Mongolian, which originally referred to wells and later

means small streets and lanes". Hutongs in Beijing were formed in Yuan Dynasty and constantly developed in Ming and Qing Dynasty. The 384 lanes and 29 Hutongs of Yuan Dynasty were expanded to approximately 1,170 streets and Hutongs in Ming Dynasty, including about 459 directly named as Hutong. By Qing Dynasty, the number of streets and Hutongs had attained 2,077, including 978 directly named as Hutong. After liberation in 1949, the number of Hutongs in Beijing has been growing along with increase of the population. According to *Zaishuohutong* (On Hutong) by Zhang Qingchang in 1990, the total number of streets and Hutongs in Beijing had reached 6,029 by 1980.

The names of Hutongs in Beijing feature profound cultural connotation. Hutongs are generally named in two different ways. The first is the industry of the Hutong, such as Yangrou (Mutton) Hutong, Pichai (Firewood) Hutong and Biandan (Shoulder Pole) Hutong etc.; while the second is the residents of the Hutong, such as Jiangjia (Jiang's) Hutong and Wuding (roof) Hutong etc.

In addition to names of Hutongs, Beijing also has many other place names embracing a long history and rich connotation. Along with demolition in many places, original buildings and streets in Beijing have been destroyed and replaced by new buildings. As the transformation of the old town and urban development process are facilitated, many traditional place names in Beijing are facing the risk of elimination, which would lead to disappearance of many cultural and historic relics, names of streets and lanes and the relevant historical and cultural connotation. Therefore, it is of great significance to investigate on, collect and study on names of traditional streets, lanes and Hutongs in Beijing.

6. Folk Houses and Folk Culture

Folk houses in Beijing mainly indicate the Si He Yuan (Courtyard House), a courtyard house enclosed by houses respectively at the east, south, west and north. It is the ancient civil architecture with the most distinctive Chinese characteristics, with Beijing Si He Yuan as the most famous representative. Si He Yuan reflects the specific geological and cultural environment and folk culture of Beijing. Beijing Si He Yuan could be traced back to a long time ago, as such kind of house had been very popular in Beijing since Ming and Qing Dynasty. Si He Yuan is characterized by the arrangement of houses and the courtyard in a symmetrical manner along the north-south central axis. Generally, the room facing the south is called the "principal room" or "main room" where the elders reside; the room facing the north is called the "reversely-set house" which is provided to guests or servants; and the rooms at the east and west are known as "wing rooms" where the juniors reside. The courtyard is enclosed by walls all around and all doors and windows are opened to the courtyard for safety and quietness. In most cases, the gate is opened at the eastern part of the southern wall, with a screen wall set up inside the gate to keep strangers from seeing the layout of the courtyard. The space between the first gate and the second gate is generally called the "foreyard" or "outer court", and a row of "reversely-set house" is placed at the south of the foreyard. The second gate, also known as "festooned door", usually features luxurious decorations. Inside the second gate lays the inner court or principal court, which makes the central part of Si He Yuan. The principal rooms and wing rooms are often connected with corridors. Flowers and trees are planted in most courts. Some houses of large size consist of several Si He Yuan, where the garden is built beside or behind the Si He Yuan.

Beijing has been the political, economic and cultural center of China since Yuan, Ming and Qing Dynasty, and thus has accommodated many high officials, noble lords, men of letters and famous names. For this reason, many notable former residences are preserved in Beijing. The study on folk houses in Beijing also incorporates studying on notable former residences and exploring their historical and cultural connotation. Famous notable former residences in Beijing include Former Residences of Cao Xueqin, Pu Yi, Kang Youwei, Liang Qichao, Tan Sitong, Ji Xiaolan, Lin Zexu, Li Dazhao, Guo Moruo, Lu Xun, Mao Dun, Lao She, Ouyang Yuqian, Ye Shengtao, Qi Baishi and Song Qingling, Memorial Halls of Mei Lanfang and Xu Beihong, and Sun Yat-sen's Temporary Dwelling etc.

7. Mausoleum Architecture and Fengshui Culture

Confucius, the famous thinker in China, has a famous saying, "How can you know what death is before knowing what life is". It fully reflects Chinese people's spiritual pursuit of holding positive attitude to life and treasuring current life. However, grand funerals had always been popular in ancient China due to restrictions of the ancient patriarchal system and funeral customs. It can be seen from the importance attached to the mausoleum by Chinese people.

Feudal emperors in Chinese history all thought highly of the address of mausoleum and the funeral after death. Therefore, the famous Ming Tombs and the Eastern and Western Qing Tombs have been left in Beijing and neighboring Zunhua and Yixian County. When studying on these mausoleums of feudal emperors, *Fengshui* ideas must be involved. To sum up in the simplest way, *Fengshui*, literally the wind and water in Chinese, means "storing the wind and getting the water". Guo Pu, a thinker in Jin Dynasty, mentioned in his *Zangshu* (Book on Funeral) that "When the *Qi* of *Yin* and *Yang* moves, it generates the wind; when it rises, it generates the cloud; and when it falls, it generates the rain. When the *Qi* is stored in earth, it is the vitality" and "The ancient people therefore collected and stored the

Qi, which is called *Fengshui*". The idea of *Fengshui* has a long history. The inscription on animal bones and tortoise shells in Shang Dynasty had recorded the investigation and prediction of the house or residence, while the *Book of History* also mentioned the investigation and prediction of the residence or the land. According to the *Rites of the Zhou*, "Tufang (an official rank in Zhou) applied the method of land investigation and measured the shadow of the sun to predict the appropriate land for residence for establishment of cities and fiefs, and to identify the proper plants for the land and teach local officials". It demonstrates that the special officials were assigned to investigate and predict on the land and residence as early as in Zhou Dynasty in China.

Fengshui is also known as "Kanyu". According to *Huainanzi Tianwenxun* (Analysis of Astronomy), "*Kan* comes with *Yu*, the male knows the female by voice, thus *Ji Chen*". In which "*Kan*" means the heaven, while "*Yu*" means the earth. *Fengshui* is also known as "geography". As mentioned in the *Book of Changes - Xici*, "Look up to observe astronomy and look down to investigate on geography". Wang Chong explained the word of "geography" in *Lunheng Ziji* (*On Balance - Self-Discipline*) that "Astronomy refers to the sun, moon and stars in the sky, while geography refers to mountains, rivers and valleys on the ground". Moreover, *Fengshui* is also called *Yin-Yang*. According to the *Book of Songs - Gong Liu*, "Observe the sun shadow and mountains to predict the *Yin* and *Yang*". *Shuowen Jiezi* (the Origin of Chinese Characters) gave the definition that "*Yin*, the shadow place, indicates the south of the water and north of the mountain; while *Yang* refers to the brightness". Therefore, *Yin-Yang* suggests the direction of the land, directing the sun or against the sun. *Fengshui* is also known as *Qingnang* (black bag literally in Chinese). According to the *Jiutian Xuannv Qingnang Haijiaojing – Preface* (Scripture of the Fairy of Ninth Heaven in the Black Bag), "The black bag contains the secret scripture that incorporates the eight diagrams and eight gates and the six combinations, and embraces everything of the world". *Fengshui* is also called *Qingwu*, named after Qingwuzi who was a master in selecting the residence in Yellow Emperor times. Zhang Junfang in Song Dynasty recorded the *Xuanyuan Benji* (History of Xuanyuan) in the *Yunli Qiqian* (Essence of Seven Taoist Books) that "When the Yellow Emperor defined the territory, he requested Qingwuzi to investigate on the geography to establish the system".

In addition to the long history, *Fengshui* has always been widely applied to the traditional architecture in China. The core of *Fengshui* is to determine the address, direction and layout of architecture, discuss on the relation between the nature and human and aim to realize good and prosperous life of people, render the safe and comfortable living environment and carry out production and social activities that satisfy people's material and spiritual demands in the safe and comfortable living environment.

8. Conclusion

The study on the traditional architecture culture in Beijing still consists of many other aspects in addition to those stated above, including the study on origin and changes of Beijing city, study on local chronicles of the city, counties and districts of Beijing, study on old town and cultural sites of Beijing and study on time-honored shops and landmarks in Beijing etc. All the study aims to explore the historical value and cultural background by researching on traditional architecture in Beijing preserved by now, thus to offer beneficial reference to transformation of the old town of Beijing and protection of ancient architecture.

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STUDY ON SEISMIC RESPONSE OF CANTILEVER STRUCTURE IN CONSTRUCTION OF LARGE CABLE STAYED BRIDGE PYLON

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Keywords: diamond shaped bridge pylon, long cantilever, dynamic analysis

Abstract

Based on the appearance of long cantilever during the construction of diamond shaped tower-of Cable-Stayed bridges with few researches available, numerical models are established to investigate the dynamic characteristics of cantilever structure during the different periods of construction based on the practical engineering of a large cable stayed bridge pylon. Results based on the different methods are compared for a convincing conclusion. Different length of cantilever and variable bracing condition are concerned based on the earthquake response spectrum data in the newest Chinese national seismic code with ABAQUS software for modeling. Vibration mode combination method and modal analysis are used to determine the reaction of cantilever during the different types of earthquake. The result shows that under the proper bracing condition, the constructing bridge pylon can sustain local rarely occurred earthquake. The conclusion of the research gives an important instruction for the design of such kind of cable-Stayed bridges and provide a practical theory for the certain type of project.

1 Introduction

Climbing Formworks are widely used during the erection of cable-stayed bridge pylon, which separate the work into different segments and improve the speed of erection significantly. In recent years, with the span of the newly-built cable-stayed bridges get larger and larger, higher pylon were built for the convenience to attach the cables. In this way, huge cantilevers are formed. A cable-stayed bridge pylon with diamond shape in construction, which have a cantilever for about 100m high during the erection, is taken as an example in this paper. Considering much of the cable-stayed bridges are built in seismic zone, it's important to determine whether this kind of cantilevers can survive strong earthquakes during the different stages of erection. In2003.8.14, The cable stayed-Rio-AntiRio Bridge which was under construction suffered an earthquake magnitude 6.2[1], called alert for the problem of seismic design for the erecting-pylon. In most of the climbing formworks' projects, Bracing bars are set between all the limbs of the erecting pylon to combine them together, it's important to find out how the bracing system works. To solve these two problems, an FEM model was built with the engineering data of our bridge with ABAQUS software, and works out how the tower and bracing system will react.

2. Theory and Numerical Model

2.1 Preview

In ABAQUS/Standard, Dynamic problems are usually solved by Modal Dynamic method, which is

a linear perturbation procedure and need to extract frequencies of the particular structure in advance, in our model, the first 100 eigenvalues were extracted for analysis.

The basic set of differential equations of the structural dynamic are shown below[2].

$$\mathbf{M}\ddot{y}+\mathbf{C}\dot{y}+\mathbf{K}y=F_p(t) \qquad (1)$$

Where M, C, K are mass, damping and stiffness matrix of the structure respectively. While $\ddot{y},\dot{y},y$ are acceleration, velocity and displacement vector, and $F_p(t)$ means the load vector added at the structure.

The main problem is that the $F_p(t)$ may be very complicated, which make the differential equations solving become impossible. this problem can be solved by using the orthogonality of the vibration modals and rewrite the displacement vector into the linear combination of the mode shape vectors. After that the equations will be uncoupled. And it's easy to solve each of them by Duhamel integral:

$$y(t)=\frac{1}{m\omega}\int_0^t F_p(\tau)\sin[\omega(t-\tau)]d\tau \qquad (2)$$

Where ω is the angular frequency of the modal.

To simulate the erection procedure of the tower, we cut the model of the pylon from the top with different diameter, leave two cantilevers with different length, for the simulation of bracing bars, we choose to join the two points from each cantilever together with all of their relative translation DOFs being restricted. The total procedure of modelling will be discussed below in detail.

2.2 Geometrical Features

Geometrical modelling is based on the constructing plan of the bridge, to make it clearer, some simplify was done, for example, it's assumed that each cantilever has invariable section area. The sectional view of xy plane are showed below in Fig1, where Dashed lines mean the cutting lines. The initial model indicated that the breach between the two cantilever parts are just blocked.In the First level 2m was cutted form the top to compare the difference before and after closure. Then, each line cuts 25m more to simulate the different stages of erection.

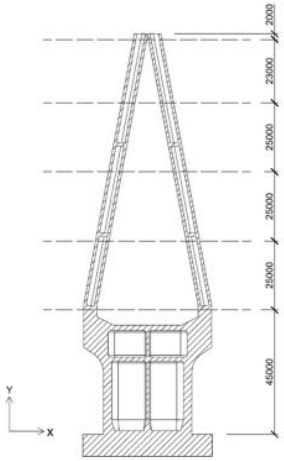

Fig.1 Cross section of the xy Plane

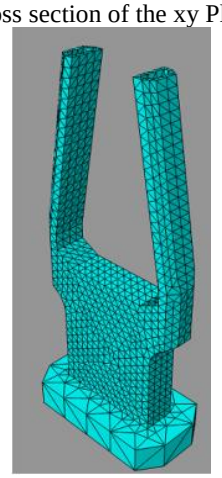

Fig.2 Mesh for the 50m-cutted tower

the model is meshed with C3D10 Element which is a 10-node quadratic tetrahedron in ABAQUS, and the base is meshed coarser to save the computing resource.

2.3 Material Properties

The materials of the bridge pylon are mainly reinforced C50 concrete. To integrate them together, the method in reference[3] and [4],is chosen which assume the reinforced concrete to be an orthotropic material with different mechanic properties in 3 orthotropic axes, the ratio of reinforcement is set to be 1% based on the construction plan, following engineering constants are worked out.

Table 1. Engineering Constants

Item	Value
E11	34740MPa
E22	39615MPa
E33	34740Mpa
ν12	0.178
ν13	0.207
ν23	0.178
G12	28750MPa
G13	28750MPa
G23	28750MPa

2.4 Loads and Boundary conditions

El Centro wave (NS) will be adopted, while primary calculation finds that only the data from former 30 second have a significant effect, so reaction after that will be ignored. For each cutting line, the model is calculated twice with load on X-direction and Z-direction respectively. The base is fixed to the ground.

2.5 Bracing bars

To find out how the bracing bars work, it's chosen to set 4 bars at the top and the center of the cantilever of the 50m-cutted tower. The two ends of the bar must have the same translation in all the three axes. The Reference Point are coupled with the inner face of the cantilever.

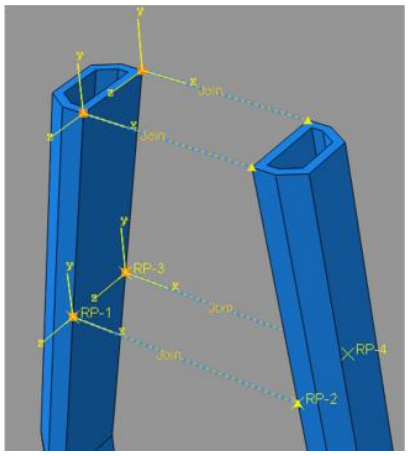

Fig.3Bracing condition

2.6 Summary of the Analysis Job

All the main factors this model discussed are summarized below with a different job name:

Table2. Job Summary

Name	Cutting level/m	Load direction	Bracing
Job-0-X	0	X	No
Job-0-Z	0	Z	No
Job-2-X	2	X	No
Job-2-Z	2	Z	No
Job-25-X	25	X	No
Job-25-Z	25	Z	No
Job-50-X	50	X	No
Job-50-Z	50	Z	No
Job-75-X	75	X	No
Job-75-Z	75	Z	No
Job-50-X -Link	50	X	Yes
Job-50-Z -Link	50	Z	Yes

the result will be discussed by job name for convenience.

3 Results

3.1 Data Collect:

For a dynamic analysis, we mainly focus on the displacements and the stress, and it's important to find out where is the crucial point for analysis, for displacement, it's obvious to take the top of the cantilever, and for stress, the bottom of the cantilever is taken.

We took the data and work out how these variables change with the time. In the flowing diagrams, $\sigma 1$ means the max principal stress, U1 means displacement in X direction and U3 means displacement in Z direction.

3.2 Diagrams and analysis:

Fig.4 U1 underX-load

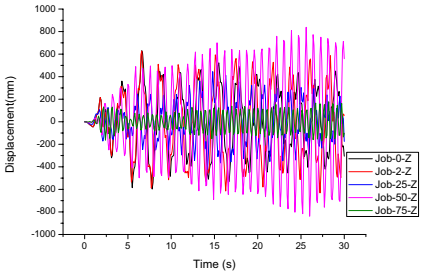

Fig.5 U3 under Z-load

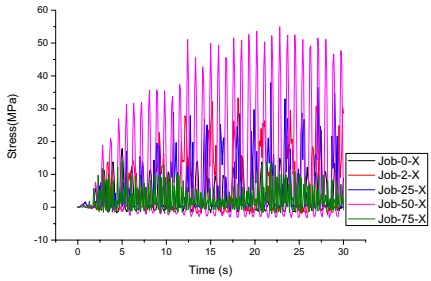

Fig.6 $\sigma 1$ under X-load

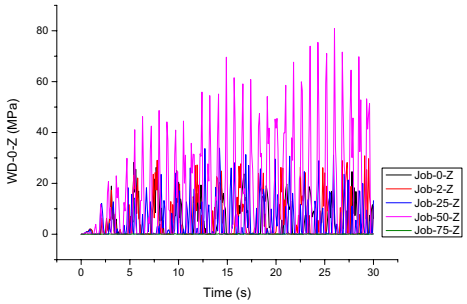

Fig.7 $\sigma 1$ under Z-load

As we can see, the displacement data fits the typical curve of the forced vibration, in X direction, the Job-2-X with the longest cantilever has the highest value of displacement. While in Z direction, the Job-50-Z with the medium cantilever length gets to the top. The stress data however, don't fit the curve. In Both direction, the 50m-cutted pylon has the highest stress.

To make it clear, the peak values of each data list is extracted and the following diagrams can be made:

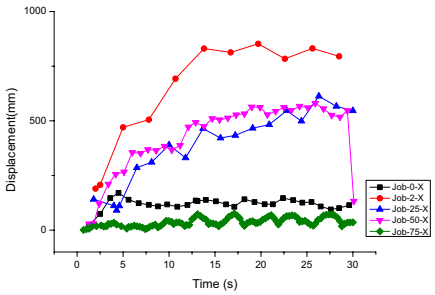

Fig.8Peak values ofU1 under X-load

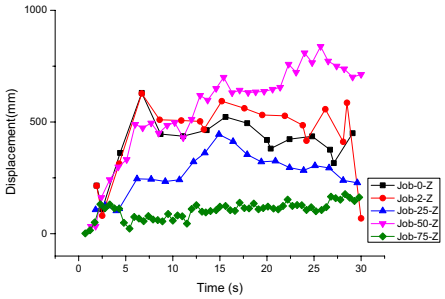

Fig.9 Peak values ofU3 under Z-load

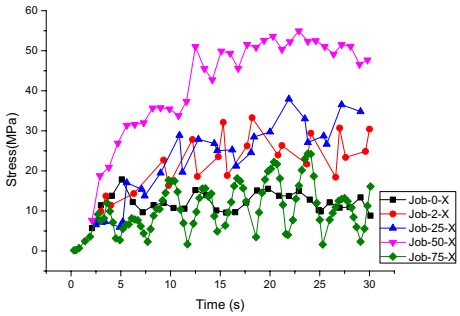

Fig.10Peak values of $\sigma 1$ under X-load

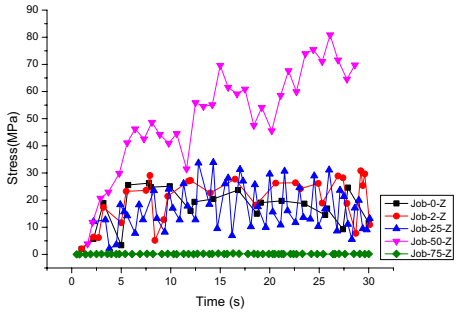

Fig.11Peak values of $\sigma 1$ under Z-load

Following diagrams are shown to compare the difference made by bracing:

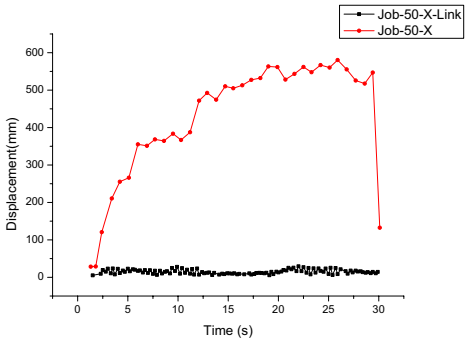

Fig.12Peak values ofU1 under X-load

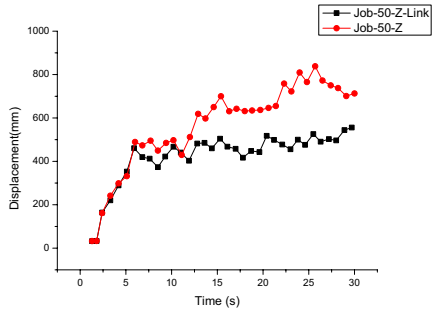

Fig.13Peak values ofU3 under Z-load

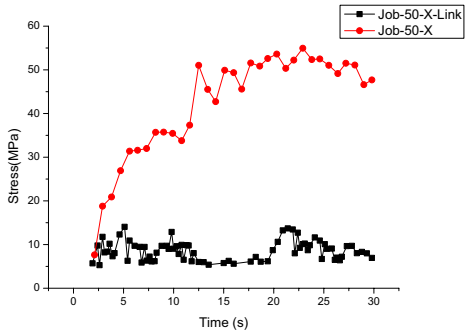

Fig.14Peak values of $\sigma 1$ under X-load

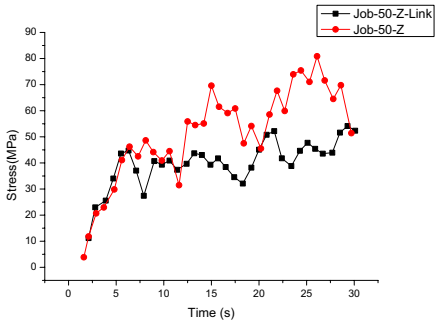

Fig.15Peak values of $\sigma 1$ under Z-load

From the data, following maximum values can be figured out:

Table3.Maximum values

Job Name	Stress /MPa	Displacement /mm
Job-0-X	17.8607	170.231
Job-2-X	33.2804	852.38
Job-25-X	37.9426	612.639
Job-50-X	54.9548	580.397
Job-50-X-Link	14.0565	29.448
Job-50-Z-Link	54.1477	555.845
Job-0-Z	28.1988	630.215
Job-2-Z	30.8189	627.706
Job-25-Z	33.8729	444.705
Job-50-Z	80.8779	838.256
Job-75-Z	0.3624	177.026

To our surprise, the highest maximum stress didn't appear at the bottom of the longest cantilever, but at the 50m-cutted tower with the medium cantilever length, that may cause by the sympathetic vibration.

For the discussion of bracing condition, it can be found that under the X-direction load, the stress and the displacement go down drastically after the link is added, but in Z-direction, when there is a load added, the stress and displacement don't have such kind of significant decrease.

4 Conclusion

Taking all the analysis into account, following conclusions can be drawn: (1) the stress and the displacement will not monotonically increase with the length of the cantilever, thus a critical length exists and depends on the shape of the earthquake wave. (2) The bracing bars have a significant effect when the earthquake load is parallel to the bars while help less when the load is perpendicular to them.

Acknowledgements

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STUDY ON TRANSITION STAGE OF NONLINEAR VIBRATION OF UNDERGROUND STRUCRURE UNDER VERTICAL SEISMIC ACTION

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Abstract : In this paper, a nonlinear soil dynamic model is used to study the dynamic response of rigid frame under vertical seismic action. The relationship between reaction force of soil and relative displacement of rigid structure $p=K_1w-K_3w^3$ is used, and the motion equation of the rigid structure is obtained by the dynamic D'Alembert's principle. The approximate analytical solution of the motion equation is obtained by means of the averaging method, and is analyzed in details. It is found that the nonlinearity of the model increases the period of vibration and the irregularity of the motion. Also, the seismic acceleration has a significant influence on the characteristics of vibration.

Keywords: Underground structure; Seismic action ; Approximate analytical solution ; Nonlinear vibration

1. introduction

Subways have the priority over other traffics with its advantages of savings the ground space, little influence on the urban environment, saving energy, being fast, punctual, convenient and safe. Comparing to above-ground structures, underground structures suffer less under seismic actions. So the seismic research of underground structures has not been taken seriously. However, the situation changed when the San Fernando earthquake and the Kobe earthquake in the 20th century damaged many subway stations and lifelines seriously. In the late 1960s, the construction of the bay area rapid transit (BART) in the United States San Francisco proposed that "underground structures are not against inertial force but should have the ductility to adapt the deformation of, surrounding medium, and only the vibration effect of tunnel should be considered under seismic actions".[1] In the 1970s, Japanese scholars have put forward the practical design methods, including response displacement method, strain method of surrounding rock and so on, based on large numbers of observations and experiments, which have been widely used. In the late 1980s and early 1990s, other methods, for instance, methods of Shukla[2] and ST.John[3], emerged.

Soil-structure interaction analysis methods can be divided into two categories[4]: the wave motion method and the structural dynamic method. The results of structural dynamic methods are relatively conservative and widely used. The simplified method usually includes two steps. The first step is to determine the free-field motion of soil. Then, the contact surface between the foundation and the structure is modeled as a series of independent springs. And the free-field motion of soil is input to determine the underground structure–soil interaction[5]. Considering the deformation of underground structure, analyzing underground structure-soil interaction is very complex. In order to simplify the analyzing, two steps can be taken[5]: in the first step, it is assumed that the structure is a rigid body and the dynamic soil-structure interaction is solved to obtain the motion law of the structure as a whole. In the second step, the motion law of the structure is input to analyze the soil-structure interaction with the consideration of the structural deformation. In this paper, only the first step, i.e., the dynamic response of the rigid structure will be solved.

In practical, the interface between soil and structures can be simulated by linear elastic springs in the case of small vibrations. Under strong earthquakes, the above model will give inaccurate results. Because soil and structures under strong earthquakes will behave visco-elasto-plastically, and the dynamic response of the structure will be non-linear. Therefore, consideration of the non-linear performance of soil will improve the calculation results.

It is generally believed that there are three kinds of nonlinearity in soil-structure interaction model[6][7]:

The first is the physical nonlinearity, that is, the nonlinearity of the material constitutive relation. The physical nonlinearity of soil includes the nonlinearity of stress-strain curve, the deformation accumulation and the hysteresis loop[8].

The second is the geometric nonlinearity, that is, the nonlinearity of the deformation when the large deformation occurs.

The third is the contact nonlinearity[9]. This nonlinearity is caused by the normal departure, closure, slip and friction between the underground structure and the surrounding soil [10]. So that the displacement of the structure and soil does not satisfy the deformation compatibility condition.

By using ABAQUS software and unified hardening constitutive model, considering the dilatancy and strain softening characteristics of the simulated soil, Xiuli Du et al[11] determined the influence of the non-linearity of soil behaviour on the dynamic response of the middle column. He[12] used finite element software to study the vibration and damage of the Dakai metro station considering non-linear foundation, and systematically analyzed the failure mechanism of the station. He also proposed the concept of "key supporting column". But his paper did not consider other factors such as spacing, size, etc. Using the ABAQUS software, Xujiang Fu[13] compared the results based on the nonlinear dynamic model and the pseudo static model. He also analyzed the effects of seismic intensity, structural stiffness and buried depth on the structural responses, and proposed some new suggestions. Grigorios Tsinidi[14] used ABAQUS software to analyze the effect of other factors, such as flexibility ratio, soil plasticity and nonlinear properties, on dynamic response of underground structures, but did not give quantitative equation.

In summary, the nonlinear dynamic soil-underground structure interaction is complex. The above analysis did not give quantitative description of the effect of different factors on dynamic response of underground structure, especially in the transition phase. In practice, the transition phase of nonlinear vibration is very complex. It is likely that peak may occur. Therefore, it is necessary to study it seriously. In this paper, we will consider the nonlinear properties of the interaction between soil and structure by using the average method which is good at solving the nonlinear motion equation approximately. We will carry out numerical analysis to study the influence of the nonlinearity on the maximum displacement, vibration frequency and the vibration law. Also we will reveal the basic law of nonlinear vibration of the underground structure in transition phase.

2. The establishment of the motion equation

2.1 Model simplification

Under the vertical seismic excitation, the force acting on structure and displacement are shown in Fig.1 The stresses acting on underground structures includes the normal stresses sustained by the roof and the floor, the frictional forces between the sidewall and soil, and the structural inertia force[15] Because of the fact that the structures are always stiffer than soil, this paper assumes that the structure is rigid, so the first step is to calculate the dynamic response of the subsurface structure under seismic action. Under vertical seismic excitation, the absolute displacement of the structure is $w(t)$, the absolute displacement of the soil is $w_g(t)$, the relative displacement between structure and soil is $y(t) = w(t) - w_g(t)$. The interaction between the structure and soil can be modeled by springs and the damping pots, which are the shear spring K_τ, the compression spring K_e, and the viscous damping coefficient C, respectively. The inertial force is equivalent to the static load applying to the structure. The analytical model is shown in Fig2.

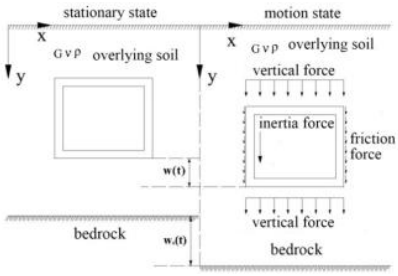

Fig1 Forces Acting on Structure

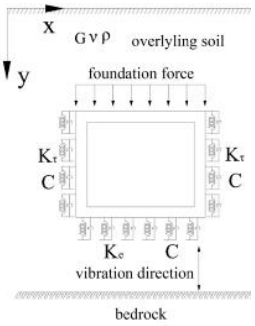

Fig.2 Analytical Model of Underground Structures

2.2 The determination of spring factor

In the model shown in Fig.2, the value of the spring coefficient will affect the final calculation results to a great extent and is closely related to the nature of the soil. The ground reaction is expressed by the Gorbunov-Posadov power series[5] in the following form

$$P(W) = k_0 + k_1 W + k_2 W^2 + k_3 W^3 + \cdots \qquad (1)$$

where P is force of the unit length of the foundation; W is the relatively displacement between the structure and the foundation. High accuracy can be achieved by considering several terms in Eq.(1). Reference[16] collected and analyzed a large number of experimental data to fit the relationship between the P/W and the W^2, where P is the load during the test. From Fig.3, it is clear that P/W is inversely proportional to W^2. So

$$P = k_1 W - k_3 W^3 \qquad (2)$$

where k_1 and k_3 are coefficients, with the unit of N/m^2 and N/m^4, respectively. Their values depend on the size, the bury depth of the structures and so on. Reference[17] indicated that the foundation spring coefficients k_1 and k_3 are proportional to the nonlinear properties of soil. Table

1 lists the approximate range of k_1 and k_3 for various soils.

Table 1 Recommended Values of Soil Moduli[16]

Type of Soil	$k_1(\times 10^7)$ N/m^2	$k_3(\times 10^{11})$ N/m^4
Loose sand	3.0-5.0	0-2.5
Clayey silt	4.0-20.0	0-100
Silty clay	7.5-25.0	0-100
Silty clay (soft)	1.0-10.0	0-40
Poorly cohesive soil	7.5-10.0	0-0.5
Cohesive soil	1.0-40.0	0-500

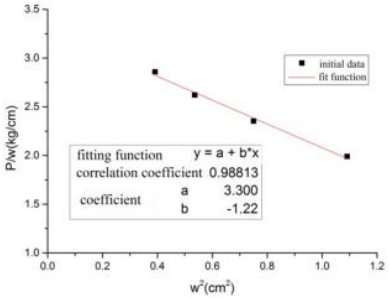

Fig.3 the Relationship between Loads and Foundation Settlement [16]

In this paper, the influence of the nonlinearity of soil on dynamic response of the structure is specifically considered. $K_{\tau 1}$ is the first term of the interaction between the sidewall and soil. $K_{\tau 3}$ is the third term. K_{e1} is the first term of the soil-structure interaction. K_{e3} is the third term. So, the sum of the coefficient are $K_1=K_{\tau 1}+K_{e1}$ and $K_3=K_{\tau 3}+K_{e3}$, respectively. C is the total damping which is between both the sidewall and the floor with soil. The overlying soil vibrates with the structure. M' is the total mass of the structures and the overlying soil. M^r is the mass of soil replaced by the structure. It is required that when recovered, the soil replaced by the structure should move identically with the surrounding soil[18]. So the effective mass is

$$M=\left|M'-M^r\right| \qquad (3)$$

According to the D'Alembert's principle, under vertical excitation, the motion equation of the structure is

$$M\ddot{w}+C(\dot{w}-\dot{w}_g)+K_1(w-w_g)-K_3(w-w_g)^3=0 \quad (4)$$

when $M\neq 0$, the equation is reduced to

$$\ddot{y}+\frac{C}{M}\dot{y}+\frac{K_1}{M}y-\frac{K_3}{M}y^3=-\ddot{w}_g \qquad (5)$$

For simplicity, the ground motion is taken as a simple harmonic vibration $\ddot{w}_g=\alpha\sin\theta t$, where α is magnitude of seismic acceleration, and θ is seismic frequency.

3. Solution of the motion equation

Eq. (5) is a generalized Duffing equation. In this paper, Eq. (5) is solved by the averaging method[19],[20]. First, the vibration equation needs to be dimensionless. For this end small parameter ε should be defined : $t^*=\frac{t}{T}$, $y^*=\frac{y}{Y}$, in which T is a characteristic time, $\frac{1}{T}=\omega=\sqrt{\frac{K_1}{M}}$; Y is the maximum displacement of the free-field. So Eq.(5) is transformed into

$$\frac{d^2y^*}{dt^{*2}}+\frac{CT}{M}\frac{dy^*}{dt^*}+\frac{K_1T^2}{M}y^*-\frac{K_3T^2Y^2}{M}y^{*3}=-\frac{\alpha T^2}{Y}\sin\theta Tt^* \qquad (6)$$

Define $\alpha^*=\frac{\alpha T^2}{Y}$, $\theta^*=\theta T$, $\varepsilon=\frac{CT}{M}\ll 1$, $\beta=\frac{K_3MY^2}{K_1CT}$; and $\gamma=\varepsilon\times\beta=\frac{K_3\times Y^2}{K_1}$ is non-linear characteristic parameter. Eq(6) can be rewritten as :

$$\frac{d^2y^*}{dt^{*2}}+y^*=\varepsilon\left(\beta y^{*3}-\frac{dy^*}{dt^*}\right)-\alpha^*\sin\theta^*t^* \qquad (7)$$

When $\varepsilon=0$ the model becomes linear, and the solution of equation(7) is:

$$\left.\begin{aligned} y^*&=a\cos(t^*+\varphi)-\frac{\alpha^*}{1-\theta^{*2}}\sin\theta^*t^* \\ \frac{dy^*}{dt^*}&=-a\sin(t^*+\varphi)-\frac{\alpha^*\theta^*}{1-\theta^{*2}}\sin\theta^*t^* \end{aligned}\right\} \qquad (8)$$

where a and φ are constants.

When $\varepsilon\neq 0$, a and φ in Eq.(8) are functions of t^*. It is assumed that the solution at this time has the same expression as the linear case. By taking derivative of the first equation of Eq. (8) with respect to time, and require that the obtained equation has the same form as the second equation of Eq.(8), we obtain

$$\frac{da}{dt^*}\cos(t^*+\varphi)-a\frac{d\varphi}{dt^*}\sin(t^*+\varphi)=0 \qquad (9)$$

By taking derivative of the second equation of Eq.(8) and substituting the obtained equation together with Eq.(8) into Eq.(7), we obtain:

$$-\frac{da}{dt^*}\sin(t^*+\varphi)-a\frac{d\varphi}{dt^*}\cos(t^*+\varphi)=\varepsilon\left(\beta y^{*3}-\frac{dy^*}{dt^*}\right) \qquad (10)$$

The solutions of Eq.(9) and Eq.(10) are

$$\left.\begin{aligned} \frac{da}{dt^*}&=-\varepsilon\left(\beta y^{*3}-\frac{dy^*}{dt^*}\right)\sin(t^*+\varphi)=\varepsilon\phi(a,\varphi,t^*) \\ \frac{d\varphi}{dt^*}&=-\frac{\varepsilon}{a}\left(\beta y^{*3}-\frac{dy^*}{dt^*}\right)\cos(t^*+\varphi)=\varepsilon\chi(a,\varphi,t^*) \end{aligned}\right\} \qquad (11)$$

where

$$\phi=-\sin(t^*+\varphi)\times\left\{\beta\left[a\cos(t^*+\varphi)-\frac{\alpha^*}{1-\theta^{*2}}\sin\theta^*t^*\right]^3+a\sin(t^*+\varphi)+\frac{\alpha^*\theta^*}{1-\theta^{*2}}\cos\theta^*t^*\right\}$$
$$=-\frac{a}{2}+\frac{a}{2}\cos 2(t^*+\varphi)-\frac{\alpha^{*3}\beta\cos\left(t^*-3t^*\theta^*+\varphi\right)}{8(1-\theta^{*2})^3}+\frac{3a^2\alpha^*\beta\cos\left(3t^*-t^*\theta^*+3\varphi\right)}{8(1-\theta^{*2})}\cdots\cdots \quad (12)$$

$$\chi=-\frac{\cos(t^*+\varphi)}{a}\times\left\{\beta\left[a\cos(t^*+\varphi)-\frac{\alpha^*}{1-\theta^{*2}}\sin\theta^*t^*\right]^3+a\sin(t^*+\varphi)+\frac{\alpha^*\theta^*}{1-\theta^{*2}}\cos\theta^*t^*\right\}$$
$$=-\frac{3a^2\beta}{8}-\frac{3\alpha^{*2}\beta}{4(1-\theta^{*2})^2}-\frac{3a\alpha^*\beta\sin(3t^*-t^*\theta^*+3\varphi)}{8(1-\theta^{*2})}+\frac{\alpha^3\beta\sin(t^*-3t^*\theta^*+\varphi)}{8a(1-\theta^{*2})^3}\cdots\cdots \quad (13)$$

In Eq.(11), ε is small parameter, therefore da/dt^*, $d\varphi/dt^*$ are slowly varying function of t^*. The main part of a and φ is also slowly varying function of t^*. In time integration over one period time interval, the slowly varying function can be looked as a constant. By averaging the two ends of Eq.(11) over the interval $[0\ T]$, one obtains

$$\frac{1}{T}\int_0^T\frac{da}{dt^*}d(t^*+\varphi)=\frac{da}{dt^*}$$
$$\frac{1}{T}\int_0^T\varepsilon\phi d(t^*+\varphi)=\varepsilon\frac{1}{T}\int_{-\varphi}^{T-\varphi}\phi dt^*=-\frac{a\varepsilon}{2} \quad (14)$$

$$\frac{1}{T}\int_0^T\frac{d\varphi}{dt^*}d(t^*+\varphi)=\frac{d\varphi}{dt^*}\frac{1}{T}\int_0^T\varepsilon\chi d(t^*+\varphi)$$
$$=\varepsilon\frac{1}{T}\int_{-\varphi}^{T-\varphi}\chi dt^*=-\frac{3a^2\beta\varepsilon}{8}-\frac{3\alpha^{*2}\beta\varepsilon}{4(1-\theta^{*2})^2} \quad (15)$$

Analyzing Eq.(12) and Eq.(13), when $\theta^*\approx 3$, $\theta^*\approx 1/3$, $\theta^*\approx 0$, the coefficients of variable t^* in ϕ and χ approaches 0. For instance, when $\theta^*\approx 1/3$, the third term of the right side of Eq. (12) becomes $-\frac{\alpha^{*3}\beta\cos(\varphi)}{8(1-\theta^{*2})^3}$. When calculating the average, it becomes constant. Taking the mean value, the right term of Eq.(14) becomes $-\frac{a\varepsilon}{2}-\frac{\alpha^{*3}\beta\cos(\varphi)}{8(1-\theta^{*2})^3}$. Similarly, when $\theta^*\approx 3$ and $\theta^*\approx 0$, the mean value varies comparing with the Eq.(14) and Eq.(15), after integration. It is called sub-resonance[20]. However $\theta^*\approx 0$ does not exist in reality. When $\theta^*\approx 1$, the denominator of fractional term of (12) (13) is small divisor. It is called main resonance[20].

Integrating the integrals of Eq.(14), Eq.(15) yields

$$a=e^{-\frac{\varepsilon t^*}{2}}a_0$$
$$\varphi=\frac{3\beta a_0^{\ 2}}{8}e^{-\varepsilon t^*}-\frac{3\alpha^{*2}\varepsilon\beta t^*}{4(1-\theta^{*2})^2}-\frac{3a_0^{\ 2}\beta}{8}+\varphi_0 \quad (16)$$

where a_0 and φ_0 are constant. By substituting Eq.(16) in the first form of Eq.(8), performing dimensionalization, we obtain the solution of the motion

$$y=a_0Ye^{-\frac{\varepsilon\omega t}{2}}\cos(\omega t+\frac{3\beta a_0^2}{8}e^{-\varepsilon\omega t}-$$
$$\frac{3\alpha^2\varepsilon\beta t}{4(\omega^2-\theta^2)^2}-\frac{3a_0^2\beta}{8}+\varphi_0)-\frac{\alpha}{\omega^2-\theta^2}\sin\theta t \quad (17)$$

When the initial displacement and the initial velocity are both 0, the equations for determining φ_0 and a_0 are as follows:

$$0=a_0Y\cos\varphi_0$$
$$0=-\frac{\alpha\theta}{(\omega^2-\theta^2)}-\frac{a_0Y\varepsilon\omega}{2}\cos\varphi_0-$$
$$Ya_0(\omega-\frac{3\alpha^2\beta\varepsilon}{4(\theta^2-\omega^2)^2}-\frac{3\varepsilon\omega\beta a_0^{\ 2}}{8})\sin\varphi_0 \quad (18)$$

Substituting different K_3 and seismic acceleration values into equation set (18), φ_0 and a_0 could be solved. Then the minimum value of φ_0 is 0.5π. a_0 is insensitive to the change of $\beta=K_3MY^2/(K_1CT)$, but increases almost the same times as the seismic acceleration α. Submitting φ_0 and a_0 into (17), the displacement equation of structure is obtained. From Eq.(17) it is clear that, when time is infinite, the term containing $e^{-\varepsilon\omega t/2}$ will vanish, and only the stationary phase of the forced oscillation is left.

When K_3=0, the equation degenerates to the equation for Winkler model, the approximate analytic solution of the Eq.(5) is[21]

$$y=e^{-\xi\omega t}\frac{\alpha\theta\sqrt{(2\xi^2\omega-\omega+\theta^2/\omega)^2+4\xi^2\omega^2}}{\left(\omega^2-\theta^2\right)^2+4\xi^2\omega^2\theta^2}\sin(\omega t+\phi')$$
$$+\frac{\alpha\sin(\theta t-\phi)}{\sqrt{(\omega^2-\theta^2)^2+4\xi^2\omega^2\theta^2}} \quad (19)$$

where $\phi=\tan^{-1}\left[\frac{2\xi\omega\theta}{\omega^2-\theta^2}\right]\approx 0$,

$\phi'=\tan^{-1}\left[\frac{2\xi\omega}{2\xi^2\omega-\omega+\theta^2/\omega}\right]\approx 0$。

4. The influence of nonlinearity on structural vibration

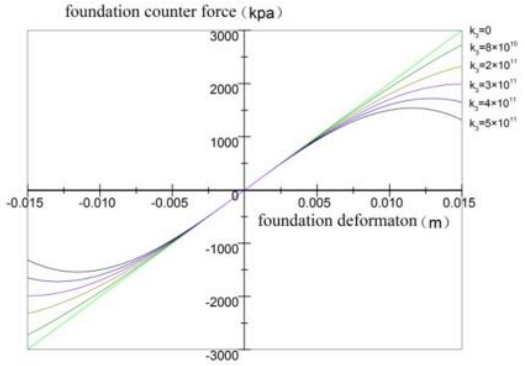

Fig.4 Subgrade reaction and displacement relationship

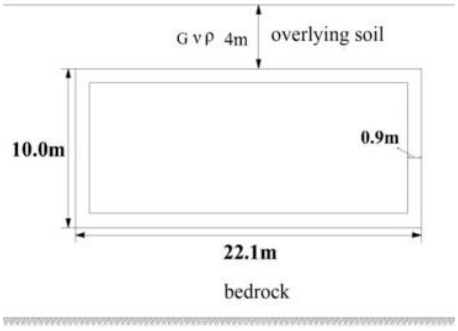

Fig.5 calculating model

We consider the motion of shallowly buried structure with rectangular cross-section. The depth of overlying soil is 4m, the width is 22.1m, the height is 10m and the thickness of the roof, floor, and exterior walls is 0.9m, as shown in Figure 5.

Concrete density is $\rho_s = 2400\,\mathrm{kg/m^3}$ and soil density is $\rho_s = 1800\,\mathrm{kg/m^3}$. The total mass of the structure and the overlying soil are $M' = 54.54\times2400 + 22.1\times4\times1800 = 2.9\times10^5\,\mathrm{kg}$, the mass replaced by the structure is $M^r = 22.1\times10\times1800 = 3.98\times10^5\,\mathrm{kg}$, so $M = 1.08\times10^5\,\mathrm{kg}$. The values of soil parameter, $K_1$and K_3, are as follows:

$K_1 = 1\times10^8\ \mathrm{N/m^2}$ (ω=30.43rad/s)

$K_1 = 2\times10^8\ \mathrm{N/m^2}$ (ω = 43.03rad/s).

In calculation, K_3 takes the following values 0N/m^4、8×10^{10}N/m^4、2×10^{11}N/m^4、3×10^{11}N/m^4、4×10^{11}N/m^4、5×10^{11}N/m^4, consecutively.

Subgrade reaction and relative displacement relationship is shown in Figure 4. In order to study the maximum displacement of the weak damping system, take the damping ratio as $\xi = 0.01$, and $\varepsilon = 0.02$.

For simplicity, the ground motion is taken as a simple harmonic vibration $\ddot{w}_g = \alpha\sin\theta t$ with α being 0.50m/s^2, 1.25m/s^2, 2.50m/s^2, 5.00m/s^2, respectively. We take the durability of seismic action as 2s, and the frequency as θ =50rad/s。

In the following, we study the effects of nonlinear parameter K_3 on the maximum displacement and the vibration frequency.

4.1. The influence of K_3 on the maximum displacement in transition phase

Regardless of the degree of non-linearity, relatively large displacement may occur in transition phase. Fig.6 shows the maximum displacements in transition phase with $K_1 = 1\times10^8\ \mathrm{N/m^2}$.

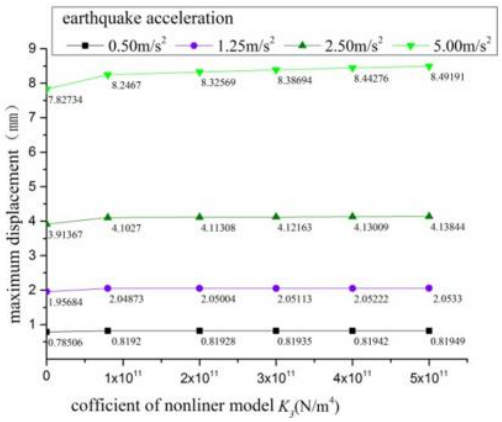

Fig.6 The calculated maximum displacements

When calculating the maximum displacement, the maximum displacement of linear model is calculated according to Eq. (19), and the maximum displacement of nonlinear model is calculated according to Eq. (17). For different seismic intensity and non-linear term coefficients, the maximum displacements of the underground structure are shown in Fig.6. It can be seen from Fig.6 that the maximum displacement increases with the increase of the nonlinear term for the same earthquake intensity, and the change is more obvious when the earthquake intensity increases. The reason is that when the nonlinear term coefficient increases, the equivalent secant modulus of soil decreases and the deformation increases for the same seismic acceleration. For example, when the seismic acceleration is 5.00m/s^2, K_3=0N/m^4, the equivalent shear modulus is 1×10^{8}N/m^2; when the K_3=5×10^{11}N/m^4, the equivalent shear modulus is 6.394×10^{7}N/m^2. It is clear from Eq. (17) that, when K_3 increases, β increases also. But β has little influences on the solution because of the coefficient of the first item in "cos" in the solution. So when K_3 increases, the maximum displacement will increase very slowly.

4.2 The Influence of K_3 on the Vibration Frequency of Structures

For study of the influences of K_3 on the vibration frequency, the liner model solution (19) of the motion equation is first expressed as the product of two terms with two frequencies, and then the equivalent secent frequency of the nonlinear structure is calculated to analyze the vibration frequency change with the

nonlinear term coefficient K_3. The linear model solution Eq.(19) is rewritten as follows

$$y = \frac{\alpha\theta e^{-\xi\omega t}\sqrt{(2\xi^2\omega-\omega+\theta^2/\omega)^2+4\xi^2\omega^2}}{\left(\omega^2-\theta^2\right)^2+4\xi^2\omega^2\theta^2} \times[\sin(Bt)\cos(At)+\cos(Bt)\sin(At)] + \frac{\alpha[\sin(Bt)\cos(At)-\cos(Bt)\sin(At)]}{\sqrt{(\omega^2-\theta^2)^2+4\xi^2\omega^2\theta^2}} \quad (20)$$

where $A=(\omega-\theta)/2$, $B=(\omega+\theta)/2$

The transition phase response is divided into two parts. One part is the motion with high frequency of vibration "B", and another part is the motion with low frequency of vibration $|A|$, which is the envelope of the first part. Frequencies $|A|$ and the "*B*" change with ω and θ. Considering the vibration of the structure in nonlinear soil, the equivalent frequency ω in equation (20) is

$$\omega' = \sqrt{\frac{K_1 - K_3 y^2}{M}} \quad (21)$$

To evaluate the influence of the nonlinear term coefficients, we take K_1=2×10^{8}N/m^2 and earthquake acceleration 5.00m/s^2 for example, the period of low frequency vibration is about 1s and the period of high frequency vibration is about 0.02s. For nonlinear term coefficient K_3=0 and 5×10^{11}N/m^4, the structure vibration is showed in Fig.7. In the transition phase, with the increase of nonlinearity, ω' and the high frequency $B=(\omega'+\theta)/2$ decreased and the period increases. Meanwhile, the low frequency $|A|=|\omega'-\theta|/2$ increases and the period decreases. In addition, the number of high frequency vibration contained in one lower frequency vibration decreases. This is due to the increase of nonlinear term coefficient, and the decrease of equivalent secant modulus. When K_3 is 0N/m^4, the equivalent secant modulus is 2×10^{8}N/m^2; when K_3 is 5×10^{11}N/m^4, the equivalent secant modulus is 6.394×10^{8}N/m^2.

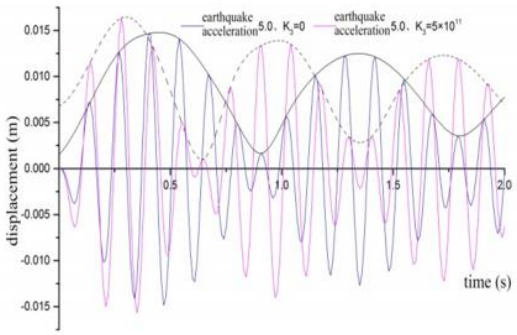

Fig.7 the Influence of Different Nonlinear Coefficients on Vibration Period

5. Conclusion

By considering the nonlinear properties of soil, the nonlinear vibration equation of underground structure is derived, and the approximate analytic solution of the motion equation is obtained by the averaging method. The influence of K_3 on the vibration of underground structures is analyzed. Once the seismic acceleration is fixed, by comparing the solution of motion equation of the underground structure in liner soil with that in nonlinear soil, the high frequency "B" decreases with the increase of the nonlinear term coefficient K_3 in transition phase, and the low frequency $|A|$ increases. In the same time, the amplitude of nonlinear vibration displacement increases. Therefore, in the design practice, it is necessary to consider the nonlinear vibration of underground structure in transition phase.

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Bearing capacity calculation of shear wall based on Artificial neural network system

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abstract Artificial neural network system is a data processing model inspiration from biological neural network. Artificial Neural network Process the data with a large number of connected artificial neurons, Artificial Neural network change their structure according to the input information, in that way it has the ability to solve practical problems. This article Introduces the basic theory of artificial neural network especially the RBF neural network system and give the source code of matlab that can predict the bearing capacity of shear walls by artificial neural network . This article adopted the experimental data of 15 pieces of different shear wall, use 11 pieces of shear walls experimental data as the training data, the rest of experimental data of 4 pieces shear wall was used as the test data and then analysis and comparative the experiment data and forecast data to prove that using artificial neural network system and the past experimental data to predict the bearing capacity of shear wall is viable.

Keyword: Artificial neural network; shear wall; bearing capacity

1. Introduction

Shear wall is the main component of a buildings to resist lateral force and the limit bearing capacity of shear wall is very important to the safety of a building. An experienced engineer can roughly known the bearing capacity of a structure without calculate, as long as he had seen a similar structure, No matter it is a concrete wall or concrete plate or concrete column because he has the previous experiences, when he see a similar structure he can determine the bearing capacity roughly by comparison with previous structure. Artificial neural network system is simulated to biological neural network when it deal with practical problems. neural network system has been get some use in the civil engineering but the field is not very extensive so we should do more explore and research, especially with the advent of the era of big data, large amount of experimental data accumulate will do good to the development of neural network system. This paper discusses the neural network system in the calculate of the shear wall ultimate bearing capacity.

2 Theory of Artificial neural network system

In the 1940s, psychiatry and anatomists McCulloch and Pitts put forward the neuron mathematical description and structure in the biological institute of physics journal, and proved that if there are plenty of simple neurons and these neurons are interconnected ,when they are operation at the same time, under that condition the neural network can calculate any function. After decades of development ,we have put forward different artificial neural network model to solve different problems.

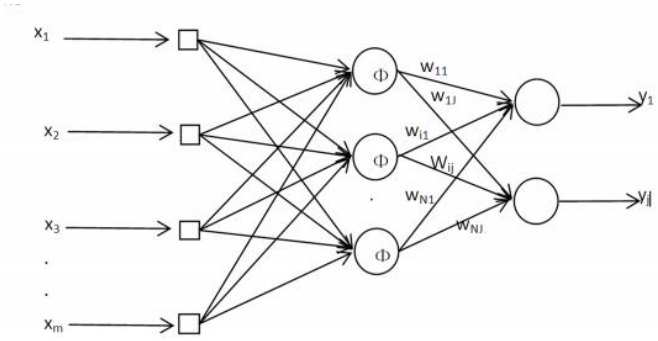
In the artificial neural network, the most important concept is the node function and Connection coefficient.. Neuron nodes make up the basic structure and the Connection coefficient can store a training or learning knowledge. nodes in the directed graph represent node function, how closely mutual connection between nodes is decide by the Connection coefficient. the most important thing of Artificial neural network is its connection coefficient that all can be adjusted according to the specific training data, a lot of simple nodes connected to set up a complicated data processing system. Artificial neural network training stage is the stage that adaptation of Connection coefficient. When the Connection coefficient adapt rightly the neural network system can reach the best

results. artificial neural network stored knowledge in the Connection coefficient, this is the most important thing of artificial neural network.

3、RBF neural network system predict the bearing capacity of the shear wall

Now The most common artificial neural network is BP neural network and RBF neural network. But the BP neural network learns slowly, not suitable for the demanding. 1988 Broom head and Lowe imitation biological neurons local response characteristics create a kind of artificial neural names BF neural network system. In many practice we found that the radial basis function neural network showed a very good approximation ability for nonlinear network. Slowly it got practical application in more and more field. BF neural network is made up of three layers it is forward network. The first layer is for variable input called the input layer. The number of variables equal the number of input layer nodes, the second layer is the data processing layer that called the hidden layer, the more complicated the question is the more hidden layer nodes will need. the last is the data output layer, The number of output data latitude equal the number of output layer node. the formula of Shear wall shear bearing capacity is $V\leqslant 1/(\lambda-0.5)[0.5f_tbw_hw+0.13NA_w/A]+f_{yh}A_{sh}h_w/s$ formula of bending strength is $M_u=A_sf_yl_w+A_{sw}f_{yw}l_w/2+Nl_w/2$ and there are a series of calculation formula was set up according to the shear wall's failure mode. we can see the complex function relationship between the variable and the bearing capacity from the all kinds of formula of the bearing capacity of the shear wall. it has the same formula if the failure mode are the same. And RBF neural network can approximate any continuous function. Then it can prove that RBF neural network can predict the ultimate bearing capacity of the shear wall in theory.

Assume: there are N training data, Connection coefficient is W_{ij} so regularization RBF network basic structure is as follows



The actual output is $Y_k=[y_{k1}, y_{k2}, \ldots, y_{kj}, \ldots, y_{kJ}]$, J is the number of output layer nodes, the real output of neural network of the first j neuron is as follows:

$$y_{kj}=\sum \omega_{ij}\phi(X_k, X_i),\ j=1,2...,\ j$$

Work out the connection coefficient With training date input, and then use the trained network to calculate.

4、Experimental data

This article selects the experimental data of Nan Jing university of technology institute of civil engineering Huailiang Chen paper as RBF neural network training and testing data to establish a radial basis networks. This network predict bearing capacity of the shear wall by comparing with previous data that is similar to the present. The detailed information of the shear wall is in the sheet 1. RBF neural network use 01 ~ 4 shear wall experimental data as inspection data、5 ~ 15 shear experimental data as training data.

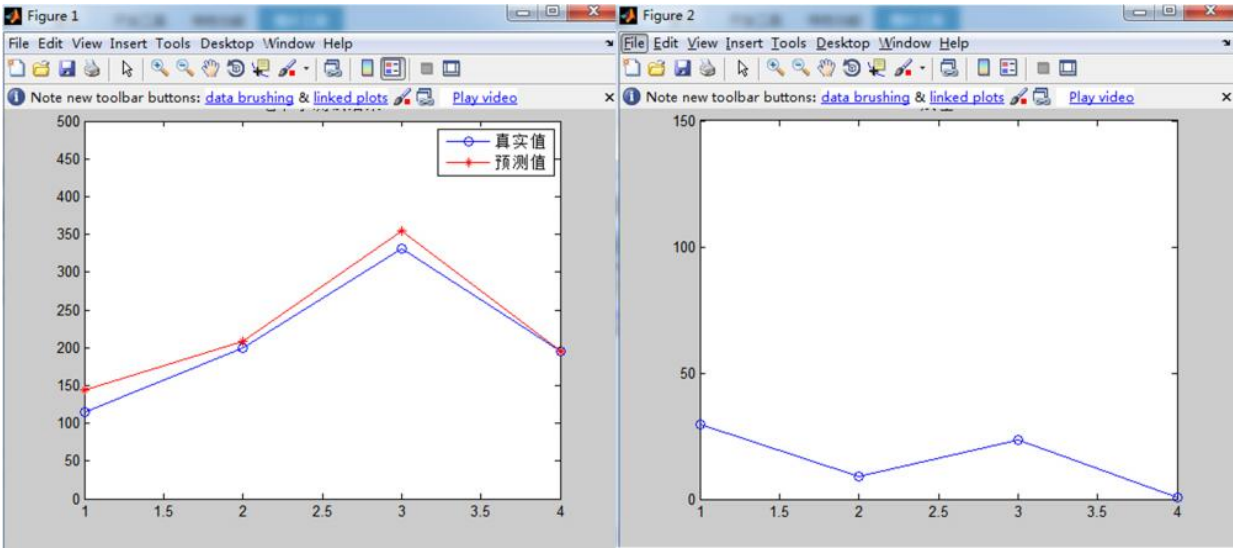
sheet 1

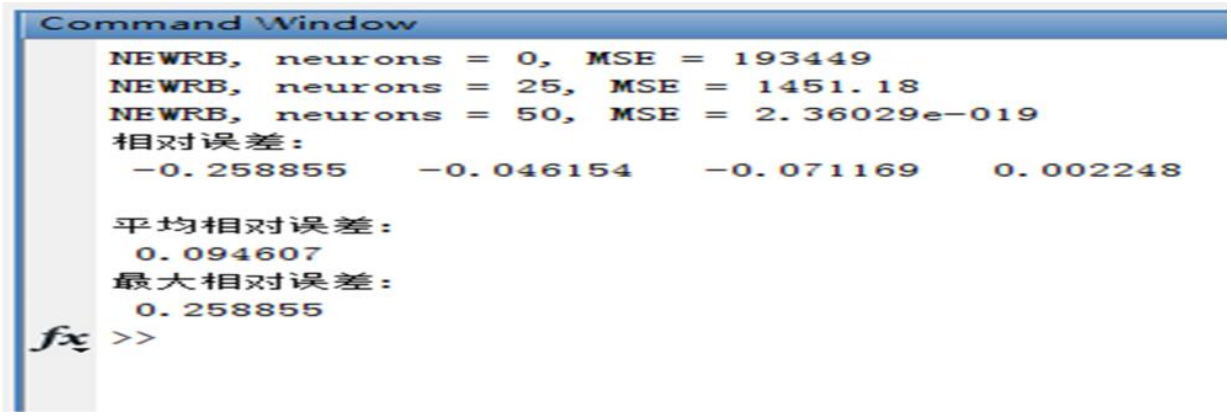
sampl e	b /	D /	h′/	l′/	t /	f_c /	f_y /	fy v =fyh	ρ	ρ_s	N_0 /kN	Qexp
i	mm	mm	mm	mm	mm	(N /mm -2)	(N /mm - 2)	(N /mm - 2)	%	%	KN	KN
1	120	120	400	600	25.9	34.3	376.3	283.9	0.89	1.25	0	114.7
2	120	120	400	600	26.0	42.3	361.4	284.9	1.98	1.25	0	198.8
3	120	120	400	600	25.1	38.5	356.1	285.9	5.52	1.29	0	330.6
4	120	120	400	600	26.4	42.5	361.4	286.9	1.98	1.23	0	195.9
5	120	120	400	600	28.0	40.2	343.4	287.9	3.53	1.26	0	261.0

6	120	120	400	600	26.2	43.2	356.1	288.9	5.52	1.24	0	362.4
7	120	120	400	600	24.7	42.9	361.4	289.9	1.98	1.32	0	188.9
8	120	120	400	600	22.6	37.9	343.4	290.9	3.53	1.44	0	269.4
9	120	120	400	600	24.5	44.5	356.1	291.9	5.52	1.33	0	326.0
10	200	200	970	1800	100.0	29.1	360.9	292.0	3.98	0.75	117.7	1380.0
11	400	200	970	1800	100.0	29.1	360.9	293.0	1.99	0.75	117.7	1360.0
12	150	150	800	1000	33.3	32.9	354.2	294.5	1.37	0.93	0	257.2
13	120	120	500	880	35.0	21.7	381.5	295.2	1.38	0.94	39.2	254.8
14	120	60	500	940	34.0	22.3	381.5	296.2	4.94	0.95	39.2	22.3
15	120	120	500	1380	33.0	23.5	381.5	297.2	2.47	0.95	39.2	295.9

5、 Matlab and RBF neural network

This paper use generalize RBF neural network system. In matlab we calls the generalized radial basis function (newrb) to calculate and following is the result：





picture one is the contrast between the real value and predicted values, picture two is the actual error value. picture three is the displays data in Command box. It can be seen that the largest relative error is 25.88%, the average relative error is 9.46%

It can be seen from the results that the RBF network can be used to predict the ultimate bearing capacity of the new shear wall based on the previous experimental data, the changing trends of the predict date is similar to the real values and the error is

acceptable In engineering. The reason of the error is that the training shear wall and forecast shear wall is not consistent, some are single row reinforcement and some are double row reinforcement. Some have axial pressure and some don't. In other hand the training data is too little and training data is leap too much, so using RBF neural network method to predict the ultimate bearing of shear wall has some error.

6、Conclusion

It can be proved that use the exist shear wall experiment data to predict the bearing capacity of the new shear wall is feasible, when calculating the bearing capacity of the new shear wall.in other word we can select exist shear wall experimental data as the training date of the neural network system and then predict the bearing capacity of the new shear wall with the trained neural network. Compared with the previous calculation formula of shear wall bearing capacity, we can see that using RBF neural network to calculate the bearing capacity of the shear wall don't need to take on any assumption, if the date is large enough we should be able to close to the actual real results. so we should pay more attention to collect all sorts of measured data in the future, and then we can class various types of data bases, set up all kinds of database. If we have done that work we can apply the big data thinking way in civil engineering.

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Experimental Investigation on the Punching Capacity of Recycled Concrete Two-way Slabs Strengthened with CFRP Laminates

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Key words： CFRP; recycled concrete; two-way slabs; punching Capacity

Abstract： Recycled concrete is a kind of green building materials, and using it in the buildings instead of normal concrete can not only deplete wasted concrete, but also reduce the exploitation of natural stone, so the environmental pressure will be relieved, large amounts of energy can be saved. Although scholars have done a lot of experiments and theoretical studies, these are mostly based on the basic mechanical properties of recycled concrete and there are few studies on structure design and application of recycled concrete. There will be 11 Recycled Concrete Two-way Slabs in this experiment, ten of which were strengthened with CFRP laminates by different types and numbers. Then we can prove the effect of different types and numbers on punching capacity of recycled concrete two-way slabs through loading vertically on the center of the slabs.

1. Introduction

With the Chinese modern economic development, construction industry develops rapidly, and outlook of cites increasingly update. But the traditional construction industry is at the cost of high consumption of resources and environmental pollution, but it‘s not consistent with the target of sustainable development and natural harmony, what’s more it will produce a large amount of construction waste. According to statistics energy consumption in the construction industry accounts for about 25-40 percent of total world energy consumption, and construction waste accounts for about 30-40 percent of municipal solid waste[1]. Now people have realized the seriousness of the problem, so more attention is paid to recycle concrete for it can recycled construction waste, reduce natural minerals exploitation, save energy. Although scholars have done a lot of experiments and theoretical studies, these are mostly based on the basic mechanical properties and durability of recycled concrete[2], and there are few studies on how to improve the Punching Capacity of Recycled Concrete Two-way Slabs. According to pasting CFRP in the negative moment area can improve punching capacity of concrete slabs[3], CFRP is short for Carbon Fiber Reinforced Polymer. In this paper, we will mainly explore the effect of different types and numbers on punching capacity of recycled concrete two-way slabs.

2. Test Introduction

2.1 Experimental Material

The recycled concrete strength grade used in this experiment is C40. Concrete mix proportion is designed according to <Code for design of recycled concrete structures>[4], the coarse aggregate replacement rate of recycled concrete is 40 percent, the mix proportion of recycled concrete is in the table 2-1. The diameter of intraplate two-way arranged rebar is 8mm, and steel grade of the rebar is HRB400, The diameter of the rebar arranged in the short columns is 8mm, and steel grade of the rebar is HRB400, and the diameter of the stirrup is 8mm, and steel grade of the rebar is HRB335.

Table 2-1 The mix proportion of recycled concrete（ kg/m^3）

Name	Water	Cement content	Coarse aggregate	Fine aggregate	Recycled coarse aggregate	Fly ash	Water reducer
Recycled	181	353	609	677	406	85	5.3

2.2 Test Scheme

We design 11 recycled concrete slabs in this experiment, ten of which are strengthened with CFRP laminates. To control variables all slabs are the same, and the layout and reinforcement drawing of all the slab-column structures is in figure below.

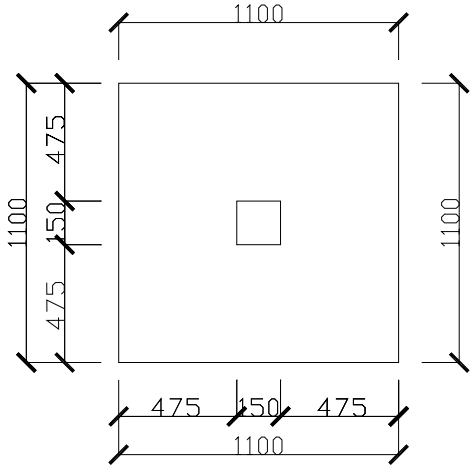


Fig.2-1 The layout of all the slab-column structures

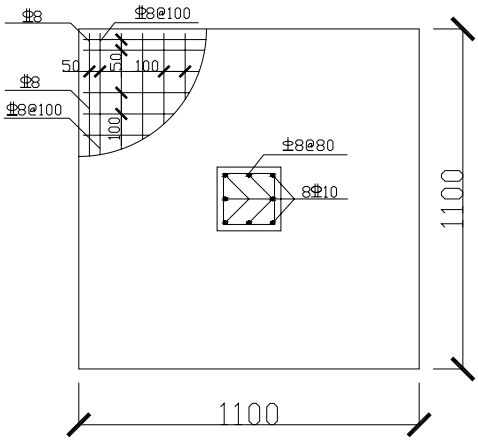


Fig.2-2 The reinforcement drawing of all the slab-column structures

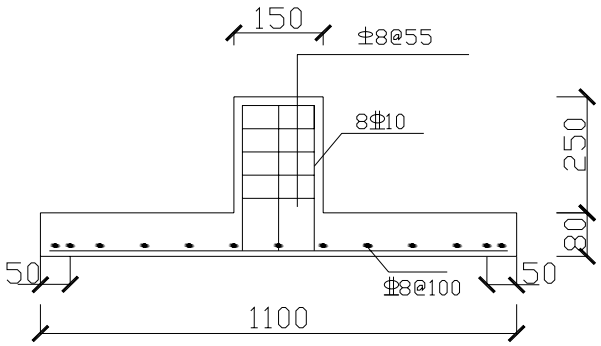
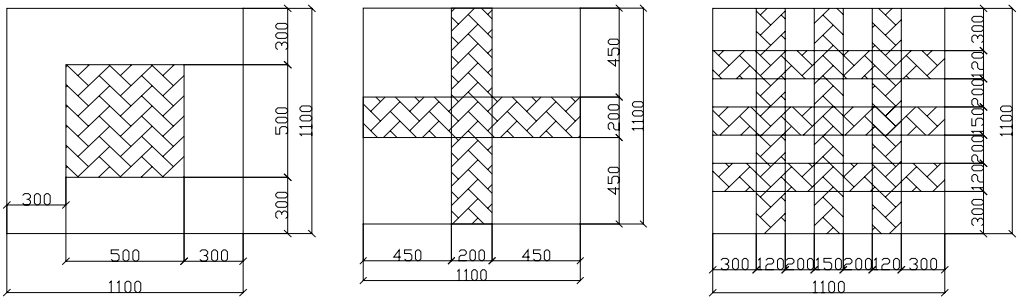


Fig.2-3 The profiles of all the slab-column structures

And the different ways of strengthening slabs with CFRP are shown below.



C-1 C-2 C-3

Fig.2-4 The layout of CFRP

Table 2-2 The layout and layer of strengthening slabs with CFRP

Member number	Layout	Layers

RCB-1	-	-
CRCPB-1-1	C-1	4
CRCPB-1-2	C-1	4
CRCPB-2-1-1	C-2	2
CRCPB-2-1-2	C-2	2
CFRPB-2-2-1	C-2	4
CFRPB-2-2-2	C-2	4
CRCPB-3-1-1	C-3	2
CRCPB-3-1-2	C-3	2
CRCPB-3-2-1	C-3	4
CRCPB-3-2-2	C-3	4

2.3 Experimental Device

The experimental device is assembled by counterforce device, steel beam, steel buttress, jack and data acquisition systems as shown in the Figure 2-5. According to <Standard Methods for Testing of Concrete Structures>[5], we use steel balls and rollers to simulate the slabs simply supported on four sides just like the figure below shown.

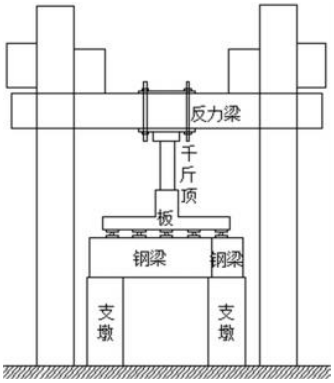


Fig.2-5The schematic drawing of test loading device

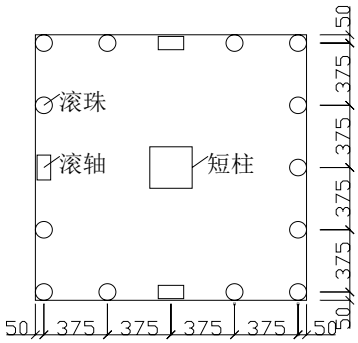


Fig.2-6 The layout of the bearing

Loading system: data acquisition device should be connected before loading and debugged through pre-loading, but pre-load should not be bigger than cracking load. Then load under multi-stage loading 10KN per level. To observe cracks and other phenomenon detailedly, load is applied 5KN per level.

2.4 Analysis of test results

2.4.1 Test results

Table 2-7 The punching capacity of the slabs strengthened by CFRP

Member number	Area of CFRP（ m2）	Punching capacity（ KN）	Increase (%)	Increase per unit area (%)
RCB-1	—	102.1	—	—
CFRPB-1	0.36*4	128.54	25.9	17.99
CFRPB-2	0.44*4	145.22	42.2	23.98
CFRPB-3	0.99*4	165.92	62.5	15.78
CFRPB-2*	0.44*2	122.71	20.0	22.96
CFRPB-3*	0.99*2	143.56	40.6	20.51

Postscript: The member number with * means slabs strengthened with 2 layers, and the others are strengthened with 4 layers.

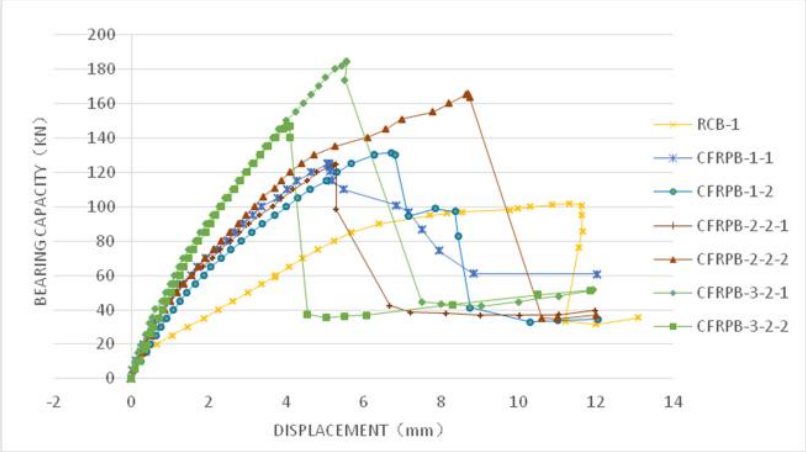


Fig.2-7 The load-displacement diagram of the loading point on the slabs strengthened with CFRP

We can summarize from the data above that strengthening in different ways has different effects with bearing capacity, stiffness, ductility of the slabs. The bearing capacity of RCB-1 is 102.1KN, and the average bearing capacity of CFRPB-1-1(2), CFRPB-2-2-1(2), CFRPB-3-2-1(2) which are strengthened with CFRP are 128.54KN、145.22N、165.92KN respectively, so we can conclude that strengthening with CFRP can improve the bearing capacity of recycled concrete slabs, and then we can figure out the increase of bearing capacity of slabs strengthened with CFRP is 25.9%, 42.2%, 62.5% respectively, so the layout of C-3 is best in increasing the bearing capacity. Considering the use efficiency we can see from table 2-7 that the layout of C-2 is the best. And we can find that the slope of load-displacement curve of slabs strengthened with CFRP increases, so the stiffness of slabs increases, but the ductility will decline, the displacement reduces when the punching failure happens.

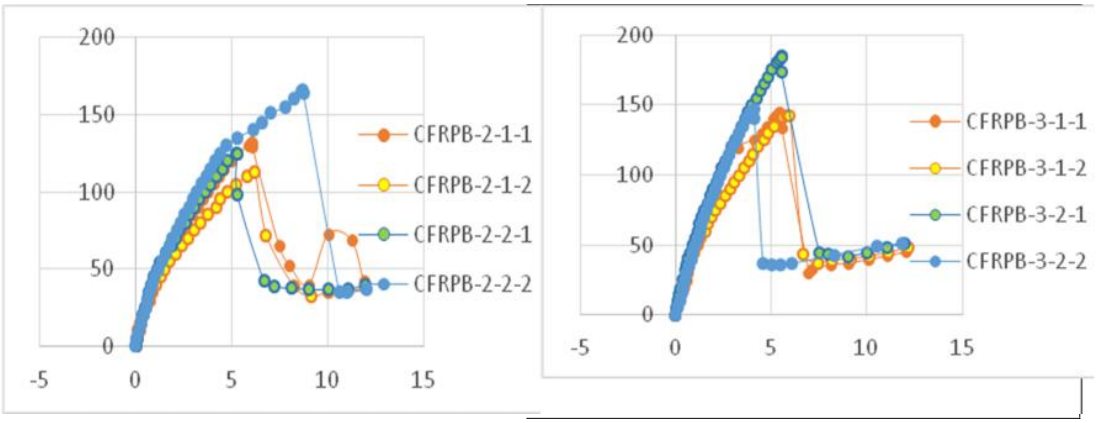


Fig.2-8 The load-displacement diagram of the loading point on the slabs strengthened by different layers of CFRP

We can summarize from figure 2-8 that the bearing capacity and stiffness of slabs increases as the number of CFRP layers increases, while the ductility declines. And in addition, we further discover from table 2-7 that the number of CFRP layers has the biggest influence on the bearing capacity of slabs strengthened with CFRP in the way of C-1. In practical engineering construction requirement, economy request and bearing capacity requirements should be considered before choosing the layout and layer of CFRP.

2.4.2The process of punching

The process of punching can be roughly divided into three phases:

The first stage: this stage starts from the beginning of load to the moment cracks appear, the slabs perform elastically, and load-displacement curve is a straight line. When the load turns to 40%-50% ultimate load, cracks start to appear at the bottom of the slabs. Most of the early cracks develop from the center to supports, and then some cracks which are parallel to sides of column arise around the area of column. It is because tensile stress of recycled concrete on the bottom of slabs exceeds the ultimate tensile strength.

The second stage: this stage starts from the appearance of cracks to punching failure. With the increase of load, cracks begin to develop, the length and width of inclined cracks which extend to support at last increases; the cracks which are parallel to sides of column develop and connect to each other, and turn to a closed circular cracks at last. During the whole process, CFRP begins to work when the load increases. CFRP of C-1 mainly limit the development of cracks at the bottom of the slabs, when we cut down the CFRP after experiment, we can find that there are few cracks in pasting range, so we know CFRP extends the bottom area of shearing-awl to increase the bearing capacity; CFRP of C-2 mainly increases the bearing capacity by increasing slabs' flexural capacity, and limiting the development of cracks in pasting range is also helpful to increase the bearing capacity; CFRP of C-3 mainly increases the bearing capacity by increasing slabs' flexural capacity and tensile bearing capacity, then flexural capacity and Stiffness increases significantly, and deformation reduces, the increase of bearing capacity is the highest.

The third stage: when punching failure happen, failure mode of CFRPB-1 is the same as the punching failure of slabs without CFRP, column goes down rapidly, shearing-awl forms, component damages. Failure mode of CFRPB-2 and CFRPB-3 is similar, but after punching failure CFRP laminates have effects on limiting shearing-awl and reducing concrete pieces to fall.

3. Conclusion

Based on the phenomena observation and data analysis of the punching capacity experiment of recycled concrete two-way slabs strengthened with CFRP, the paper draws several conclusions as following:

(1) According to the observation from the experiment, , recycled concrete slabs strengthened with CFRP show a similar failure process to the concrete slabs ,they all can be divided into elastic working stage, crack development stage and destruction stage.

(2) The punching capacity of the recycled concrete slab got significant improvement after strengthened with CFRP. The increase rate varies from 20% to 60% according to the different layouts and layers. The stiffness of the slab also increased resulting in decreasing deformation after getting loaded and almost no cracks developed within the range strengthened by CFRP.

(3) With the condition of the same layers, the layout of C-2 shows the maximum efficiency in enhancing the punching capacity of the recycled concrete slab.

(4) With the condition of the same layout, the punching capacity and stiffness of the recycled concrete slab increase as the number of CFRP layers increases.

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Author Index

A

Aloyan Artyom A. 356

Ambrosova Galina 106,112

Arzumanyan Artavazd 92

Arzumanyan Avetik A. 17

Avetisyan Siranush.....58

Arsenyan Lilit 342

B

Baghramyan H.M. 15

Balanchivadze Lia 388

Barseghyan M.G. 15

Bediashvili Malkhaz 426

Bing Xie 283

Bliadze Seit 385

Bowen Zhang 463

C

Chachava N. T. 378

Chanturia Irakli 388

Chang Yuan 475

Chao Wang 42,64

Chavleshvili Gocha 151

CHEN Huifang 278

Chiyun Zhao 283

D

Dondoladze Nana 83

Drozdova Irina 47

Du Chenxu 479

Dzodzuashvili Ucha 385

E

Efimov Viacheslav 424

Evtyukov Sergey 41

F

Fan YANG 47

Faresishvili Akaki 85

Feng Xiaowei 288

Funk Anna 112

G

Gabrichidze Guram 151

Galstyan Gagik 92

Gao Mingcong 60

Gasparyan Anush 73

Gasparyan Marietta 437

Geradze Paata 162

Gevorgyan Armine.....82

Ghazaryan Marine 113

Gigineishvili David 397

Gigineishvili John 397,404

Golovina Svetlana.....52

Grigoryan A. 104

Grigoryan Ashot 66

Grigoryan Marine 97
Grigoryan Nune 58
Grigoryan V. I. 89
Grigoryan V. V.....89
Grigoryan Vahan 66
Grigoryan Vardges 66
Guangda Zhang 216, 317
Gulyan Armine 129
Yunpeng Guo 468
Gurgenidze David 375,385
Guzik Marcin 29
Grigoryan Armin 28

H

Hakobyan Gevorg 137
Hakobyan Mariana 82
Harutyunyan Anahit 129
Harutyunyan Emma 91
Hayrapetyan Yeghisabeth.....85
Hongzhan Shi 48
Hou Songsong 291
Hoveyan Hovsep 92
Huang Weizhen 56

I

Ivanov Dmitry 13

J

Janelidze O. 412
Jun Dong 125,167,172,176,181,189, 193, 200,205,258,265,271,278, 312,326, 463
Jończyk Damian 25

K

Kalabegishvili Miriani 3
Kanetsyan Aminia 336
Kanetsyan Eghine 117
KANG Chunxiang 68,265
Kochar Karine 22
Karapetyan Lusine 145
Kartashyan Tatev R. 356
Khachatryan Artur 137
Khachatryan Emil 137
Khachiyan Eduard 107
Kharshiladze Oleg 385
Khmelidze Kakhaber 375
Khalatyan Sargis 104
Khmelidze Tamaz 375
KHOKERASHVILI Zaza 390
Khoshtaria T. K. 375
Khudoyan Anush 359
Kikava Teimuraz 416
Kikvidze Omar 165
Kiowski Artur 29
Kipiani Gela 430
Kipiani Parmen 162
Kirakosyan Lyuba V. 101
Klimiashvili Levan 375,385
Knyazyan Shakespeare.....73
Kochar Karine.....434
Kondrateva Lidia..... 424

Korzhova Antonina 102
Kot Yauheni 103
Králik Juraj 105
Kruglikova Anastasiia 106
Kurzak Lucjan 19

L

Lanchava Omar 390
Larionova Viola Anatolievna 419
Laroze D. 15
Lausova Lenka 9
Levonian Levon 107
LI Bo 181, 205
Li Chenguang 298, 301
Li Guohua 54
Li Qinglei 4,61,142
Li Dongqing.....137,167,172,176,258, 321,326
Li Youlu 243, 248
Liang Xu 458
Liu Anran 227
LIU Dianyuan 193,200, 205, 312,463
Liu Fei 125,291
Liu Jun 52
Liu Jia.....475, 479
Liu Yingqi 61
Li-yang ZHANG 46
Luo Jian.....475,479
Lu Zhang 283

M

Ma ShunChang 181,326
Ma Tianlei 298
Machaidze Edisher 133
Magradze Tinatin 13
Maisuradze Rusudan 118
Manzi Jiujin 475
Margaryan Albert 82
Markosyan Mher 113
Matsaberidze Temur 124
Matyushenko Evgeny 112
Medzmariashvili Elguja 151
Megrelishvili Zurab 423
Minadze Elguja 165
Mirnyy Anatoly 34
Mkhitaryan Suren.....117
Mkrtchyan Musheg 117
Moseshvili Tamar 447

N

Nikabadze M.U. 71
Nozadze Giorgi 390
Nazaretyan Artavazd S. 345

O

Obgadze Tamaz 8
Okropiridze G. 100
Okroshidze Z. 99
Osokin Anatoly 424

P

PANG Jianrong 271

Partskhaladze Gaioz 151

Pegin Pavel 119

Peiheng Long 216

Petrosya Hovseph.....85

Pirumyan Narine 90

Panin Alexander 37

Perov Fedor 37

Poghosyan Manvel 104

Q

Qi Chengzhi 301,306,61,142,468

R

Rajczyk Jarosław 16

Rajczyk Marlena 25,29,120

Rajczyk Paweł 29

Ramzaha Vedna 29

Ren Chunguang.....181, 193, 200, 312

Ren Ren 125,167,172, 176,258

Repelewicz Aleksandra 369

Repin Sergey 41

S

Sadunishvili M. 3

Safaryan Ashot Yu. 348

Safaryan Yuri A. 348

Sahakyan Emma 63

Sargsya Emilia M. 351

Sargsyan Artashes 137

Sarukhanyan Arestak.....69

Shahinyan Samvel 329

Zhang Liyang.....181. 189

Shao qin WANG 47

Shen Siran 468

Shesterov Evgeny 47

SHI Guosong 278

SHI Hongzhan ...189,193,200,205, 312,326,463

Shun-Chang MA 45

Skotnicova Iveta 9

Slyusarenko Yuriy 136

Smirnov Evgeny 47

Sołkiewicz-Kos Nina 115

Stachecki Bartłomiej 25

Stepanova Natalia Romanovna 419

Stepanyan Sargis 137

Sulshvili Vzha 100

T

Ter-Martirosyan Armen 34

Ter-Petrosyan Gevorg..... 123

Timchenko Igor 397,404

Tokunova Galina.....52

Torosyan Arman 75

Tsikarishvili Malkhaz 385

V

Voskanyan Viktoria P..... 351

W

Wang Chao 176,258

Wang Hongsen 301, 306

Wang Liang 243,248,59

Wang Siqi 248

Wang Yonggang 25,167,172, 176,258

WANG Weihao 271

Wang Xiufang 60

Wang Yan 227

Wenfa Yu 216

Wu Shuang 222

X

XIANG Xuejian 193,200,205, 312,463

Xu Wang 65

Xuan Wu 452

Xu long 236

Xie Long 321

Y

Yedoyan Rafik 78

Yedoyan Tatevik 78

Yedoyan Vardges 97

Yi Weitong 243, 248

Yinqi Liu 306,142

YU Jianfu 265

Z

Zadworny Mariusz 362

Zambakhidze Leri 95

ZHANG Hong-Bing 212

Zhang Yanxia 227

Zhang Hexin 227

Zhao Chiyun 288

Zhao Dongfu 222,55

Zheng Mingzhao 56

Zhu Chunjie 291

Zhu Huating 301

Zurashvili Nino 158

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