DRUŠTVO ZA ISPITIVANJE I ISTRAŽIVANJE MATERIJALA I KONSTRUKCIJA SRBIJE

SOCIETY FOR MATERIALS AND STRUCTURES TESTING OF SERBIA

INTERNATIONAL CONFERENCE MARKING 60 YEARS OF OPERATION OF DIMK

RESEARCH IN THE FIELD OF BUILDING MATERIALS AND STRUCTURES

MEĐUNARODNA KONFERENCIJA POVODOM OBELEŽAVANJA 60 GODINA RADA DIMK SRBIJE

ISTRAŽIVANJA U OBLASTI GRAĐEVINSKIH MATERIJALA I KONSTRUKCIJA

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Građevinsko-arhitektonski fakultet Univerziteta u Nišu, Niš

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Scientific committee:

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S A D R Ž A J

F O R E W O R D

These Proceedings were prepared on the occasion of marking 60 years of operation of the Society of Testing and Research of Materials and Structures of Serbia – DIMKS. The work of this Society is a continuation of the operation of the Association of Yugoslav Laboratories – SJL, that is, later of the Yugoslav society for Research and Testing of Materials and Structures – JUDIMK.

The association of Yugoslav laboratories was established on the inaugural meeting which was held on 10th of May 1952 in Belgrade. Thirteen eminent experts from Belgrade, Zagreb, Ljubljana and Sarajevo took part in the work of the inaugural meeting, and they represented twelve institutes, laboratories and faculties. Engineer Milutin Maksimović was elected chairman of the Association, who was the head of the 1st department of the Institute for Material Testing of the Serbian Academy of sciences. The basic organizational concept of SJL was based on collective and individual membership, and it remained such until the present day. Immediately after having been established, the SJL proceeded with the attracting as many laboratories and institutes to join it. In this period, around 40 potential collective members were registered, including the laboratories operating within the companies and on the construction sites. Initially, only ten laboratories answered the call to join in.

As early as at the inauguration meeting, the ambition was made clear to make SJL as similar to RILEM as possible, both in organizational terms and in terms of its activities. Therefore, at the inauguration meeting, a standing deputy and representative of SJL in RILEM was elected. Such actions resulted in becoming a member of RILEM on 12th September 1952 at the congress in the Hague. It should be emphasized that at the XI Congress held in London in July 1958, professor of the Technical Faculty of Sarajevo, Julije Hahamović was elected the president of RILEM. This created even more favorable conditions for the work of SJL, which resulted in holding the XII Congress of RILEM from 6th to 12th September 1959 in former Yugoslavia. The congress was attended by the representatives of 25 countries, and the activities of the Congress took place in Belgrade, Zagreb and Ljubljana with numerous and substantial accompanying activities.

Apart from being a member in RILEM, SJL was the member of the Association of Civil engineers and Technicians of Yugoslavia (SGITJ). In this sense, there is certain continuity until now, as DIMK is also the member of SGI of Serbia. With its activities in the society, primarily in the field of improvement of civil engineering, the Association became a very important factor in the circle of engineering organizations.

For the purpose of affirmation of achievements and work in the field of research of materials and structures, SJL initiated publishing of the journal „Bilten" (Bulletin). The decision was made at the VIII annual meeting held in the month of May 1957 in Sarajevo. At the XI assembly meeting of SJL held in Belgrade on 27th April 1963, three years after holding the X assembly meeting in Zagreb, a proposition of the editorial board of the "Bulletin" was accepted to change the title of the journal into „Materijali i konstrukcije" (Materials and Structures). Simultaneously with the publishing activity, the association initiated holding of scientific-professional conventions, and so on a bygone 12th of June 1958, a conference was held in Belgrade under the title "Materials and *structures in housing construction". Organization of symposia, conventions, round tables and other forms of propagation of scientific thinking became a permanent form of activities of the society and survived till the present days. In the further text, there is a list of the most important meetings:*

- *„Conference on some issues in civil engineering", held by the beginning of February 1965 in Vrnjačka Banja.*
- *„Conference on quality and assortment of domestic cements", held in April 1980 in Trogir.*
- *„Conference on the standards for clay products", held in December 1988 in Novi Bečej.*
- *Seminar „Masonry walls, floor slabs and pressed clay tiles", held in April 1992 in Aranđelovac.*
- *Conference „Engineering structures testing", held in December 1993 in Belgrade.*
- *Conference „Materials and carriageway structures of roads", held in December 1994 in Belgrade.*
- *Conference „Construction of additional floors of housing and public buildings", held on December 2000 in Belgrade.*
- *Round table „Condition and perspectives in production of basic civil building in Yugoslavia", held on June 2001 in Belgrade.*
- *Conference „Masonry structures in contemporary civil engineering practice", held on December 2001 in Belgrade.*
- *Conference „Building physics and materials " and the monograph of the same title, October 2003, Belgrade.*
- *Scientific-professional meeting „Harmonization of national and European regulations in field of concrete technology compliant with the standard EN 206-1:2000", October 2004, Belgrade, April 2005, Niš and May 2005, Novi Sad.*
- *Scientific-professional meeting „European regulations in the field of clay and calcium silicate elements in masonry structures – EUROCODE 6 and accompanying regulations", December 2006, Belgrade.*
- *Scientific-professional meeting „Elements of concrete based on light and normal aggregate of autoclaved aerated concrete, elements of artificial stone and elements of natural rocks such as EUROCODE 6 – part 3 – simplified rules for design of masonry structures", May 2007, Belgrade.*
- *Conference „Civil engineering and sustainable development", June 2009, Belgrade.*
- *Round table „Discussions on the application of clay products in masonry structures", December 2009, Belgrade.*
- *Conference „Masonre structures – bearing capacity, durability and energy efficiency", November 2010, Beograd.*
- *Conference „Aplication of clay products in construction of energy efficient buildings", April 2012, Beograd.*

Particularly important activity of SJL had during 1963 in relation to the catastrophic earthquake in Skopje. In the background of intensive works on mitigation of the consequences of catastrophic devastation the experts o of SJL from the institutes from Skopje, Belgrade, Zagreb, Sarajevo and Ljubljana recorded and later processed by scientific methods, and studied the collected material on earthquake and its *consequences. This gave rise to an extensive report which was a basis for making Temporary technical regulations fro building in seismic areas.*

As early as on the occasion of its founding in 1952, SJL made the work on production of various standards for testing of materials one of its main goals, irrespective of those were national or foreign standards. Later, this activity extended to entire civil engineering.

Long ago, in 1960 at the assembly meeting in Zagreb, SJL decided to form expert committees within the Association as a competent and impartial professional organization which would be formed from the experts coming from the scientific organizations, faculties, companies and state administration in order to produce the technical regulations. Thus, during the sixties and the beginning of seventies of the previous century, a large number of Yugoslav technical regulations was assembled, as well as large number of accompanying standards (JUS). Here will be listed some of the regulations that were worked on: Regulations in area of steel structures (1961.), Yugoslav technical regulations for aseismic building (1964), Study on concrete corrosion (1963).

In the period form 1967 to publishing in 1970 and entire set of regulations were worked on - for: presetressed concrete, concrete and reinforced concrete, thermal insulation, acoustic insulation, timber structures, composite structures, assembly of steel structures and protection of steel structures from corrosion (total of 9 regulations). This extensive was successfully completed engaging eminent experts from the institute, faculties and other relevant organizations from the area of all former Yugoslav republics.

Particularly interesting is the engagement on production of regulations in the field of reinforced and prestressed concrete which were published first in 1971 – Code BAB 71, which was later changed, and enacted as Code BAB 87 issued by JUDIMK and SDGKJ.

At the beginning of the eighties, JUDIMK started cooperation with the Federal Institute for Standardization with the purpose of establishment and formation of the new system of product attesting. For instance, JUDIMK prepared the first order about obligatory attesting of cement, certain concrete prefabrications, aggregates and some waterproofing materials. The work of the Society on the production of technical regulations lasts until nowadays. It should be emphasized that JUDIMK jointly with SDGKJ and the Faculty of Civil Engineering of Belgrade in 1993 initiated the Federal Institute for standardization to commence translation and introduce the professional public with Eurocode.

In the past decades, starting from 1966, fourteen Congresses and Symposia on the application of contemporary achievements in our construction engineering in field of material and structures were held, those being:

- *XII Congress and Symposium SJL in Sarajevo, November 1966. Milutin Maksimović, grad.eng. who was the first chairman of SJL in 1952, was elected the chairman of Managing board.*
- *XIII Congress and Symposium SJL on Bled, November 1969. Viktor Turnšek, grad eng, which was the chairman of SJL in the period 1957-1958 was elected the chairman of Managing board.*
- *XIV Congress and Symposium SJL, Haludovo – Malinska, October 1972. Stanko Manestar, grad.eng. was elected the chairman of Managing board.*
- *XV Congress and Symposium SJL, Ohrid, October 1975. prof. Hololčev Krum, grad.eng. was elected the chairman of Managing board.*

- *XVI Congress and Symposium SJL, Vrnjačka Banja, November 1978. At this Congress, the Association of Yugoslav laboratories changed its name into Yugoslav Society for Testing and Research of Materials and Structures – JUDIMK. Prof. Dobrosav Jevtić, grad. Eng. was elected the chairman of Managing board, whose function was later taken over by prof. Aleksandar Flašar.*
- *XVII Congress and Symposium JUDIMK, Sarajevo – Ilidža, October 1982. prof. Seid Ferušić, grad. Eng. Was elected the chairman of JUDIMK, whose function was later taken over by ass. Prof Ph.D. Jože Vižintin.*
- *XVIII Congress and Symposium JUDIMK, Portorož, October 1986. Ph.D. Andrej Zajc was elected the chairman of JUDIMK .*
- *XIX Congress and Symposium JUDIMK, Novi Sad, September 1990. Prof. Aleksandar Flašar was chosen the chairman of the Society.*
- *XX Congress and Symposium JUDIMK, Cetinje, June 1996. Ph.D Đorđe Uzelac was elected the chairman of the Society.*
- *XXI Congress and Symposium JUDIMK, Beograd, September 1999. Prof. Ph.D Mihailo Muravljov was elected the chairman of the Society.*
- *XXII Congress and Symposium JUDIMK, Niška Banja, October 2002. Prof. Ph.D. Mihailo Muravljovu had his office extended.*
- *XXIII Congress and Symposium DIMK of Serbia and Montenegro, Novi Sad, October 2005. Prof. Ph.D. Mihailo Muravljovu had his office extended.*
- *XXIV Congress and Symposium DIMK Serbia, Divčibare, October 2008. Prof. Ph. D. Vlastimir Radonjanin was elected the chairman of the Society.*
- *XXV Congress and Symposium DIMK Srbije, Tara, October 2011. Prof. Ph.D. Zoran Grdić was elected the chairman of the Society.*

This foreword was written with the extensive usage of the data from the monograph „Yugoslav Society for Testing and Research of Materials and Structures JUDIMK 1952/2002“ which was prepared in Belgrade in 2002 professor Lazar Jovanović, and using „Bibliography of symposia on testing and research of materials and structures 1966 – 2010, as well as " Conventions, symposia, conferences and round tables 2000 – 2010" prepared by Vladimir Denić, grad. Eng. of technology in Belgrade, 2011.

SOCIETY FOR MATERIALS AND STRUCTURES TESTING OF SERBIA
DRUŠTVO ZA ISPITIVANJE I ISTRAŽIVANJE MATERIJALA I KONSTRUKCIJA SRBIJE

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Dubravka Bjegović[1]

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RESEARCH OF REINFORCED CONCRETE TOWARDS TO SUSTAINABLE DEVELOPMENT

Summary: *Sustainable development is one of the major issues of modern society, highly endangered by industrialization and technological development. Use of large amounts of non-renewable resources as well as significant air emissions during cement production placed concrete industry very high on the scale of world's pollutants. Throughout the world some progress has been made in utilizing these innovations but largely these remain outside routine practice.*

In this paper an overview is given of possible actions that can be employed in concrete technology in order to upgrade concrete industry and create one that is environmentally more friendly and sustainable. Some of the actions include using of blended cements and new types of binders, recycled aggregate from construction and demolition waste, industrial waste or from end-of-life tyres, prolongation of service life by prescribing, controlling and assuring durability in design, construction and exploitation phase of concrete structure. The benefits of given actions are highlighted through presentation of experimental results obtained on the existing scientific projects in the area of sustainable development.

Key words: *CO_2 emission, recycling, waste tyres, corrosion, fire resistance, durability*

ISTRAŽIVANJA ARMIRANOG BETONA U SKLADU S ODRŽIVIM RAZVOJEM

Rezime: *Zaštita okoliša i ušteda energije postaju krucijalni svjetski problemi u svim poljima tehnologije. Tehnologija betona je danas najveći potrošač prirodnih resursa i jedan od najvećih proizvođača otpada. Upravo zato, javlja se snažna potreba za poduzimanjem koraka koji bi tehnologiju betona pretvorili u takovu koja je održiva.*

Temelji održivosti leže u tri osnovne postavke, koje su detaljno obrađene u ovom radu: smanjenje emisije CO_2 zamjenom klinkera mineralnim dodatcima i alternativnim vezivima, očuvanje prirodnih resursa zamjenom prirodnog agregata recikliranim betonom ili proizvodima reciklaže guma, te projektiranje, gradnja i održavanje trajnijih armiranobetonskih konstrukcija. U radu se daje prikaz istraživanja usmjerenih na zadovoljenje kriterija održivosti u području armiranog betona kao materijala trajnijih armiranobetonskih konstrukcija, koja su provedena ili se provode na Zavodu za materijale Građevinskog fakulteta Sveučilišta u Zagrebu u sklopu znanstvenih projekata.

Ključne reči: *emisija CO_2, recikliranje, guma, korozija, vatrootpornost, trajnost*

[1] Građevinski fakultet, Zavod za materijale, Kačićeva 26, Zagreb; Institut IGH,d.d. Janka Rakuše 1, Zagreb

1. INTRODUCTION

Environmental protection and energy saving are crucial problems in all fields of human activities, especially in production and industry. Sustainable development has become one of the most evident solutions in order to prolong the existence of clean and healthy environment. The 1992 Earth Summit in Rio de Janeiro defined sustainable development as economic activity that is in harmony with the earth's ecosystem [1]. The best way to ensure sustainable development would be to reconcile human needs with the capacity of the planet to cope with the consequences of human activities, or in other words to take from the earth as little as possible natural resources and return to the earth as little as possible waste.

Concrete industry today, is largest consumer of natural resources and one of the largest waste producers. That is why there is a need of upgrading concrete industry and creating one that is environmentally friendly and sustainable. Yearly around 750 million m3 of concrete is being produced in Europe, which is equivalent to consumption of around 4 tonnes of concrete per EU citizen. [2]. Even though, compared to other building materials, concrete is considered to be environmentally friendly, its constituents and production technology are not. Postulates of sustainability in construction industry are most definitely: lowering CO2 emission, with the use of by-products of other industry as substitutions of Portland cement; conservation of natural resources, with the use of recycled materials instead of crushed stones for aggregate and designing, constructing and maintaining more durable concrete structures.

The benefits of given actions are highlighted through presentation of experimental results obtained on the existing scientific projects in the area of sustainable development which are being performed at Department of Materials on Faculty of Civil Engineering University of Zagreb in Zagreb, Croatia.

2. LOWERING CO_2 EMISSION

Nowadays, the world consumption of cement is more than 2.5 billion tonnes annually [1] and it is expected that this number will rise with the industrialization of developing countries. In Croatia, there is still a growth trend of CO2 emission. In year 2006, direct CO2 emission from cement industry was around 2.5 million tonnes, which is compared to year 1990 growth of 51 %. In year 2006, specific CO2 emission was estimated to be 850 kg CO2/t of clinker or 700 kg CO2/t of cement. That means that in Croatia cement industry causes around 8-9 % of total CO2 emission [3, 4]. Clinker substitution is a promising solution as it is a low-cost option that has not yet been used to the greatest possible extent, and, as such, still has great potential [4]. The most common supplementary materials are fly ash, granulated blast furnace slag, silica fume and limestone. However, in order to create market acceptance, the properties of these binding materials have to be comparable with Portland cement [5].

Experimental research performed on concrete mixtures with quaternary-blended cements shows that the substitution of cement with by-products of other industries can have a positive influence on concrete durability properties (penetrability of fluids and ions, early age cracking) [6, 7]. Even though a single type of mineral admixture can have different influence on a specific concrete property, synergic influence of mix of mineral can be achieved. Therefore, the mix of binders in quaternary blended cement can be designed in a way that the benefits of each added mineral admixture are utilised [8].

3. CONSERVATION OF NATURAL RESORCES

Waste management is nowadays one of the priorities of every community and it has become evident that good waste management can enhance the quality of life. Main principle of waste management is in lowering production of new, finding ways to recycle and reuse existing and safe and ecologically acceptable depositing of unused waste [9, 10]. One of the possible utilisations of recycled waste materials, such as C&D waste, waste slags and end-of-life tyres, is to use them as a substitution of natural aggregate obtained from natural resources.

3.1. Utilisation of C&D Waste

Most of the products obtained during demolition of reinforced concrete structures can be reused in concrete industry after separation and recycling. Reuse of C&D waste as a part of aggregate is one of the best way of conservation of natural resources and reduction of new quantities of construction waste.

Experimental research performed on concrete mixtures prepared with recycled concrete aggregate showed that the maximum amount of substitution of aggregate with recycled aggregate is 30% [11]. Adding higher amount of recycled aggregate caused significant changes in concrete properties, compared to the properties of reference mixture. But the influence of recycled aggregate on the properties of concrete is not straight forward, and depends on the vast number of parameters, mainly on the properties of concrete from which the recycled aggregate has been prepared [11, 12]. Furthermore, properties of concrete with recycled aggregate can be significantly enhanced with the application of chemical and mineral admixtures and with the protocol of concrete mixing. Therefore, before utilisation of aggregate obtained by C&D waste recycling, performing additional tests with locally available materials is highly recommended.

3.2. Utilisation of steel slag

Compared with granulated blast furnace slag (GBFS), which are covered by regulations and long experience in the cement production, application of electric furnace steel slag (EAF) is not so widespread. According to surveys conducted in the countries of the European Union, there is about 11% of the produced steel slag that has been deposited [13], although the percentages may vary from country to country. At the Republic of Croatia there are two landfills of electric arc furnace steel slag near the steel factory in towns of Sisak and Split. High price of slag depositing imposes the need for finding new fields of its utilisation. Until now, these slags were used in road construction (as a stabilisation layer) and in agriculture (fine fractions are used for soil improvement).

Recent studies [14] indicate the possibility of the utilisation of steel slag as an aggregate in concrete production. The study included steel slags from Croatian steel plants Sisak and Split, which was compared to the conventional dolomite aggregate. The results obtained showed that coarse fractions of both studied slag meet the requirements set in the Croatian Technical requirements for reinforced concrete structures [15, 16] and thus can be considered as a suitable substitute for conventional aggregate out of natural sources. Also, due to the fact that the steel slag was produced at high temperatures (around 1650°C), the study included and compared the behaviour of concrete with slag and dolomite aggregate at high fire temperatures (up to 800°C).

The results showed poorer residual mechanical properties of concrete with steel slag at temperatures higher than 600° C due to mineralogical changes which was accompanied by its volume expansion. Studies have also shown that pre-heating of the slag to a temperature of 1000 ° C make the steel slag stable and the use of such slag in concrete causes even better mechanical properties at high temperatures compared with concrete made with dolomite aggregate (Fig. 1) [17,18].

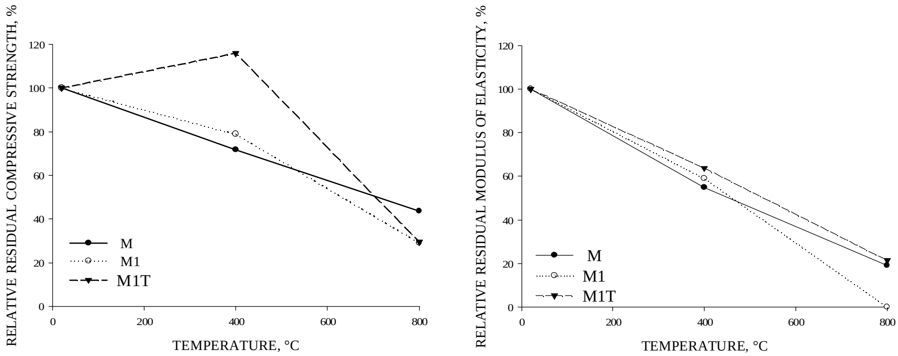


Figure 1. Relative compressive strengths and modulus of elasticity of concrete mixtures M (dolomite based mixture), M1(steel slag based mixture) and M1T (pre-heated steel-slag based mixture) vs. Temperature

3.3. Utilisation of end-of-life tyre

Every year about 3.4 million tonnes of waste tyres are generated in Europe [19]. In the EU15, only 5 % of waste tyres are uncontrollably disposed in landfills. In the 12 new EU member states and Western Balkan, averagely 29 % of waste tyres are disposed in landfills, annually. With the introduction of EU Directive [20] in those countries, which bans landfilling of whole (July 2003) and shredded (July 2006) tyres, it is clear that there is need to increase recycling capacities and develop markets for utilising recycled tyres. The Republic of Croatia harmonized its regulations with the EU Directives relating to the assessment and management of waste tyres. In other words, 70 % of all waste tyres should be processed through material recycling and other 30% can be reused as energy source.

By products obtained during shredding process are the only recycled products available on Croatian market and they include rubber, steel and textile fibres. The extensive research performed on University of Zagreb Faculty of Civil Engineering implied positive effects when those are incorporated in concrete, either as replacement for raw materials or as a certain chemical admixtures [22-33].

The main features of recycled rubber are: low density, low modulus of elasticity, insulating properties and ability to absorb energy resulting from impact. Due to those features recycled rubber is a very attractive material for the use in construction industry. Recycled rubber is typically used as a substitute for aggregates, in large volumes for a lightweight concrete [25-28] or in small shares when it serves as energy absorber in high strength concretes [30-33]. Steel and textile fibers obtained by shredding are still less applied compared to the recycled rubber. Recycled steel fibers can find their application in the construction industry as an economically feasible substitute for industrial steel fibers; especially for construction of concrete pavements, industrial floors or as shotcrete. Although the use of recycled textile fibers is possible in construction industry, this material could find larger application in textile industry.

In cooperation with industrial partners, researchers from the University Of Zagreb Faculty Of Civil Engineering have developed two different eco materials containing by – products from mechanical recycling of waste tyres for production of innovative products; lightweight concrete for production of Rubberized concrete noise barriers – RUCONBAR and rubberized hybrid steel fibre reinforced concrete for production of Concrete track systems – ECOTRACK. Both projects are focused on development of sustainable concrete technologies, consumption of waste tyres and know-how transfer from scientific community to industry.

3.3.1. RUCONBAR – Rubberized Concrete Noise Barriers

The proposed solution is to develop a concept of utilisation recycled tyres as new material for reduction of urban noise pollution, called RUCONBAR [25-28]. The concept provides benefits in three directions which are: (1) noise protection of urban areas by utilisation of recycled materials, (2) preventing landscape degradation from clay excavation by introducing new material and (3) environmental protection by preventing disposal of recyclable materials on landfills. In its nutshell, it is a concrete based solution composed of absorbing and bearing layer (Figure 2). By incorporating 40 % rubber granules recycled from waste tyres recovered from end-of-life vehicles, absorbing layer is innovative solution in production of noise barriers.

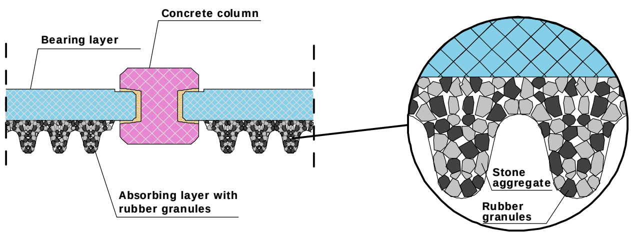


Figure 2. RUCONBAR cross section

RUCONBAR reaches two major environmental problems, noise pollution and waste tyres management through ecologically and economically more efficient way – using waste to develop new product while the product itself is used for noise pollution protection. According to the absorption measurement RUCONBAR noise protection barrier has been listed under A2 class of sound absorption based on the sound absorption value DLα = 6dB. Some of the competitive products can achieve higher classes of sound absorption, which greatly depends on the cross section of the absorption surface. Conducted testing indicate satisfying absorption properties and the possibility of their improvement through further development with the goal of reaching class A3 of sound absorption [28].

3.3.2. Concrete track systems – ECOTRACK

ECOTRACK is an eco-innovative product of a modern high speed railway structure (Figure 3). Solution is made of two-part concrete sleepers built in the concrete slab, together making a ballastless concrete track system. Although, similar solutions are already present on the market ECOTRACK incorporates by-products from mechanical recycling of waste tyres as a replacement for usual natural raw materials [30-33].

Figure 3. A conceptual prototype of the ECOTRACK railway track

The usage of rubberized hybrid fibre reinforced concrete elements assures an adequate resistance ability of the structure under various strain conditions. Furthermore, the appearance of first cracks on concrete surface is prolonged and thus a higher durability of such construction elements is achieved. In accordance with the starting expectations, the initial testing of the ECOTRACK confirmed the possibility of the application of ecologically acceptable resources (recycling products) for the production of high performance concrete for special application. Comparing the achieved results with the criteria set up in relevant standards for concrete railway tracks, it has been confirmed that concrete with specific ratio of recycled products satisfies the mentioned conditions [31-33].

4. DURABILITY REQUIREMENTS

Designing, constructing and maintaining more durable structures is one of the key postulates of sustainable development. Research have shown that if concrete structures would be build with service life of 250 rather than 50 years, usability of natural resources would increase 5 times [34].

Figure 4. Holistic approach towards durability in entire life cycle of structure

The study of durability of concrete started several decades ago, and is still a subject of numerous scientific and technical committees [35-37] and national and international conferences. The main conclusion of all intentions in this field is that a holistic approach towards durability design is necessary, consisting of tailoring appropriate material and its properties in the design phase, controlling and assuring that properties are met in the construction phase, and maintaining and assessing for pro-active instead of reactive repair during entire service life [38].

4.1. Design phase: the importance of service life modelling and material choice

Nowadays environmental conditions are considered as a loading structure has to bear during entire service life. This loading is then an input parameter in service life modelling and is used for tailoring the properties of construction materials. In marine environment, where chloride-induced corrosion is the main degradation mechanism, the amount of chlorides in the concrete are considered to be a loading which is used in the modelling. This parameter is highly depended upon distance of structure from the sea, influences of temperature and wind, orientation of elements [39]. During research in this field, expressions correlating chloride loading and wind effects are proposed, Figure 5. With the increasing speed of wind, the amount of chloride in the air and accumulated on the surface of concrete also increases.

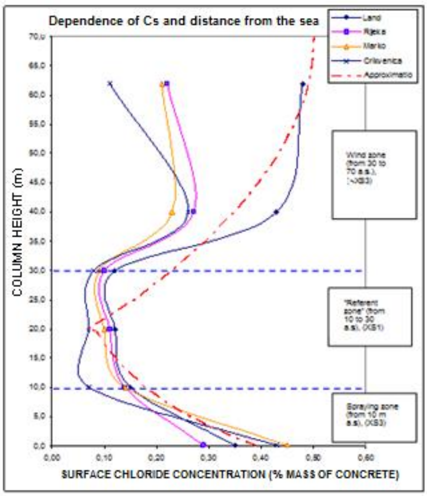


Figure 5. Proposed wind zones for determination of chloride loads on concrete construction [39]

As a part of the research mathematical dependency between durability properties of concrete and concrete mix design were established and introduced into the model CHLODIF++ for modelling and predicting service life of reinforced concrete structure exposed to marine environment [39-41].

Beside modelling and tailoring concrete material, the choice of reinforcing material also becomes prevailing in very aggressive environments. Recent results indicate that some low alloy steels under certain conditions behave significantly better than ordinarily used black steel reinforcement. Since the price of low alloy steel reinforcement is comparable to the price of normal black steel reinforcement, the idea of using low alloy steel as reinforcement is economically justified [42]. Research was performed to establish initiation and propagation diagrams for different types of reinforcing steel. Figure 6 shows expected time to certain level of corrosion propagation [45] for 10 different steel types.

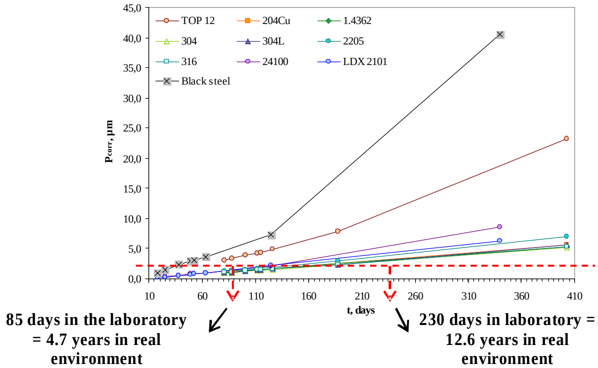
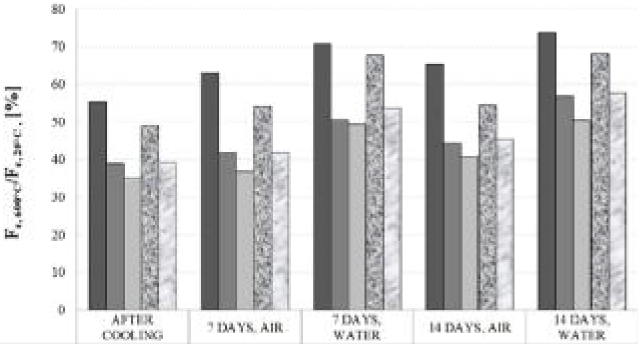


Figure 6. Depth of corrosion penetration and expected time to reaching 5 µm of depth [43, 45]

Results show that different types of steel exhibit different levels of corrosion resistance. That would allow the designer easer and tailored choice of corrosion resistant reinforcing steel during design of more durable structures exposed to aggressive environment.

In recent times more attention is paid to the risk of fire when designing structures. This is a consequence of the fact that fire is levelled at the European and Croatian legal and technical regulations with all other usual actions to structures. Although it is generally said that the concrete has a good behaviour in the fire, depending on its type, high temperatures can adversely affect the physical and mechanical properties of concrete (compressive strength, stiffness, modulus of elasticity, etc.), especially in high-strength concrete and self-compacting concrete [46, 47, 48]. The goal of the research, which began at the Department of Materials, is to conduct a comprehensive study of the impact of high fire temperature on the mechanical properties (compressive strength and modulus of elasticity) of self-compacting concrete with the different mineral additives connecting changes in microstructure and mechanical properties. Past research has shown that fire-damaged concrete can recover strength to some extent if it is properly recurred in water or in a moist environment. Preliminary results indicate that the subsequent re-curing in water up to 14 days can recover up to 20% of the residual strength of self-compacting concrete with different mineral additives after exposure to 600°C (Figure 7).



	AFTER COOLING	7 DAYS, AIR	7 DAYS, WATER	14 DAYS, AIR	14 DAYS, WATER
■ L	5,5	6,3	7,1	6,5	7,4
■ LFA30	3,9	4,2	5,0	4,4	5,7
□ LFA30-PPF	3,5	3,7	4,9	4,1	5,0
□ LSF5	4,9	5,4	6,8	5,4	6,8
□ LSF5-PPF	3,9	4,2	5,4	4,6	5,8

Figure 7. Residual strength behaviour of tested concrete mixtures [48]

4.2. Construction phase: the importance of quality control and assurance

Durability indicators of concrete are fundamental in evaluating and predicting the durability of the material and service life of structure. They are prescribed during design of concrete structures, tested during prequalification testing, used in service life models and tested during construction as a part of quality control on site [49]. They must be quantifiable by laboratory tests in a reproducible manner and with clearly defined test procedures. Nowadays, many testing procedures for performing durability properties tests on concrete are standardized or being already used for longer period, and proved to have satisfactory precision. Some of them are already recommended or prescribed in current technical requirements for concrete structures [15] or in national annexes. In the case of Croatia, waterpermeability, freezing/thawing resistance and abrasion are prescribed in the national annex [50], but the limiting values are still not correlated to environmental exposure classes. There are some initiatives to add air permeability, chloride diffusion and resistivity to the list of durability indicators, but the concept is still done on the basis of willingness of the contractors and investors [51, 52].

4.3. Exploitation phase: the importance of monitoring and residual service life assessment

Long-term monitoring of material behaviour is of great importance to enable pro-active instead of reactive maintenance and repair strategy [53, 54]. Periodic assessment of structures can enable realistic calculation of residual carrying capacity and service life, through for example calculation of time-to-cracking of concrete due to corrosion. Furthermore, models correlating the level of corrosion activity with reinforced concrete carrying capacity are being developed, which would allow even more precise calculation and management of structures [55 - 57].

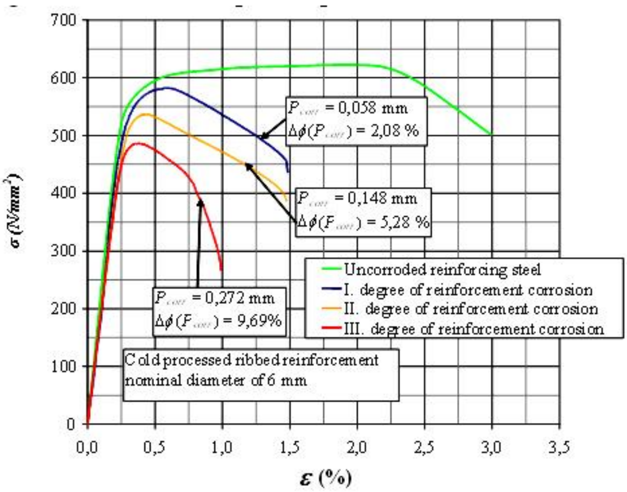


Figure 8. Stress-strain diagrams (s-e) of reinforcement corroded to different penetration depths [57]

The extensive project was done at Faculty of Civil Engineering, University of Zagreb and at Civil Engineering Institute of Croatia with the aim to propose procedure for calculating remaining load carrying capacity of reinforced concrete elements damaged by chloride induced corrosion [57-59]. The experimental research comprised 16 reinforced concrete beams and 16 reinforced concrete slabs that were prepared and exposed to combined influence of mechanical and environmental loading. After certain, predefined level of corrosion is reached, mechanical properties of reinforcement were performed, and stress-strain diagrams of corrosion reinforcing steel are proposed, Figure 8.

Within this research an experimental correlation between corrosion penetration depth, chloride amount and carrying capacity has been obtained. Procedures, such as one proposed in aforementioned research, would enable calculation of loss of serviceability and safety of structures affected by chloride-induced corrosion.

5. CONCLUDING REMARKS

Sustainable development is development that meets the needs of the present without compromising the ability of future generations to meet their own needs. More specifically, it represents a balance between the use of non-renewable resources, recycling and waste disposal. When speaking of reinforced concrete, material that meets the high technical and relatively low technological requirements, it should be noted that in the concrete production, there are no negative effects on the environment. However, the production of its main components, particularly cement, causes significant environmental pollution. It follows that reinforced concrete sustainability lays in three aspects: CO2 emissions reduction by replacing part of the cement with other by-products (slag, fly ash, etc.); waste recovery for the purpose of replacing part of non-renewable with renewable resources; and design, construction and maintaining of the structure in order to meet mechanical and durability requirements. Even though some advances have been made in quick implementation of new concrete technology, significant barriers to innovation and implementation remain. Continued coordination of on-going international research and educational programs is needed. In this paper, based on the research carried out through undergraduate, graduate and doctoral works at the Department of Materials, Faculty of Civil Engineering, University of Zagreb, it is shown that concrete could be considered as sustainable material. Once the research has been completed, a number of possible implementation mechanisms need to be considered in order to select the right approach for successful transfer of the technology to the practitioner. Multiple strategies including information dissemination, training workshops, field demonstration pilot projects, hands-on training, equipment loan programs, technical support and continuing educational courses should be considered for research product implementation. But this action is a long-term process and may require several years of effort.

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SOCIETY FOR MATERIALS AND STRUCTURES TESTING OF SERBIA
DRUŠTVO ZA ISPITIVANJE I ISTRAŽIVANJE MATERIJALA I KONSTRUKCIJA SRBIJE

International Conference **"Marking 60 Years of Operation of DIMK**
Belgrade, October 19, 2012.

Radomir Folić [1]

UDK:624/627(497.11)
69(497.11)

REVIEW OF RESEARCH IN THE FUNCTION OF STRUCTURAL ENGINEERING DEVELOPMENT IN SERBIA

Summary: *Research at faculties in Serbia after WWII, even though they were fruit of individual efforts, resulted in significant theoretical contributions. Organized, mostly applied research was conducted within institutes with participation of teachers and researchers of the faculties. Since 1952, the Association of Yugoslav Laboratories (SJL) united al those researches, thus providing a considerable contribution to the structural engineering in Serbia. This contribution in theory and practice was a basis for remarkable results that brought world renown to our structural engineering. Theoretical contributions contained in the doctoral dissertations in the field of Theory of structures and Geotechnics are of importance, so they were listed in the paper. Majority of these studies became a part of research project as late as in 1976, and they have been briefly presented in the paper. The synthesis of results of fundamental and technologic development researches was published in numerous monographs, articles in the papers and proceedings of scientific and professional meetings, and some of them have been commented in the paper. The necessity to introduce technical regulations which has already been adopted in the EU countries EN 1990 to 1999, that is the Eurocode, was emphasized. There is a brief discussion of several research directions which are topical nationwide and worldwide.*

Key words: *Research, PhD theses, scientific project, code, Euro-code, Association of laboratories, Society for materials and structures testing (SMST), Faculty, Institute*

PREGLED ISTRAŽIVANJA U FUNKCIJI RAZVOJA KONSTRUKTERSTVA U SRBIJI

Rezime: *Istraživanja na fakultetima u Srbiji posle Drugog svetskog rata, iako su bila plod individualnih napora, rezultirali su značajnim teorijskim doprinosima. Organizovana, uglavnom primenjena istraživanja, obavljana su u okviru instituta u kojima su učestvovali i nastavnici i saradnici sa fakulteta. Od 1952. g. Savez jugoslovenskih laboratorija (SJL) objedinjavao je ova istraživanja, čime je dao značajan doprinos razvoju konstrukterstva u Srbiji. Taj doprinos na polju teorije i prakse je bio podloga za zapažene domete koji su u svetu proslavili naše konstrukterstvo. Od značaja su teorijski doprinosi koji su sadržale doktorske disertacije iz oblasti teorije konstrukcija i iz geotehnike, pa je i njihov popis dat u radu. Većina takvih radova je tek 1976. g. postala deo istraživačkih projekata, koji su ukratko prikazani u radu. Sinteza rezultata projekata iz osnovnih, a i tehnološko-razvojnih istraživanja publikovana je u mnoštvu monografija, članaka u časopisima i zbornicima radova sa naučno-stručnih skupova, a neki od njih komentarisani su u radu. Naglašena je potreba uvođenja tehničke regulative, koja je usvojena u zemljama Evropske unije EN 1990 do 1999, tj. Evrokodovi. Sažeto je ukazano na nekoliko pravaca za istraživanja koja su aktuelna u nas i u svetu.*

Ključne reči: *Istraživanja, doktorati, naučni projekti, regulativa, Evrokodovi, Savez laboratorija, DIMK, Fakulteti, Instituti*

[1] Professor Emeritus, University of Novi Sad, Faculty of Technical Sciences, email: folic@uns.ac.rs

1. INTRODUCTION

Yugoslav Society for Testing of Materials and Structures (JUDIMK), now Society for Testing of Materials and Structures DIMK of Serbia was preceded by the Association of laboratories for testing materials and structures founded on May 10^{th} 1952. Then the scope of work of the Association was discussed, and it was adopted that its activities should follow the program of RILEM, and that jointly with the Association for Standardization it should contribute to formulation of the standards in the domain of methods of material and structural testing (MiK). Also agreed was the formation of section by the republics. Formation of the Bulletin, as a precursor of the present day journal Building materials and structures (GMiK), provided better information of the working results in the field of material and structures. In the first phase, apart from the information, also published were the short abstracts of the papers delivered on the annual assembly meetings. Although in creation of the structures, the most important place belongs to the designers, their relation with the researchers and manufacturers (prefabricated elements building) and contractors is significant. Materials and structures are often treated jointly, as the research and improvement of materials and their properties is of special interest for the development of structural engineering.

The research described in this paper, was realized in different periods and in different conditions, and span the period from WWII till the present days. However, in the monographs [44] several periods were included. The first refers to the work of the Great school 1903-1905, the second to the operation of the Department at the Technical Faculty of the University of Belgrade with the status of the civil engineering teaching in the period 1918-1941. The Technical faculty, after the WWII worked on consolidation of the studies until 1948 when the Civil Engineering faculty was formed (GF), whose teaching reforms until 1978 were described in the monograph. The faculties in other places, in Niš, Priština, N. Sad and Subotica were formed with a significant effort of teachers and researchers of the GF Belgrade. Even though several institutes were formed after WWII, faculties were also engaged in scientific-research work and cooperation with the economy, mostly through their institutes. The research at the faculties in Serbia after WWII was the result of enthusiasm and individual efforts with significant theoretical contribution. Organized, mostly applied research was conducted within institutes with participation of teachers and researchers of the faculties.

Association of Yugoslav laboratories (SJL) since its foundation in 1952 unites the research of those institutes and direct it to the then topical issues in civil engineering, with strong synergy of theory and practice. Such operation, even though it was not sufficiently financially supported, considerably contributed the development of structural engineering in Serbia in realization of a number of structures which represent remarkable achievements at the global level [1] and [8]. The theoretical contributions of doctoral dissertations and master thesis in the field of theory of structures and geotechnics were a basis and encouragement to the development of structural engineering in our country. Only since 1976 the work on the doctoral dissertations was connected to the research projects, which were financed from the funds of the Republic and Autonomous regions, or from other sources (economy, chamber). Here is presented the list of doctoral dissertations with the titles from which one may view the themes treated and their importance for structural engineering. Only some of the research projects in the field of fundamental and technological development research were commented upon and also the important publications resulting from them are listed. Since ten years ago, there has been work in

Serbia on studying and translation of technical regulations adopted in the EU countries with the designations EN 1990 do 1999, which are often called Eurocode. The need for further systematic work on putting them to practice in our country is emphasized.

In the recent years, an increasing number of researchers is focused on publishing of the research results in international journals and proceedings from international scientific meetings. The synthesis of results of earlier research projects is published, mostly in our country in a number of monographs, articles in journals and proceedings from scientific and professional meetings which is only partially commented in this paper. In brief, several directions of future researches which are topical both in our country and worldwide were mentioned. In this sense, a wider list of references indicating the topicality of the themes present in research in Serbia.

2. DEFENDED DOCTORAL DISSERTATIONS AT FACULTIES OF CIVIL ENGINEERING IN SERBIA

2.1. Doctoral dissertations defended at the Faculty of Civil Engineering of Belgrade

Dragoš Radenković: Bending of curved members in plane (1953); *Milan Đurić*: Theory of long prismatic polyhedral shell (1953); *Vladimir Bogunović*: On bending of rectangular plate with one degree of freedom (1953); *Nikola Hajdin*: One procedure for numerical solving of ultimate (boundary) tasks and its application on some problems of Theory of elasticity (1956); *Vlatko Brčić*: Toward the solution of flat problem of the Elasticity Theory (1956); *Aleksandar Vesić*: Fundamental problems of theory and calculation of a system of piles (1956); *Dušan Krsmanović*: Impact of stiffness, continuity and discontinuity on calculation of structures supported on the soil (1957); *Milorad Ivković*: Behavior of concrete in the area of limit equilibrium (1962); *Živojin Hiba*: Towards a theory of steel anchored suspended bridges with the beam for stiffening the incomplete flexibility (1963); *Branko Zarić*: Buckling of steel members in the plastic area (1963); *Vlade Vračarić*: Cooperation of deck and main beams of steel truss railway bridges (1965); *Dimitrije Dimitrijević*: Behavior of RC and composite members with participation of concrete creep at dynamic load (1973); *Jakov Lazić*: Application of linear visco-elasticity in the structural theory (1973); *Miodrag Sekulović*: Thin-walled member curved in space (1973); Beograd); *Mihajlo Muravljov*: Behavior of thin-walled open-profile members of pre-stressed concrete at limited torsion with the concrete flow effects (1975); *Ratko Stojanović*: Resistance of paraseismical structures on the action of explosive shock (1975); *Vera Lazić*: Dynamic stability of curved members of visco-elastic material (1975); *Milutin Marjanov*: Solution of composite problems of thermal-elasticity in a confined environment (1977); *Milan Gojković*: Stone bridges from XIV to XVIII century in Yugoslavian boundaries (1977); *Milan Lazić*: Parameters of general optimality when using light-aggregate concrete in panel construction of housing buildings (1977); *Boško Petrović*: Behavior of skeletal buildings of pre-stressed concrete under the action of seismic forces (1977); *Radoje Vukotić*: Limit states of members of reinforced and pre-stressed concrete loaded by torsion and bending (1977); *Vojislav Mihailović*: Generalized procedure of design of composite and pre-stressed structures (1978); *Dragoljub Nikolić*: Some problems of non-linear analysis of curved members (1978); *Predrag Jovanović*: Towards a structural matrix analysis (1978); *Mirko Aćić*: Towards solving the issue of limit states of reinforced concrete support walls (1978); *Stevan*

Stevanović: Towards a solution of in-plane problem of the theory of eleasticity for some semi-infinite areas bounded by polygonal contours (1978); *Milivoje Stanković*: Towards a design of thin-walled members with deformable cross-section (1978); *Živorad Radosavljević*: Analysis of rationality of ultimate limit state of concrete (1978); *Života Perišić*: A contribution to determination of creep and shrinking effects in cracked reinforced concrete cross sections with cracks (1979); *Aleksandar Pakvor*: Towards a study of thermal stresses and strain of concrete blocks (1979); *Šerif Dunica*: Towards a plastic analysis of spatial linear systems, composed of truss, full and thin-walled elements (1979); *Živojin Praščević*: Non-linear theory of reinforced concrete member (1979); *Miloš Manojlović*: Towards a theory of dynamic behavior of soil (1979); *Branislav Kolunžija*: Towards a second order theory of spatial linear systems composed of truss, full and thin-walled elements (1979); *Dušan Milovanović*: Latest findings in the theory of arch dams (1980); *Čedomir Vujičić*: Towards optimizing of design of reinforced concrete walls built directly on soil (1980); *Savo Vukelić*: Analysis of mechanical states of hardened concrete – thermo-dynamical approach (1981); *Milorad Ristić*: Towards a analysis of elastic, pre-stressed networks (1981); *Slavko Zdravković*: Mathematical modeling of bridge road decks structures to dynamic and seismic action (1981); *Branislav Ćorić*: Theoretical and experimental analysis of local and lateral buckling of steel I beam of a deformable cross section (1982); *Dragoljub Grbić*: Oscillations of elastic members at ultimate displacements (1982); *Radomir Folić*: Towards a study of T beams – analysis of active slab width and ultimate states of reinforced concrete pre-stressed concrete elements (1983); *Srđan Venečanin*: Impact of thermal incompatibility of concrete components on its strength (1983); *Jovanka Zurovac*: Towards a analysis of elastoplastic behavior of reinforced concrete beam (1983); *Petar Petrović*: Numerical solution of Lame's equations of the elasticity theory using the integral equation method (1985); *Dimitrije Rajić*: Towards a non-linear theory of thin elastic shells and its application (1985); *Kisin Srđan (then from Sarajevo)*: Towards a theoretical experimental analysis of lateral buckling of mono-symmetrical beams of deformable cross section (1985); *Aleksandar Babović*: Ultimate limit state of eccentrically compressed RC columns in steel tubes (1985); *Dejan Bajić*: Towards non-linear analysis of RC linear elements (1985); *Stanko Brčić*: Dynamic behavior of structures in a fluid environment (1987); *Cvijetin Kanjerić*: Analysis of flat frame beams stressed beyond the proportionality limit (1987); *Živorad Bojović*: Elastic damped systems exposed to the load which depends on their configurations under small strain and large displacements (1988); *Miloš Lazović*: Towards a non-linear analysis of axially loaded pile (1988); *Đorđe Vuksanović*: Non-linear analysis of reinforced concrete slabs by the finite element method (1988); *Rastislav Đorđević*: Towards a analysis of long-stroke tooling machines foundation behavior in service conditions (1989); *Gligor Radenković*: General non-linear analysis of shells based on triangular and general quadrangular finite element (1989); *Branislav Pujević*: Towards a non-linear analysis of thin-walled reinforced concrete structures (1989); *Dragan Buđevac*: Towards a design and structural formation of cold-rolled profiles of an open cross-section (1990); *Mihajlo Đurđević*: Behavior of concrete composite prefabricate elements and connections in the area of failure (1990); *Aleksandar Prokić*: Thin-walled beams of open-closed cross-section (1990); *Ljubomir Savić*: Towards a mathematical research of some statistical problems of non-linear behavior of structures (1990); *Gajin Slobodan*: Non-linear dynamic analysis of foundations of electro-mechanical transmissions and their vibro-insulation (1992); *Biljana Deretić-Stojanović*: Design of composite structures using strain method (1992);

Mira Petronijević: Analysis of dynamic interaction of soil and structure applying finite element method (1993); *Rastislav Mandić*: Modeling of interaction of reinforcement and concrete in reinforced concrete structures applying the finite element method (1994); *Milan V. Matović*: Towards an analysis of stress and displacement of thin-walled composite beams (1996); ***Želimir J. Kovačević***: Concrete bridges maintenance managemwent systems (1997); *Dragan Č. Lukić*: Towards a method of defining the stress state around the cavity in the form of rotating ellipsoid applying elliptical coordinates (1998); *Miroslav T. Bešević*: Towards a analysis of centrically loaded steel members of complex cross-sections of cold-rolled profiles (1999); *Petar B. Santrač*: Analysis of strip foundation behavior on sand (1999); *Zoran M. Mišković*: Application of stress fields based on the theory of plasticity for determination of ultimate limit state of reinforced concrete bearing walls (2000); *Ratko M. Salatić*: Analysis and control of behavior of steel frames at earthquake action (2001); *Snežana B. Marinković*: Ultimate limit state at punching shear of prefabricated pre-stressed slabs in the area of fringing columns (2002); *Dušan I. Kovačević*: Numerical modeling of behavior of reinforced concrete frames loaded by seismic forces (2002); *Milorad T. Komnenović*: Analysis of stress states of curved beams of laminated glued timber loaded by bending (2002); *Zlatko A. Marković*: Towards an analysis of bearing capacity of mechanical connectors for thin-walled steel elements (2002); *Boško D. Stevanović*: Behavior of composite beams timber-concrete constructed with mechanical connectors at service and limit load (2003); *Vlastimir S. Radonjanin*: Parametric analysis of characteristics of repair mortar from the aspect of their application during rehabilitation of reinforced concrete structure (2003); *Nenad G. Marković*: Buckling of tin beams under the action of the local load (2003); *Mirjana Ž. Vukućević*: Application of elasto-plastic models for soil in design of flexible support structures (2007); *Ljudmila /Timofejev/ Kudrjavceva*: Thrmoviscoelasticity and damage of composites on polymer basis (2007); *Vladan M. Kuzmanović*: Towards a thermal design of gravity dams of rolled concrete (2007); *Špiro L. Gopčević*: Non-linear analysis of structures with cables (2007); *Snežana Mašović*: Redistribution of impacts in subsequently extended composite reinforced concrete beams in time (2008); *Ruža Okrajnov-Bajić*: High early strength SCC in elements with prominent main tensile stresses (2009); *Ljiljana Žugić-Zornija*: Non-linear analysis of cable-stayed bridges (2009); *Dimitrije Zakić*: Research of the parameters of ductility and impact resistance of fine grain concretes micro-reinforced by synthetic fibers (2010); *Marina Ćetković*: Non-linear behavior of laminate composite slabs (2011). At the Faculty of Mathematical and Natural sciences, *Slavko Ranković* defended the dissertation in 1973, and at E.P.F. in Lausanne *Dušan Najdanović* (1987).

There are several dissertations defended by the candidate from other Republics, mostly from Macedonia with very topical themes at the time dealing with earthquake engineering, and several from Montenegro, dealing with experimental and theoretical research of structures. From Slovenia, there is a title of the dissertation related to interaction and rheological material issues - *Houška Mladen*: Interaction of the structure and foundations regarding the static and rheological characteristics of material (1980); From Macedonia: *Apostol Poceski:* Impact type earthquakes and paraseismical construction (1968); *Jakim Petrovski:* Modeling of soil and structure parameters to the dynamical reaction of dug-in foundations (1974); *Dimitar Jurukovski*: Formulation of the mathematical model of two-storey high steel frame applying parametric identification of the system (1981); *Boris Simeonov*: Linear and nonlinear behavior of reinforced-concrete diaphragms of multi-storey structures (1982); *Predrag Gavrilović*: Shear strength of reinforced concrete structures in nonlinear area for cyclic and dynamic loads (1982); *Ištvan Kladek*: Prilog kon primenata na varijacionite metodi za rešavanje na tenki ljušpi (Towards an application of variation methods for designing thin shells) (1983); *Andrej Spasov*: Model of short duration earthquake (1984); *Goce Popovski*: Experimental and theoretical research of connections of prefabricated and subsequently placed concrete (1988); From Montenegro; *Duško V. Lučić*: Towards stability analysis of thin-walled beams (1999); *Pero A. Vujović*: Influence of weathering strain on the ultimate limit states of reinforced concrete slabs stressed in their plane (2000); *Radomir M. Zejak*: Towards an analysis of obliquely bended slender reinforced concrete elements (2003); *Olga Mijušković*: Stability analysis of rectangular slabs using accurate stress function (2008).

2.2. Doctoral dissertations defended at GFs (Faculties of Civil Engineering) in Niš, Priština, N. Sad and Subotica

At GF in Niš, during seventies, there were several leading visiting researchers, among whom the famous prof. Marcel Save, delivering lectures at Seminars, treating the very topical, both then and now, area of the Theory of plasticity. This inspired the teachers and researchers to engage in this area, so a number of dissertations was produced treating the same area and its application for optimal structural design. The following dissertations were defended: *Tomislav Igić*: Towards an optimum design of structures (1980); *Milić Miličević*: One procedure of limit analysis of transversely loaded polygonal plates with the special focus on trapezoidal plates. (1980); *Dobrivoje Stanković*: Finite elements method (1980); *Tomislav Radojičić:* Main stresses and ultimate bearing capacity of elements (1980); *Sreten Stevanović*: Ultimate bearing capacity of linear beams; *Dragutin Rodić*: Towards a solution of the problem of medium columns of bridges; *Popović Branko*: Problems of ultimate failure state of linear systems of classical RC structures (1982); *Grozdana Radivojević*: Effects of variation of stiffness on the elastic stability of linear systems (1982); *Dragoljub Drenić*: Analysis of crack propagation under the action of impact load in the high strength steel (1982); *Novak Spasojević*: Forced damped vibrations of beam systems in plane with a special focus on bridge structures (1983); *Milisav Damjanović*: Towards a solution of architectonic-structural problems of high-rise buildings in the system of spatial frame structures with wall bars (1983); *Dragan Veličković*: Determination of ultimate bearing capacity of arbitrary cross section of thin-walled beam with a deformable contour (1985); *Miodrag Đinđić*: Stability analysis of tunnel structures of horse-shoe cross-section in elasto-plastic environment, depending on construction technology (1985); *Hristo Kapsarov*: Towards aseisimic design of tower-type high RC facilities (1985); *Dušan Petković*: Concrete-concrete composite (1987); *Dragoslav Stojić*: Lateral stability of glued laminated timber beams loaded to bending and torsion (1987); *Verka Prolović*: Foundations of smith hammers as sources of vibration and their influence on adjacent structures (1992); *Ljubomir Vlajić*: Behavior of connections with friction grip bolts at service and ultimate load (1993); *Vladimir Radojičić*: Bending of beams of high early strength concrete with lateral forces (2000); *Marina Mijalković*: Analysis of stress and strain state of spatial linear girders according to second order theory (2001); *Dragan Kostić*: Towards the solution of the stability problem of double catenary systems (2007); *Zoran Bonić*: Towards a theory of calculation of failure by punching shear of foundation footings supported by deformable subsoil (2011); *Slobodan Ranković*: Experimental-theoretical analysis of ultimate limit states of RC linear beams strengthened by composition with NSM fiber composites (2011); *Predrag Blagojević*: Experimental-theoretical analysis of ultimate limit states of micro-reinforced concrete beams (2012).

At the Faculty of Civil Engineering of Priština, the following dissertations were defended: *Vukomir Savić*: Mixed Finite elements method for cylindrical shells (1986); *Petar Čolić*: Ultimate limit states of composite prefabricated concrete structures (1987); *Aleksandar Ristovski*: Behavior of pre-stressed structures in time, under long-term loads (2001); *Mirsad Tarić*: Towards the design of composite structures steel beam – concrete (2004).

AT the FTS (FTN) in Novi Sad, the following dissertations related to structural engineering were defended: *Svetlana Žorić*: Experimental and analytical research of RC walls with opening under seismic load (1990); *Milan Letić*: Structure of the system for automatic designing of industrialized prefabricated housing buildings (1991); *Mitar Đogo*: Towards a theory of foundation design of finite stiffness in multi-layered system (1996); *Ratko Maretić*: Stability and oscillations of a rotating circular slab (1997); *Slobodan Krnjetin*: Towards a determination of necessary fire resistance of concrete buildings (1999); *Đorđe Lađinović*: Multi-criterion analysis of seismic resistance of structures of reinforced concrete (2002); *Jasmina Dražić*: Analysis of interdependence of functional and structural characteristics of buildings in aseismic designing (2005); *Emil Popović*: Development of the model of maintenance of locks with segmental gates at the example of the hydro-system Danube -Tisa- Danube (2007); *Vladimir Nikolić*: Exploration of the newly proposed pile with widened stem (2007); Zoran Brujić: Analysis of ultimate bearing capacity of RC columns bended in two axes (2008); *Tatjana Kočetov-Mišulić*: Behavior of forged connections of bearing wooden wall panels (2008); *Todor Vacev*: Optimum design of a node of the spatial steel grid applying non-linear analysis (2009).

At GF in Subotica the following dissertations in the field of engineering structures were defended: *Danijel Kukaras*: Experimental-theoretical analysis and calculus modeling of behavior of prefabricated beams connected with friction grip bolts (2008); *Ilija Miličić*: Theoretical-experimental analysis of redistribution of the load when determining serviceability of bridge structures (2008); *Danica Goleš*: Rheologoical-dynamic analysis of RC polyhedral shells (2012).

3. BUILTEN, JOURNAL, CONGRESSES AND CONVENTIONS OF JUDIMK/DIMK

3.1. Bulletin and journal and research in JUDIMK and Institutes

Since 1952 when the Association of Yugoslav Laboratories was founded (SJL) until 1966, assembly meetings and Conventions were held, where articles and information were presented. They were related to the activities of RILEM-a, research programs of the Institute, and in 1957 it was decided to publish the Bulletin, which is a precursor of MiK (Materials and Structure). In the later period, the publication of professional and scientific articles commenced. The works of the Institute researchers started to have a larger share in respect to those from the Faculty. Until 1960, there was only the Faculty of Civil Engineering (GF) of Belgrade. GF in Nis was found in 1960. In Priština, in the framework of the Technical Faculty, the Civil Engineering curriculum was taught since 1967, and in Novi Sad since 1971, in Center of the Mechanical Faculty, and since 1974 as a part of the Faculty of Technical Sciences. On the same year, the GF in Subotica was formed, and since six years ago within the State University in Novi Pazar. All the mentioned Faculties were formed with engagement of the teachers from GF of Belgrade.

In the beginning of its operation, SJL held congresses with presentation of papers and discussions simultaneously with assembly meetings. At the congress of 12th June 1958 the main presentation was: Materials and structures in housing construction (M. Maksimović). In it, the need for full prefabrication and assembly of large standard building elements, or application of semi-prefabricated construction was emphasized. This was meant to save timber. Prefabrication of floors and ceiling structures was recommended and modular standard measures and usage of light concrete was prescribed. It was the initial impulse for development of prefabricated systems of building construction. On the occasion of XII Congress of RILEM held in July 1958, Beograd-Zagreb-Ljubljana, and a jubilee issue of the Bulletin 3/1959 was published. In it was published the paper of B. Žeželj: About the new possibilities offered by the pre-stressed concrete applying prefabrication in skeletal structures. This paper was the basis for development of pre-fabricates system IMS–Žeželj. In the Bulleting 3/59 Hubert Rusch published a brief abstract of the paper: a newer theory of bending of RC structures on the basis of the testing in Munich – influence of load duration. This paper affected the orientation of researchers in this field in our country. More details on this meeting can be found in [37].

The Bulletin published the presentations of significant structures. Thus in the Bulletin no. 1/1958 Novi road bridge in Belgrade (M. Radojković) was presented. Apart from that, the results of testing of rebars for pre-stressed concrete were presented and tests of steel by torsion fatiguing. The model of the Hall 1 of the Belgrade Fair (B. Petrović) was presented. By testing the influence of age on the concretes with aluminate cement (V. Matić) it was established that except for the increase of strength till 28 days of age and stagnation until 3 months, there was a decrease of strength after 20 years of 40%. For this reason, it was proposed to add anhydrides to aluminum cement so as to prevent the decrease of strength at high temperatures. Also presented is the methodology of testing of hydrothechnical structures (B. Kujunždić) and adhesive for timber structures. The review of the papers of the members of the Association in 1957 was given, in which the members of IMS stood out. For the purpose of rational application of materials in structures, resistance of concrete to corrosion and finding of protective methods was investigated. Testing of structures on the models (B. Petrović and D. Jevtić) and testing of structures until failure when using domestic rebars for pre-stressed concrete were presented. Construction of structures of laminated timber was and their application in civil engineering were studied. The structural testing with dynamic analysis of mobile load for road and railway bridges was presented. Technical conditions (specifications) for assembly of steel structures were worked on (SS-ČK).

Along with the IX annual assembly, the Association convention was held, with two topics: Structures and materials in housing construction and Problems of support (bearing) structure steel. The influence of RILEM and its Congresses and Meetings (colloquia) was present in the choice of topics of the Association, so the following topic were considered: international standards, concrete and influence of passage of time and physical and chemical causes to deformation, cement, mortar, technical timber, fungi infested timber, metals, masonry columns etc. The laboratory for structural testing GF Belgrade became a member of the Association, and it studied the spatial action of truss structures, with the presentation of tests of concrete halls and the new bridge on the Sava river, with test of welding capacity, fatigue and plastic properties of steel (designations Č52 and Č37).

An experimental multi-storey building was tested in IMS. Light cellular concretes were tested and considered (IMS), and coffered slabs made of pre-stressed concrete with hollow masonry blocks were tested. The issue of industrialization is related to the physics of buildings and flexibility from the aspect of architectonic functional design. A new type of skeletal structure for high-rise housing buildings form which the IMS-Žeželj system developed was presented. In IMS were researched floor ceilings and roof rafters made of pre-stressed concrete. Relations with architectonic requirements were established (standardization of spans between the bearing walls and openings in them, floor height and other). In the Bulletin were published the activities of RILEM and their recommendations (effects of age on concrete, influence of constant load and fatigue). Bearing steel structures were more thoroughly researched in Slovenia.

In the Bulletin no 3/60 was given a review of papers – tasks in IMS in the form: nature of the task, description of work, phases of production, results and publishing, development and improving of the equipment for pre-stressing, testing of stress losses (B. Petrović), standardization of structures in housing construction, standardization of formwork for concreting works, prototypes of high voltage pre-stressed concrete poles, buckling of pre-stressed concrete members.

At the X assembly meeting in Zagreb, in 1960 a necessity to link the research work and "production capacities" was emphasized. Also emphasized was the need to link designing and building with research work. The paper of B. Žeželj about the problems of scientific work in civil engineering was published in Bulletin no. 1/60 where the role of SJL was emphasized. As well as in the other issues, the information about the activities of the Institute was provided. Then were formed the expert committees, among which the Committee for industrialization of building (head - B. Žeželj) and subcommittee for the research of concrete corrosion. Prof. M. Radojković presented the paper about the bridge testing. In Bulletin no. 4/61 there are presented works of the Institute „J. Černi” for hydraulic structures, for the dams with model static tests; work on technical instructions for calculus of stress state in the silo bays (cells); study of arch dams (buckling of arcs). The work of the Committee for concrete in IMS and for soil mechanics and foundation engineering with conclusions was presented. In the issue no. 4/61 Vidan Matić presented the paper on application and protection of steel cables in civil engineering (types and cables) and J. Hahamović about the fire protection.

In the Institute „J. Černi“ was worked on auscultation of several dams, on testing pile bearing capacity and pre-stressing of linings of hydraulic engineering tunnels. The application of photo-elastic methods for testing of stress states and other methods adapted to the specific properties of hydraulic structures (dams) started, not only in Serbia but in other republics as well. At GF, steel was tested to impact, strength of welded seams of for the bridges across the Danube in Belgrade Also tested was the stress concentration of gusset pieces and the effects of apertures on the mentioned bridge, as well as the carrying capacity of the cranes. Stress-optical analysis was applied on the spatial models.

Similarly as in RILEM, on 3rd December 1959, the Committee for concrete was founded, whose chair was professor Dobrosav Jevtić, in order to unite the research work of concrete technology and concrete structures which the collective and individual members of the Association were conducting, and to organize conventions regarding certain issues of Concrete structures which were important for further development and improvement of concrete and concrete structures.

In issue no. 1/62 Borislava Zakić’s paper on the application of the theory of failure in timber structures was published. The activities of the Institute „J. Černi” and Subcommittee for concrete corrosion were described. In issue no. 2/62 V. Joksić wrote about concrete corrosion, and L. Jovanović wrote about the devices for loading of static models in the Institute „J. Černi. In issue no. 3/62 there is presentation about the bridge across the Danube in Novi Sad (B. Žeželj) and about the prefabricated pre-stressed concrete system (PB). V. Brčić wrote about the VI Congress for theoretical and applied mechanics in Split. In issue no. 4/62 B. Kujundžić wrote about the state of affairs in the field of rock mechanics and underground works, and I. Karpinski from IMS wrote about research in the world aimed at increasing of concrete resistance to corrosion. In issue no. 1/63 grouting instructions; arch dam design instructions and photo-elastic tests instructions were published and the text about brittle coatings.

Along with the XI assembly meeting (Belgrade, 1963) the convention of the topical issues in civil engineering was held. The submitted papers treated the then topical issues of materials and structures, but also of soil mechanics and foundation engineering (D. Krsmanović). Since 1963 the Bulletin is transformed into journal Materials and structures (MiK) after the RILEM journal. Very intensive activities in SJL were stirred by the earthquake in Skopje that occurred on 26th of July 1963. In Bulleting no. 2/63 Ž. Hiba wrote about insufficiently strong structures for seismic actions. In issue no. 4/63, D. Jevtić wrote about some problems regarding the earthquake in Skopje with the program of production of technical documents and recommendation and regulations for construction in seismic areas, and authors from Ljubljana wrote about the theory of plasticity. The experience of the engaged researchers of the Institute gained recording the damaged buildings was used in part for making of the first Temporary technical regulations for construction in seismic area. A special issue was published regarding the earthquake in Skopje. In the journal MiK no. 5/63 B. Zakić's paper of the IMS was published - "Design, production and testing of support elements of solid timber and various panels of timber, and testing of wrought and glued support structures, and about durability of concrete structures made with aluminized cement.

The work of D. Jevtić in the European Committee for Concrete (CEB) and B. Žeželj and B. Petrović in the International organization for pre-stressing FIP, which made us an active part in the international cooperation was very important. D. Jevtić wrote about CEB and the effects of the earthquake in issue no. 1/65. In no. 3/65 Natalija Naerlović-Veljković published: Towards the study of thermal deformation of cylinders of non-elastic material, in no. 6/65: Towards the calculation of thermal stresses in a hollow sphere. In no. 4/65 V. Brčić published: Application of photo-elastic coatings when testing structures and their models, and in no. 2/66: Towards photoviscoelastic testing of structures. Z. Pavlović wrote the review of the technical regulations (TP) for support (bearing) steel structures. In issue no. 5/66 V Joksić from the Institute „J. Černi" wrote: Grouting instruction, D. Milovanović wrote about testing of counterforts of multiple arch dams; O. Marković wrote on Standardization of bridges on irrigation and drainage canals, K. Ivanović wrote about the Effects of earthquakes on tall dams. Z. Pavlović and K. Ivanović wrote about the Study of rational designing and construction of locks and gates. V. Brčić et al. wrote about Stresses and abutments of dams, and V. Brčić and A. Pakvor wrote about Photoelastic testing of the nuclear reactor casing and model tests of the stress in the system dam-rock (interaction). The themes of the Bulletin no. 4/66 were: Construction and technical regulations from the aspect of contemporary needs and adaptation of foreign regulations; About control and safety of tall dams. The accent was on the attitude that improvement of the regulations was a condition for improvement of the quality of GMiK (building materials and structures).

The research of the calculation methods of arch dams was performed with static model tests, and on spatial models in the elastic area, and until failure. The tests of the dynamic calculation of dams *in situ* was done on the dams on Zlatibor and Modrac (B and H). In relation to these, the mechanics of rocks and geotechnics was researched, by P. Anagnosti and R. Stojadinović with associates. Ž. Radosavljević with B. Kujundžić and associates in the Institute „J. Černi" developed the design methods of pressurized hydraulic (hydrothechnic) structures tunnels, and published two monographs: Grouting of hydraulic engineering tunnels and shafts under pressure and Pre-stressed linings of hydraulic engineering tunnels under pressure.

Since the Congresses, apart from the papers of the members of the Association are accompanied by the reports, in issue no. 5/66 was published the general report of M. Krstić: New and improved structures which was prepared for the Symposium and XII Congress (Sarajevo 1966). It is underlined that "it is necessary to construct sufficiently safe structures with the minimum consumption of material and labor. It is a basic task of structural engineers and constructors. The science should not provide theoretical elaborations of already realized practical achievements, but it should take the lead". The papers organized in groups were presented. The first group of papers was theoretical, the second was tests of constructed buildings, the third was application of new materials in structures, the fourth was presentation of finished buildings. R. Stojadinović wrote about the potential of founding of new engineering structures on the deposits of waste rock dumps in the industrial zone of Trepča in Zvečan.

In those conditions, there was tendency to use nationally conceived devices for structural testing. In issue no. 1/67 was presented domestic device for production of strain gauge with wide application in structural testing. The cooperation of teachers at the GF Belgrade and Institute „J. Černi" resulted in several papers and monographs. In issue no. 2/67 the following paper was published: Towards the study of underground pressures by Ž. Radosavljević, Čakarević and B. Čolić, which is important for the tunnels. In paper no. 4/67 was published the paper by S. Đurić dealing with mechanics of continuous media: Plastic flow of COSSERAT materials. The decree of work on technical regulations in the field of civil engineering was issued: for pre-stressed concrete (PB); and for concrete and reinforced concrete (BAB); for the walls; for composite structures steel-concrete; for assembly of steel structures and their protection from corrosion. In issue no. 2/68 was published the paper of Đ. Lazarević: Towards the theory of buckling and bending of RC members.

M. Ivković wrote Concrete behavior in the field of plastic deformations as a part of hydraulic engineering structures. He provided the initial impulse in this area with his research and paved the way many young a researcher to deal with concrete rheology. Multiple dams were auscultated, bearing capacity of the piles and pre-stressing of hydraulic engineering tunnels were tested. Application of photo-elastic methods in testing the stress states began, as well as other methods – adapting to the specifics of Hydraulic structures, i.e. dams, not only in Serbia but in other republics as well.

A wider consideration of the research of GF is given in the separate sections. Here is given review of those already published in the journal MiK. Academician Đ. Lazarević pointed out that research work is inseparable from the practice, construction sites, factories and control. That techniques of work and basis for engineering creativity requires knowledge of details of processes taking place in the material used for construction, and particularly interesting for the structural engineers from the economic and environmental aspectist.

Particularly important is engagement of the teachers of GF, particularly of M. Ivković (instructions for calculation of cracks, deformations and rigidity of RC and pre-stressed structures) and D. Jevtić (IMS) at preparation of the Manual for implementation of Code for RC 1972 SJL, i.e. JUDIMK. M. Ivković proposed the calculation of RC member to long lasting load. At that, he proposed the connection of the stress-strain tensor with introduction of time, that is, age of concrete. Also important are the works of J. Lazić with visco-elastic models, M. Sekulović with thin-walled members and implementation of FEM. The design of building structures under the action of earthquakes (D. Dimitrijević), and structure-soil interaction (M. Manojlović).

An extensive list of papers published in Bulletin/journal until 2010 is given in reference [10] and they are available in the office of DIMK Serbia. The journal was not published in the interval 1984-1988.

3.2. Congresses and meetings

Similarly to Bulletin/Journal apart from the papers which were published in the proceedings of the conferences, there were also published the general reports prevalently dealing with the field of material testing. Here were listed only the papers which were in the function of development of our structural design, with undoubtedly significant achievements published in [1] and [8]. The names of the authors were given after the titles of papers or groups of papers. The full list of the papers published in Bulletin/ Journal, which was prepared by V. Denić, are published in the Journal no. 4/2010, pg. 57-116. From the list of the papers, it can be observed that in the beginning, the authors from the Institutes such as IMS , "J. Černi" and others were dominant, while from the beginning of the 1990s those are the authors from the Faculties. Having in mind that the society of structural engineers presents the achievement in the area of engineering structures and bridges, here the topics concerned with the Theory of structures and their testing during construction and under test loads for verification dominated here. Only exceptionally, there are presentations of the bridges across the Neretva river (D. Ćertić) and across the bay of Šibenik (I. Stojadinović) which were the globally recognized achievements at the time. During the last 10 years, there are obligatory themes regarding application of the set of documents EN 1990 to 1999, that is, Evrocode for structures. These topics were treated at the Congresses of the Society of the structural engineers, and in 1995 and 1997 along with GF Belgrade, it organized two large conventions dealing with Eurocode. They and Congresses of JUDIMK were dominated by the papers dealing with Concrete structures, metal and timber structures. Significantly less papers were devoted to masonry structures (ZK). For this reason, DIMK organized several Meetings.

The first was "Addition of floors on housing and public buildings" (2000) with several sections: socio-economic aspects; architectonic-town planning aspects; seismic resistance, structural aspects; geotechnical aspects etc. Convention "Masonry structures in contemporary civil engineering practice" along with other topics dealing with structural and aseismic aspects (2001). Scientific-professional meetings "European regulations …Eurocode - 6 and accompanying regulations " (2006) and (2007) with the topic of simplified design of non-reinforced masonry structures. Durability of structures is very topical in the world and nationwide. Let us mention the Convention "Civil engineering and sustainable development" with different thematic segments, some of which were very important for us: Designing of engineering structures from the aspect of durability – extension of their service life. Conference: Masonry structures – bearing capacity, durability and energy efficiency was held on 24th November 2010. The list of papers presented at these conventions was given in [10]. The papers of the participants, some of them being mentioned further in the text were printed in the proceedings.

At all Congresses, there were submitted the general reports which were composed by the researchers appointed in advance. We mention some of them, who had this role several times: Dobrosav Jevtić, Milan Krstić, Boško Petrović, Miloš Marinček (Ljubljana), Ljubomir Jevtović, Borislav Zakić and others. More details were given in [37].

XII congress 1966 (Sarajevo): About testing of pre-stressed structures using the photo-elastic method (V. Brčić and A. Pakvor); Effects of high temperatures on characteristics of steel for pre-stressing and Influential diagrams for determinations of moments and deformations created due to pre-stressing and other influences (D. Jevtić); Stress and strain state testing on dams using models, L. Jovanović; New possibilities of design of reactor casing of pre-stressed concrete in nuclear power plants (B. Žeželj); presentation of bridges: Road bridge of pre-stressed concrete across the Neretva near Rogotin on the Adriatic highway (D. Čertić) and Construction of the bridge across the Bay of Šibenik on the Adriatic highway (I. Stojadinović); The results of research –Towards an analysis of phenomena and behavior of loaded structures (M. Ratajac); Some pathological phenomena in concrete structures of bridges (B. Zakić); Control and safety of tall dams (B. Kujundžić and L. Jovanović); General report was submitted b M. Krstić (as mentioned previously).

XIII Congress 1969 (Bled): Model studies of a nuclear reactor casing of pre-stressed concrete. Tests by photo-elastic method and application of photo-elastic coatings (V. Brčić with the associates from the institute "J. Černi"); Contemporary fundamental research in the continuum mechanics and potential for application in our civil engineering (R. Stojanović from Faculty of Mathematics); Constants of stress couples (S. Ranković); On the remediation of several buildings by pre-stressing (D. Jevtić and B. Vojinović); General report on theoretical and experimental research of the structures was submitted by D. Jevtić.

XIV Congress 1972 (Haludovo-Krk): Testing of pre-stressed beams and cross sections exposed to limited torsion (M. Muravljov); Redistribution of effects of the linear systems from pre-stressed concrete (D. Jevtić and V. Mihajlović); Time distribution of impacts under the action of stress creep of concrete (Đ. Lazarević et al.); Influence of extremely short-term loading on the behavior of pre-stressed concrete structures (D. Jevtić et al.); Ultimate bearing capacity of locally loaded concrete elements (M. Ivković and M. Aćić); Stiffness of concrete and reinforced-concrete beams at torsion and normal force before and after the onset of cracks (R. Vukotić); General report was submitted by V. Simović.

XV Congress 1975 (Ohrid): Fatigue resistance of laminated glued structures made of poplar wood timber (B. Zakić); Influence of flow and shrinking in the cross sections of RC beams (M. Ivković and Ž. Perišić); Towards the research of parameters of transverse deformations of concrete from the aspect of integral relations between stress and strain (M. Muravljov); Extreme values of stress in composite structures (J. D. Lazić); Parametric resonance of simple beam of highly elastic material (V. B. Lazić); Some issues regarding dynamic testing of bridge models (M. Radojković); General report on Concrete structures research was submitted by D. Jevtić, and on Steel Structures M. Radojković.

XVI Congress (V. Banja): Viscoelastic deformation of thin-walled member of open profiles of pre-stressed concrete tensioned to limited torsion at loading and unloading (M. Muravljov); Potential of generalization of relation between the stress and strain of concrete (V. Mihajlović); Diagram of stress of compression in cross-sections of RC beams with cracks due to long-term loads (Ž. Perišić); Some results of experimental tests of reinforced-concrete rectangular beams loaded by combined bending moment by torsion and transverse force (M,T,Q) (R. Vukotić); Equations of the moment of failure of timber beams submitted to torsion and axial force (B. Zakić); Ultimate stress values in concrete for the flat state of stress, M. Aćić; Limit bearing capacity of locally loaded concrete elements (M. Ivković); Scientific research support to the open industrialization of housing buildings (B. Žeželj); Recommendations of new regulations for testing of reinforced-concrete and pre-stressed structures of the Committee TBS-20-Rilem (B. Zakić). General reports were submitted by D. Jevtić (BK-CS), S. Ferušić (MK-SS) and M. Velkov (seismic).

XVII congress 1982 (Sarajevo): Experimental examination of thermal incompatibility of the concrete components (S. Venečanin); Analysis of stress and strain of pre-stressed TT beams (R. Folić); Limit bearing capacity of complex reinforced-concrete cross-sections at symmetrical bending (V. Mihailović)**;** Influences in statically indeterminate reinforced concrete structures caused by time displacement of support points (M. Muravljov); Influence of yield on the curve of eccentrically compressed RC elements in stress state I (Ž. Perišić)**;** One procedure for calculation of stress in the cross sections of composite systems (M. Ivković and M. Đurđević)**;** Influence of non-tensioned reinforcement on the force losses of the pre-stresses (Ž. Perišić and V. Alender); General reports were submitted by B. Petrović (BK), Lj.Jevtović (MK) and B. Zakić (timber structures).

XVIII congress 1986 (Portorož): Experimental research of behavior of RC beams at short-term loads (D. Bajić)**;** Towards a study of ultimate states of RC T-beam (R. Folić)**;** Analysis of concrete yield in homogenous RC linear structurs applying the force method (D. Najdanović); On the pr-stressed metal structures and on the welding of cold-rolled profiles (Buđevac et al.); General reports were submitted by B. Petrović (BK), M. Marinček (MK) and B. Zakić (DK).

XIX congress 1990 (N. Sad): Testing results of laminated glued timber beams after long-term action of load (B. Zakić i dr.**);** One of theoretical-experimental approaches to determination of elasto-mechanical characteristics of timber (M. Gojković et al.). Presentation of basic parameters of calculation of timber structures according to limit states (B. Stevanović and S. Vasić**);** Energy aspact of behavior of hollow brick panels reinforced with RC grillwork-grid (R. Folić et al.**);** Influence of caple beams on the behavior of RC walls with openings under seismic load (S. Žorić and R. Folić); General reports were submitted by B. Petrović (BK), T. Nikolovski (MK) and B. Zakić (DK).

XX congress 1996 (Cetinje): On matrix modification of stiff joints, in frames, into flexible node joints (D. Bašić et al.); One numerical procedure for desing of anchored support structures (M. Lazović)**;** Bearing capacity of piles loaded by vertical compression and Design of settlement of foundation slabs of finite stiffness (D. Milović and M. Đogo); Determination of the class of cross sections according to Eurocode 3 (D. Buđevac et al.); Design of profiles of aluminium alloys from the aspec of local stability of the cross-section element (B. Gligić)**;** Basic characteristics and calculation of composite beams of timber and concrete (B. Stevanović); Design of composite columns B. Deretović-Stojanović; On the ultimate bearing capacity of horizontally loaded frames (R. Đorđević); Longscrew design for composite beams (N. Marković); Verification testing of members and nodes of spatial steel structure (D. Buđevac et. al.); Methodology for assessment of bearing capacity and reliability of existing railway bridges. (M. Pavišić); Assesment of the status of silo cells for soy and grains and analysis of some test results under test load (R. Folić, V. Radonjanin**,** M. Malešev). General reports were submitted by A. Vujović, R. Pejović and M. Ulićević (BK), G. Srećković (MK), B. Zakić (D K) and V. Brčić for theoretical analysis of structures.

XXI congress 1999 (Belgrade): Determination of indicators of deformability of soil for analsys of interaction with shallow foundations (I): modulus of soil reaction and equivalent elastic soil constant (M. Samardaković); Soil-foundation interaction – incompressible substratum (D. Milović and M. Đogo); Procedure of formation of matrix equation of the foundation girder of variable cross section arbitrary loaded on its ends (V.Prolović et al.). On the assessment of the status (condition) of the Theatre building in Subotica and other structures (S. Grković); General reports were submitted by S. Venečanin (BK), D. Buđevac (MK), B. Zakić (D K) and M. Milićević for theoretical analysis of structures.

XXII congress 2002 (Niš): General theory of laminate slabs – analytical solution for simply supported slabs and Composite slabs with delaminations – analytical solution for simply supported slabs (Đ. Vuksanović, M. and Rakočević); Calculus of bearing capacity moment of the composite cross-section to lateral torsion buckling according to Eurocode 4 (B. D-Stojanović)**;** Influence of differential shrinking on the stress state in the cross sections composed of two concretes of different qualities (S. Mašović); Non-linear dynamic analysis of structures exposed to action of impulse earthquakes (Đ. Lađinović); Aspects of the choice of calculation mode for verification of structural testing results by testing loading (Z.Mišković and Lj. Jović); Testing of composite beam timber-concrete, constructed by mechanical connections (B. Stevanović); Experimental testing of self-tapping screws loaded to tension (Z. Marković); Products on the basis of carbon fibres – tests and appliction and One example of application of carbon strips for rehabilitation of RC structures (M. Muravljov);

XXIII congress 2005 (N. Sad): Research of seismic reliability of old masonry buildings in Belgrade (N. Stojanović et al.)**;** Finite element based on the general laminate theory of plates (M. Ćetković,Đ. Vuksanović)**;** Some dynamic actions on foundations and dynamic properties of soil (B. Folić and R. Đorđević)**;** Design of structures of timber prefabricated housing in seismically active areas (T. Kočetov-Mišulić et al.)**;** Influence of history of loading on the ultimate limit state of pre-stressed elements cracks (B. Popović); Remediation of beams of glued laminated timber with carbon strips (M. Komnenović and B. Stevanović)**;** RC elements exposed to bending, strengthened with additional concrete and FRP (fibre reinforced polymer) (R. Folić)**;** Comparative methods of analysis of reinforced concrete beams strengthened with FRP (R. Folić and D. Glavardanov)**;** General reports wer submitted by R. Pejović and D. Najdanović (BK), Z. Marković (MK), D. Stojić (D K) andi Đ. Vuksanović for theoretical analysis of structures.

XXIV congress 2008 (Divčibare): Application of elasto-plastic models for soil in calculation of geotechnical constructions FEM (M. Vukičević)**;** Approximate calculation of slender RC columns bended in two axes (Z. Brujić); Static treatment of concrete structures with effects of long lasting loading (S .Mašović)**;** Dynamical behavior of wooden ceilings (floor structures) (B. Stevanović and I. G.); Maintenance Models of concrete structures strengthened with composites-(FRP) elements (R. Folić and D. Glavardanov).

XXV kongres 2011 (Tara) and International Symposium: About research and application of modern achievment in civil engineering in the field of materials and structures. Papers: FEM modeling in assessment of real structural behaviour (D. Kovačević); Actual aprroach to shear design of the prestressed and reinforced concrete beams (B. Popović); Models of failure of NSM strengthening method of RC beams-experimental research (S. Ranković and R. Folić); Pile integrity testing SIT method-theoretical basis and case study (D. Berisavljević and N. Šušić); Analysis of stainless steel members in axial compression (J. Dobrić et al.); Reinforcement of wooden laminated beams with carbon strip (R. Solarov et al.); Optimal design of rectangular RC cross-sections subjected to uni-axial bending according to EC 2 (Z. Brujić); Modelling multy-storey RC frames for nonlinear static pushover analysis (A. Radujković et al.); Determination of embedding strength of wood for material dowel type fasteners (B. Stevanović, et al.).

4. RESEARCH AT FACULTIES UNTIL 1976 AND AFTER 1976

4.1. Research at faculties until 1976

It is very difficult to make periodization regarding the scientific (NI) work at GF in Belgrade. There is quotation in [44] that the faculty used to be almost exclusively an educational institution until the WWII and even to sixties. Scientific work of teaching staff was mostly individual and certain number of teachers took part in activities of other institutions, for instance „J. Černi“ Institute. It was only in 1960 and 1970 when the laboratories were founded the cooperation with industry starts, while the scientific work is still individual. In 1976 the Faculty started getting finances from the Republic for three tofive years lasting projects. There is no doubt that in the very beginning academics Ivan Arnovljević and Jakov Hlitčijev, and later academics Đorđe Lazarević, Milan Đurić and Nikola Hajdin had the influence on development of Theory of Structures (Construction). A great influence was made also by professors Vlatko Brčić, especially with his monographs Structural Dynamics [5], Milorad Ivković with dissertation: Concrete behavior in the domain of the limit equilibrium (1962) and Miodrag Sekulović, with book MKE [46] at first and then with non-linear analysis of structures [47]. Academic Đ. Lazarević was the initiator of basic as well as applied researches, and monograph of M. Đurić Theory of composite and pre-stressed concrete structures with original contributions to analysis of those structures (SANU, 1963) by applying time deformations of concrete, was widely used in designing and had strong influence on younger scientists and designers of such structures.

Some of the results of individual work were widely applied in structural analysis, too. Such is for instance integral equations method N. Hajdin applied in calculation of arch dams. Our science was also affirmed by papers related to application of the method of holographic intereferometry in photo elasticity by V. Brčić (Udine-Itali). Papers of V. Bogunović about the slab flexion and elastic stability of slabs and grillwork were published abroad.

The condition of plastic-failure of concrete which was presented by M. Ivković according the own experiments and results gathered from other researchers still enables applying the plasticity theory in solving the problems of limit equilibrium. Work with D. Radenković is adapted to problems of soil mechanics, e.g. determination of soil bearing capacity.

The monograph of Đ. Lazarević: Slender segmented arches as one fold and stepped multiple systems of dams and Stress calculation of eccentrically loaded ring cross-sections and many other papers regarding these problems, enriched the domain of applied theory of structure (constructions) and influenced the development of modern approaches to a calculation of complex constructions, in a domain of limit state and optimum dimensioning. He enriched the topic as well as profession with his papers [48]. By model research of parabolic hyperbolic shells with A. Božanović it was proven that deflection of a compressed field of hyperbolic surface turns two axial system into one axial tensioned stress system on the biggest part of the surface. It should be noted that M. Trojanović with four published books about concrete bridges gives theoretical grounds for analysis of those structures (book one and two) and the books three and four give us the analysis of chosen examples of bridges performed by AB (RC) and PNB (PCS). With his books he strongly influenced the founding of Belgrade school of concrete bridges which resulted in many highest achievements.

Apart from the mentioned contribution M. Đurić developed the general method of deformations in statics, stability and dynamics of structures with wide practical application. His doctoral dissertation is one of first papers with matrix calculation application.

Papers of N. Hajdin and M. Sekulović with Kolbuner, were published in two monographs, about founding (numerical analyses of beams, grids and plates on elastics ground, and for analysis of diaphragm embedded in elastic semi infinite space/body). Well known was a paper of Ž. Radosavljević about the calculations of group of piles. The papers of S. Stevanović in the domain of funding were prominent.

The papers of J. Lazić about highly elastic models [39], M. Sekulović about thin-walled rods and MKE application were important. Calculation of building structures under the earthquake effect (D. Dimitrijević), then structure-soil interaction (M. Manojlović). Research of calculation methods of arch dams with statical models of dam testing on spatial models in elastic domain and to failure. He performed the verification of a dynamic calculations of dams in situ on Zlatibor and Modracu (BiH) dams. In relation to these problems the mechanics of rocks and geotechnics with important papers of P. Anagnosti and R. Stojadinović with associates was researched.

Professors Živorad Radosavljević and Branislav Kujundžić with associates in „J. Černi“ Institute developed methods of designing of hydro technical tunnels under pressure. The result of that were two published monographs: Grouting of hydraulic engineering tunnels and shafts under pressure and Prestressed lining of hydraulic engineering tunnels under pressure. In Bulletin no. 3/60 B. Kujundžić wrote a paper about the observing of tall dams.

4.2 Research in Serbia and Vojvodina after 1976

Scientific projects in the period from 1976-1995 were conducted mostly in cooperation of Institutes for materials and structures of faculties (GF) with Institutes „J. Černi“ and „K. Savić“ and also with GF and Faculty of Occupational Safety of Niš. Projects covered a wide domain of technical sciences with a potential for individual engagement. Those Projects were cross over between fully individual work and phase of organized research in this domain with aims clearly determined. The following projects were conducted:

1. Contemporary problems in research of structures (Milan Đurić);
2. Theoretical and experimental methods of testing and researching of structures, materials and building environments (Vlatko Brčić);
3. Contemporary problems of materials, constructions and environments in Civil engineering 1986-1990;
4. Plasticity and stability of steel structures 1991-1995 (Nikola Hajdin) and later (Miodrag Sekulović) as a basic Sciences Project.

In a project No. 4, steel structures are studied in a domain of plasticity and stability of equilibrium regarding the influence of research on practice of designing and performance. The project included following subprojects:

1) Comparative study of scientific results in a domain of plasticity and steel structures stability and their influence on the change in standards and regulations;

2) Plasticity and stability of the system with application in steel structures of buildings, with the following topics: Plasticity and stability of frame beams, ultimate capacity of rigid angles in a beam-column connection that is made by friction grip bolts and front plates, Boundary bearing capacity of structures in building including effects of diaphragms made from profiled sheets, Performance and protection of steel structures from effects of high temperature,

3) Plasticity and stability of metal sheets of solid and rectangular girders with the following topics: Application of MKE on study of post critical stadium of thin sheets and ultimate capacity of solid and rectangular girders, Experimental and theoretical research of lateral stability of solid girders with or without stiffening that are exposed to effect of concentrated load, Ultimate capacity of I girder with thin web which is exposed to repeated loads, Geometrical imperfections of sheets on constructed structures, Study of ultimate load capacity of thin-walled girders with open and closed profiles;

4) Plasticity and stability of metal shells with applications including topics: The performance of steel shells with stiffeners and ultimate bearing capacity of cylindrical shells formed of corrugated sheet with or without stiffeners. Certain number of topics has a fundamental character and other topics are based on practical experience of fire consequences on steel structures of buildings and imperfections in constructed structures, which formed a base for further analysis. Part of the results from this project was presented on International conference of Steel structures (Budva, 1986.). Project No. 3. included for topics: 1) Elasto-plastic and limit analysis of metal structures and their optimization; 2) Methods and models of numerical analysis of structures; 3) Theoretical researches in the domain of management and economics of building production and 4) Methodology of stress-deformation analysis and limit equilibrium for non linear constitutive connections and non linear criteria of failure in soil and rock mechanics. Results of topics 1 And 2 were presented at symposium called „Contemporary problems of non linear analysis of structures“, and in March of 1993. The monograph was published [47].

From a domain of mentioned projects several master papers and doctoral dissertations were published.

The fundamental research and technological development research have been financed in Vojvodina since 1975, as well as the development of scientific disciplines directly related to the narrow scientific fields during three years. The author of this paper headed the project: The theory of concrete structures and its application in concrete structures – The development of scientific disciplines - 1988 – 1990. From the field of fundamental research it was worked on the development of the joints and connections system in prefabricated building constructions and engineering structures. This was due to the orientation towards the prefabricated constructions (industrial building). The behavior of the concrete constructions with developed layout under the durable effects was researched as well. The research from the field of Construction management was financed too, and it was supported with the funds of Chamber of commerce of Vojvodina. At that period many minor themes were researched for the Federal Chamber of commerce, and those mostly lasted a year. This helped the affirmation of the researchers and younger associates in spite of limited funds. Better research financing started in 1985.

The projects from the field of constructions:

1. Research of concrete and masonry structures under the dynamic load and structure-soil interaction (1985-1990) head R. Folić

2. Research of the principle of design of the structures exposed to impact loads – 1986 - 1990. (V. Vračarić and R. Folić). The project was done for the National defense.

3. Research in the field of designing of supporting structures of electro-mechanical transmitters (for the use of the company "Sever" Subotica, 1988/89 (R. Folić)

4. Development of construction system of prefabricated timber houses "Špik" Ivanjica (1994-1995) (R. Folić).

5. Joint research with the University of Berkley since 1985-1989 on a Yu - SAD project: "Research of the system of joints and connections of prefabricated large panel concrete buildings in seismic areas " (B. Petrović and E. Folić)

In the period from 1991-1995 merging of the Republic and Regional Scientific Communities occurred which led to the gathering of great number of all GFs and IMSs scientists around projects. The organization was superb and the Ministry kept track of the work and the results of the projects. Let us say that it was habitual at that time for the heads of the projects to write an overview in which they showed results of the research. This was serious and public verification of the achieved results. Recently, the poster presentations have been introduced into practice, which is considerably lower level of verification. Here we have only a part of the results showed in the overview for the Project no. 1721: The Research of the Elements and Structures from the Aspect of Bearing capacity, Serviceability and Durability, Including the revitalization of the buildings [23].

The paer [23] shows goals, structures and general results of the scientific research project. The research of bearing capacity, durability and serviceability of the elements and constructions; studying of the behavior of certain engineering structures and bridges in situ and in laboratory; methods of maintaining, remediation and revitalization of concrete, wooden and steel structures; studying of the behavior of machine and facility foundations, concrete and masonry structures under the dynamic load, as well as the application of the theory of plasticity, stability and numerical procedures in the analysis of elements and concrete, wooden and steel structures were included. In five research years more than 200 papers were published.

Extended theoretical and experimental research was undertaken in order to better understand behavior of elements and constructions under diverse kinds of load. This was also important for finding adequate ways of designing, building, maintaining and remediation of building structures constructions. In the course of this process many different problems of contemporary timber, steel, concrete and composite structures were explored. This was also done with different methods and types of analyses of elements and structures in order to adjust them to contemporary problems of structural engineering and enhance them. Many designing regulations given in International societies and Eurocode recommendations [6] were studied, in order to make a theoretical basis for their easier application.

The following issues were studied:

- Bearing capacity, serviceability, durability and revitalization of concrete structures,
- Research of concrete constructions and masonry under the dynamic load,
- Theory of plasticity and problems of contemporary constructions stability and
- Analysis of the steel and concrete structures and large span bridges behaviour.

The themes overlapped in many aspects since they all emerged from the common goals. Those were creation of new elements and constructions with optimization of their silhouettes, which would satisfy not only economic standards but also the needs of safety (bearing capacity, serviceability, durability and maintenance). This led to the fact that after the second year the project was treated as a single set of research tasks.

The influence of the load history on limit states of serviceability of RC and pre-stressed structures was studied. The procedure of strain state calculation in the cross sections for discontinuous actions was proposed, including the change of the active part of cross section after the onset of cracks, applying the constitutive connections of integral type. The procedures for calculation of deflection of partially pre-stressed elements were analyzed, and the procedure of ultimate state of crack openings and their behavior after decompression was introduced. Apart from that, differential relations, i.e. Maxwell model for the state of stress and change of strain in concrete were used for the analysis of stress and strain state in RC and partially pre-stressed elements. The same model was used for the analysis of limit serviceability state in partially pre-stressed elements. The properties of the relaxation function and concrete aging coefficient obtained numerically on the basis of the concrete flow function according to Model Code 1990 and EC 2 were analyzed. Interaction diagrams moment – normal force – curve for the calculation of slender RC elements “column model” were designed.

Starting from the interaction of architectonic and structural designing, contemporary structures in high rise construction were analyzed as well as the choice of the systems for stiffening high rise buildings. Structural systems of prefabricated timber panel houses (MDK), were applied here, with the focus on testing of basic materials, elements and connections. A parallel with the recommendations of EC5 is emphasized for the calculation of basic elements of MDK. The methodologies of assessment of structural status, material testing, causes of damage and remedial measures of specific timber structures were proposed. The report of the state of affairs in the area of composing of timber beams and concrete slabs was produced. The theory of composing is presented with and without introduction of slipping between the layers. A number of issues regarding specific structures of foundations under the dynamic loads, that is, foundations of machines and various industrial facilities were treated (published monograph, R. Đorđević). The report of the state of affairs in many topical areas of structural engineering treating the issues of bearing capacity, durability, serviceability and maintenance of engineering structures, as well as the behavior of elements and structures under variety of loads was submitted.

In order to include the yieldability c of nodes of prefabricated RC frames, the matrix of stiffness of members with the arbitrary level of fixation was developed, and the method of determining the yieldability of corners was demonstrated. Through the numerical analysis, it was demonstrated that there occurs the redistribution of moments between the cross sections in nodes and in the span, as a consequence of yeildability of nodes, so it cannot be ignored, when it comes to prefabricated concrete structures [24]. It is important to mention, that in this area, the initial impulses were given by M. Đurić (INDIS, 1976) and M. Milićević (XVII Congress of Theoretical and applied mechanincs, 1986) from Niš. In their works, they treated static analysis, stability and dynamic analysis. In the process, they used the classic strain method approach. For timber structures, the matrix analysis was developed by (D. Bašić, D. Stojić and E. Mešić), and for steel structurs M. Sekulović, B. Ćorić and R. Salatić. This theme is still topical and researched both locally and globally.

Within the project Development of scientific disciplines which was financed by the Scientific fund of Vojvodina, the work on research of yiealdability (semi-rigid) of joints of prefabricated large pane buildings was continued. The paper was published in 2001 [26]. In the paper the effects of yeildability of joints of concrete large panel buildings (KPZ) with the accent of on the yieldability (semi-rigid) of joints of walls were analyzed. For the analysis of bearing and stiffening walls and mixed systems skeleton-bearing walls the Finite elements method for their analysis is used (MKE). Stiffness matrices used for the calculation of walls with yieldable nodes were given. At an example of a diaphragm between the RC columns of a ten-storey buildings, the results were compared assuming the stiff connection of the columns and diaphragm and assuming that columns are stiff and diaphragm joints yieldable. The effects of redistribution of stress σ_y at the bottom of the diaphragm due to yieldability of joints were presented. It was demonstrated that this phenomenon should be included in the calculus for concrete prefabricated buildings so as to describe their behavior under load in as realistic manner as possible.

It was very important for Structural engineering that for a long period of time, especially until 1993, many teachers and associates professionally engaged in Technical mechanics and Theory of structures took part in the realization of fundamental research. Those activities were coordinated through The Institute of Mathematics of SANU. Thanks to this fact the author of this text headed the theme from the area of Structural dynamics from 1991 to 1993. The same year we encountered many difficulties when the Ministry of Science insisted on separating of the fundamental and technological development research thus preventing the simultaneous work on both of them.

Teachers and associates, mainly from the field of Concrete structures from Serbia, were gathered around the realization of the research program of the Project: ***Research of Concrete Structures*** (1996-1999). The program was very extensive as well as subprojects, which can be seen in the title of the one headed by the author of this paper: *Modeling of the materials, connections, structures and soil behavior as well as soil and structures interaction under static, dynamic, seismic and incident actions and during the fire*. The researchers from N. Sad, Beograd, Niš and Subotica took part in realization of this project. Let us mention some results:

– The problem of structures interaction – foundation – soil with special foundation structures like consoles and anchoring diaphragm walls was studied. Linear and non linear analyses were used;

– Introduction of structure interaction – foundation – soil, in order to improve the method of constructions and foundation structures designing;

– Methods of concrete composition, working of contact surfaces and calculating procedures for cross-section with and without cracks, and with introduction of concrete shrinkage and yield were studied;

– Introduction of new methods of calculation and concrete composition structures modeling;

– Further analysis of static and dynamic behavior of special structures in order to improve large span structures designing;

Let us mention some other projects on which predominantly worked the researchers from Vojvodina, and which were financed from Republic funds:

- The subproject: " EC 8 – Designing of seismically resistant structures" in technical project: " Introducing the Eurocode system and adopting the new methods of designing products and technologies in Serbian structural engineering" (1995 - 2000), Head of the subproject - R Folić.

- Preparation of new regulations and instructions for Eurocode applications for structures in our civil engineering (2003-2005.) – Subproject EC 8: Designing of seismically resistant structures; Head of the subproject R. Folić
- New technologies in research, designing, building and service managing of engineering structures (2003-2005), Head of the project R. Folić, coordination IMS Beograd.
- Improving of inspection, assessment, revitalization and reconstruction of structures methodology, Ministry of Science, Technology and Environmental protection of Serbia, (2005-2008) Head of the research R Folić.

It is important to mention the work of the researchers from Belgrade, N.Sad and Nis Faculties on technological-development project: *Introducing the Eurocode and adopting new designing methods*, headed by Ž. Perišić. The work lasted from 1993-2000 and it continued in 2005 with the translation of the EN 1990 – 1999 document, and introduction of Eurocode into our practice (which was given recognition of merit by the Engineering Structures Society DGKJ for the best scientific work in 1996/7). The author of this paper headed the Subproject: EC 8 – *Seismic Design of Structures*. The task of introducing the Europeans standards into our practice was taken seriously, owing to the contribution of professor Života Perišić. This is evident in the fact that two Meetings regarding this Project were held in 1995 and 1997: Eurocode and Constructive Engineering. The interest in those was great and the attendance enormous. This period of activities was focused on Pre-standards PriEN, not towards EN being published in 2002: EN1990, until 2006. It is also important that we took part in many Congresses organized by Macedonian Constructors Society with introductory papers, such was [25].

The research from 2008 – 2010 was grouped in the following projects:

- Experimental and theoretical research of real connections of reinforced concrete and composite structures under static and dynamic load (B. Stevanović);
- Research of contemporary concrete composite based on domestic raw materials with the focus on potential of applying concrete with recycled aggregate (V. Radonjanin);
- Development and improvement of engineering structures exposed to seismic and incident actions designing (Đ. Lađinović);
- Research of long-term and short-term monitoring methods in engineering structures (Z. Mišković);
- Experimental and theoretical research of dynamic characteristics of pre-fabricated and semipro-fabricated structures and elements from the aspect of serviceability (B. Stevanović);
- Safety, bearing capacity and stability of composite and steel structures in housing and bridge building and new technical regulations (B. Ćorić).

4.3 Experimental and theoretical analysis of T Beam

Since many authors presented some of their papers, here is presented the work on the doctoral dissertation, as well as some later studies. This is the summary: The paper deals with experimental and theoretical research of RC concrete T-beams. Relation between width of the flange and span of girders, quantity of tensile reinforcement in longitudinal webs and occurrence of edge cross ribs to behavior of T-beams under loading have been experimentally and theoretically studied. On the basis of personal investigations and cited data, the determination of the effective width of flange subjected to bending, torsion and normal forces is considered. Data about cracks and deflections of reinforced concrete girders are given. It is shown that relation between width of flange and span has a dominant influence on effective width of flange, while quantity of tensile reinforcement in webs has an important influence on limit state of serviceability with reference to deflections of girders.

Permanent usage of T-beams in concrete structures led to their thorough study. They can be found in monolithic and pre-fabricated construction as a combination of planar structure and beam elements, most frequently flange-web, and the box girders can be processed by the same calculation model. These girders can be exposed to bending, torsion, normal forces or combined action. However, under the term of T-beam we usually think of the elements exposed to bending. Almost all the time the linear theory of elasticity is used for influence calculation of these beams. In case of the statically indeterminate structures, the stiffness of the elements is introduced. Although the stiffness depends on the reinforcement method and condition of the cracks, it is designed for homogeneous cross section, so it particularly important to define the cross sections as accurate as possible.

When the spacing of the webs is large, the distribution of normal stresses is not uniform, i.e. the stresses σ_χ which have the highest value at the flange/web intersection and taper off with the distance away from the joint. For this reason, it is necessary to determine to what distance from the web, in load distribution, there is the interaction from the slab (flange). Thus a phrase ***effective flange width*** (AŠP) is defined. The actual width of the slab is substituted with the smaller width, with which the value of "the highest normal stress $\sigma_{\chi,}$ occurring at the contact of the slab and the web" can be correctly calculated. The distribution of stress in the flange and in AŠP are determined by theoretical numerical analysis and/or experimentally. Here are presented and analyzed some of the results of won experimental and theoretical research of T-girders of reinforced concrete and rectangular girders of pre-stressed concrete under short-term load. Analysis of limit states relates to the serviceability states of RC girders, that is, except the AŠP, the cracks, deflections are considered as well as the problem of the slab-web joint and stability of the flange.

For the purpose of the study of T girders, 7 RC models were tested and tow pre-stressed rectangular girders, under test load, until failure. The chosen form of RC model allowed analysis of two boundary cases: T-girder with cantilever slabs $b_{2/}b_1 = \infty$ and Channel-girder with the internal slab with ***b2/b1*** = 0. On all the girders were measured longitudinal strains ε_χ on the upper surface of the slab, in the transversal direction and along the cross-section height, and on RC models and in tensioned and compressed reinforcement. The deflections were measured at 1/6 of the span on the longitudinal webs, and at ½ of the span in the girder axis and at the ends of cantilever slabs. Onset and development of cracks were recorded for each phase of load.

The load was applied by the hydraulic press "Amsler" over the axis of the web, by a pair of symmetrically positioned forces. Design load was applied in four uniform phases each being 25% of service load.

To achieve full economic effect, it is necessary to achieve equal resistance of individual component parts until the ultimate state is reached. Because of this a part of experimental results is focused on the study of ultimate states. The number of parameters that could vary in these calculations is high, but the program described here included only the most dominant ones. The following were studied:

- Effects of geometry of RC models expressed by the ratio of spacing of longitudinal webs and span of the girders,
- Quantity of tensioned reinforcement in longitudinal webs, and
- Existence of end (lateral) webs.

In order to study the influence of the quantity of tensioned reinforcement in the webs, the first batch of models was concreted with 3 rebars ø 16mm (1.54%), and the second batch with 6 rebars ø 16mm (3.24%) of RA 400/500. Thus, 6 RC models with end transverse webs were obtained. One girder was without end transverse webs. In order to check the validity of the solution of the theory of elasticity, the results of the experiment were compared to the results of the analytical solution of the theory of elasticity, finite element method and final strip method. In the theoretical analyses, linear theory of the first order was used.

The following assumptions were introduced:

– Thickness of the flange and longitudinal webs are significantly smaller than other dimensions, so bending of the flange is ignored, and flanges and webs are tensioned only in their planes and mutually connected along the lines on the intersection of the centerline of the flange and the web, where the deformations must correspond;

– Longitudinal webs receive the load in their planes; and

– Transverse girders are stiff in the direction of their center plane and prone to bending perpendicular to that direction.

– The forces acting in the center plane of the flange are predominant, and can be found in the equation:

– Of longitudinal strain of common fibers of the flange and the beam in the center plane of the slab,

– Of the curves in the center plane of the slab and the beam in the center plane of the slab.

– Of the curve of common fibers in the plane normal to the center plane of the slab,

– Of the rotation of common surface of the slab and the webs.

Applying the finite element method, the effects of the type of load and end transverse webs on the stress and strain state is examined. The structure is idealized with the set of rectangular elements loaded in their plane, and mutually connected in the nodes. It is assumed that the stress is linearly distributed within the rectangle, and the resulting distribution of displacement satisfies the compatibility conditions, only in nodes.

In order to compare efficiency of various methods and study of the surface load on the slab in respect to the equivalent linear load on the longitudinal webs, the finite band method was used.

Effective flange width of a T-beam exposed to bending, depends on a number of parameters related to cross section and loading, of them most important being:

– Ration of the flange width and span of the beam b_1/l or b_2/l, where b_2 is the width of the console slab, and b_1 is the ½ of the spacing between two webs,

– Static system, that is, support conditions and position of the cross section where the effective width is calculated,

– Types of load (uniform distributed, concentrated etc.),

– Form the form of the cross-section, and

– From the fixing conditions at the ends of the beam (degree of securing the beam against torsion).

The percentage of tensioned longitudinal reinforcement has no significant effect on the average value of AŠP. Apart from that the experimental research of AB (RC) girders are limited by the scope and by the varied parameters, they, combined with theoretical analyses, enable recommendation of the simplified expressions for determination of AŠP. The expressions are analyzed in detail in the book [45].

Comparing the size of the effective flange width under bending achieved experimentally on the full scale models, and theoretically by the application of linear theory of elasticity, we obtained acceptable compatibility in almost all the cases. Somewhat higher values were obtained in experimental research. So it can be concluded that for determining the effective width of RC and pre-stressed girders the linear theory of elasticity may be applied, because the calculation results are on the side of safety. Out of several varied parameters, the influence of the flange width and spacing of the zero points of moments is important for the effective width, and the quantity of tensioned reinforcement in the webs is almost negligible. The favorable effects of end transverse webs on the effective flange width in the zone near the supports were observed. A higher quantity of tensioned reinforcement of the longitudinal webs results in economical T-beams with wide flanges, from the aspect of stress usage, but for them it is necessary to ensure the limit states of deflection are not exceeded. Existence of end transverse webs has a positive effect on the behavior of T-beams with wide flanges, both from the aspect of the stress and the aspect of development of cracks and deflections, so they should not be deflected, unless it is really necessary. In the book [45] are given the recommendation for inclusion of torsion and normal forces when determining the effective width of the flange.

5. ANALYSIS OF STRUCTURES

Numerical analysis of structures is always a theme of interest in structure research, beside the experiments. Development of computers and their mass usage, especially in the last 50 years, enabled considerably more accurate analysis of engineering structures under mechanical, thermal and other action. Computers changed the way of calculating and enabled the development of modern numerical methods which can be applied in research and designing. Finite element method (FEM) due to its simplicity, mathematic basis and clear physical meaning pushed aside other methods. Very important contribution to the application of theoretical results in practice gave M. Đurić in the field of composite and pre-stressed structures; V. Brčić in the field of structure dynamics and M. Sekulović for application of FEM in structure analysis.

For the design of engineering structures different approaches are used. Firstly, global analysis of structures is done and then analysis of its elements and cross-section. The design according to permissible stress is abandoned except in the case of pre-stressed and composite structures. Since it was necessary to provide safety in the case of failure as well as serviceability, the concept of design according to ultimate limit states was generally accepted. Those are the states limited by some value the structure should not exceed according to the requirements the structure should meet in its service life. The most important of all mechanical properties of material is its relation stress – strain (working diagram) and because of this theory of elasticity and theory of plasticity are used in the design. Also, very important is the type of load (short-term, long-term, static and dynamic) and the influence of the environment temperature, humidity…) In the working diagram the most important are the points of elasticity limit, beginning of plastic deformations and failure deformations.

The most complex designs are for RC structures and composite structures steel-concrete, wood-concrete, and in the last few decades concrete-concrete. The reason for that lies in the fact that concrete is a heterogeneous material, which apart from the shrinking deformations during setting also possesses the viscous properties, that is, deformations increase under constant stress in time. Geometrical structures also conditions application of appropriate models (linear, planar, spatial) and the purpose of the structure.

For the design under the short-term stress, as it is well known, linear and non-linear analysis models are mainly used. Rise of computers and their advanced application facilitated RC structures analysis on more complex models than before, when the linear model was used. Non-linear properties of material, dominant in reinforced concrete emerged from the onset and propagation of cracks and non-linear stress-strain relation for steel and concrete and interaction of concrete and reinforcement. Non-linearity of structural relations is apart from the mentioned things a consequence of the non-linearity of the relation of bond stress and local slippage of reinforcement. Contemporary numerical models based on FEM facilitate easier analysis of real behavior of RC structures which is particularly interesting for special engineering structures. For analysis of bended beams and planar girders under short-term loads, the models based on FEM were analyzed. For analysis of these beams, emergence and propagation of cracks, effect of bond of concrete and reinforcement, transfer of shear forces in the cracked concrete (local effects) should be studied for determination of global behavior and ultimate load.

Three procedures are implemented: a direct procedure with classic finite elements and the material non-linearity is introduced through the effective stiffness, determined by experiments. The second procedure is with the layers of finite elements. The flat stress state defined by the biaxial relation stress – strain is assumed in the layers. The propagation of cracks along the height of the cross-section is facilitated in this way. Discretization along the height and width is used for analysis of complex stresses. The deficiency is the inability to include the emergence of diagonal (slant) cracks and shear failure, and for a more accurate analysis, a large number of layers is required. The third procedure is application of 3D finite elements. They are successfully used to analyze the beams where shearing is the key factor of concrete failure.

Constitutive relations depend on the stress state and they are based on experimental results. They do not take into account the time factor, and they describe the material behavior: linear and non-linear elasticity theory, plasticity theory endochronic theory of plasticity and nonlinear theory of failure mechanics. The linearly elastic, non-linearly elastic, hyperelastic and hypoelastic model of material behavior are based on the Hooke's law. Non-linear connections S-D irrespective of the material properties (elastic, plastic or endochronic model) are expressed by total (secant) or incremental (tangential) form. Total formulation is limited to monotones, but for its simplicity, it is often implemented for description of non-linear behavior of concrete at in-plane and spatial pressure state. More on this can be found in the paper by Sekulović, Vuksanović and Pujević [45] and papers by T. Tassios, and other authors which included analysis under long term load, and also in [47-48].

6. TOPICAL PROBLEMS AND DIRECTION OF FUTURE RESEARCHES

In order to provide the appropriate safety of bridges to seismic action, the following factors are significant: choice of the structure as early as in the conceptual design phase, modeling and analysis of the structure, as well as the formation of details. For the more significant structures such as the bridges of medium and large spans, in seismic areas, the first phase of the selection of structure, i.e. conceptual designing is significant. In the last twenty years of the last century, the modern concept of seismic protection and control of structures has been developed. Apart from the basic isolation, active and semi-active protection of structures is used [7]. These problems were treated in the paper [29]. The choice of isolation devices is very important, since through their application it is possible to ensure considerable reduction of seismic forces, so that the structure could even remain in the elastic area. The method of designing of some of these protection in order to reduce the damage to the structure during earthquakes and providing of adequate performances is topical. In the recent years, the application of integral bridges is topical, and they are gaining importance because of the higher durability in respect to the classic bridges [40].

For the concrete bridges, and particularly for the prefabricated bridges, the continuation of supports on bearings allows rational usage of material and better service performance, in particular the durability. The beam continuity is most frequently done by pre-stressing: it is monolithic, when all the cables are pre-stressed in situ, that is post-tensioning is applied; and non-monolithic when the prefabricated elements are used, such as the simple beams, and the continuity is effected on the bearings by concreting the joint in situ [23].

Concrete structures (CS) are designed so that they can satisfy requirements regarding safety, serviceability, durability and aesthetics throughout their design service life. Present design procedures regarding CS required by national or international codes and standards such as Model Code Euro International Committee of Concrete (1993) now Federation Internationale du Beton (FIB), EN, ACI, RILEM, etc. are predominantly based on strength principles and limit state formulation. The durability aspect is a natural extension of the classical resistance verification where deterioration effects are normally neglected. The review of literature and some recommendations are presented referring to the design of structures aiming to attain greater durability of CS. The accent is put on the theory of reliability, failure probability and service life probability. The basics of this analysis are given through the principles of performances and service life [12], and deterministic and scholastic methods using the lifetime safety factor [33].

Structures may be subjected to extreme events during their design-service life, which can lead to unforeseen consequences. Such situation may be caused by natural disasters such as strong earthquake, or from human errors (for instance gas explosion). Specific approach of designing PC building structures under seismic actions and abnormal loadings is described in [28]. Recommendations for design and interventions aimed to prevent progressive collapse in case of failure of part of structures are given. Alternative ways of load transmission are considered as well as measures for increasing the overall stability. The reduced the risk of progressive collapse following approaches, or tear combination are applied:

1. Reducing the risk of accidental loading.
2. Preventing the propagation of a possible initial failure.
3. Designing the structure to withstand accidental loading.

In general, accidental loads can hardly be eliminated. In design all efforts should be made to reduce the risk of accidental loading as much as possible. Impact loads are the subject of an extensive book [3].

Prefabricated-monolithic structures represent a rational combination of prefabricated and monolithic structures, as they combine the advantages of one and the other. Their application accomplishes a high level of industrialization, savings in material and labor, faster construction and more reliable quality control of materials and workmanship. This contributes to their wide application in bridges buildings and other structures. These structures are constructed faster in respect to those built in a classic way. Prefabricated elements substitute the cladding and scaffold and accept the subsequently constructed concrete elements.

Partially pre-stressed structures (PPN) occupy the space between the classic RC and completely pre-stressed structures (PN). When the reinforcement is not pre-stressed, the level of pre-stressing equals zero (classical RC structure). The other end of the scale is when the degree of pre-stressing equals one, and those are completely pre-stressed structures. PPN structures contribute to a more economical construction of many kinds of buildings, industrial and engineering structures, and bridges [30]. This area of concrete structures is very topical and has a great importance for theory and practice of concrete structures.

Composite steel/concrete structures are used widely in modern bridge and building construction. The very large amount of theoretical and experimental research, design application and construction work carried out has shown the efficiency and economy solution of composite structure. In [31] is presented the current state of affairs related to design and analysis, based on quoted references, in steel-concrete composite structures.

It is important to mention that in the last 20 years adaptation of our technical regulations with Eurocode is topical. These standards were enacted in the period from 2002 to 2005. These documents were also used in the monograph with useful analyses and propositions. Adaptation of our technical regulations with EN is very topical, it is worked on in our country, and the submitted manuscript includes theoretical foundations and other aspects of analysis and design of composite structures harmonized with European standards. Since 2006 the Faculty of Civil Engineering completed translations of several documents, and on this occasion several seminars were held in Belgrade, Novi Sad, and Niš. Recently the Institute for Standardization of Serbia were formed the Working groups working on the final adoption of these documents. The lacking finances represent large difficulty to proceed. In the paper [51] it was emphasized that one of the main priorities of the ISS, as a national body of standardization is the goal of obtaining the status of a full-fledged member of CEN. It is one of the conditions for Serbia to enter the EU.

Directive CPR 305/2011/EEC poses the conditions for putting the construction products (GP) on the market, by establishing the harmonized rules which would express performances relating to the important properties of GP and the requirement for CE designation. Among the important requirements are: mechanical resistance and stability, safety in case of fire. The following are specially considered: re-use or possibility of recycling of construction waste (GO), materials and parts after demolition; durability of GO; usage of raw materials and recycled materials which do not endanger the environment. Harmonized standards represent the document basis for production of national technical regulations and codes. The difficult part is that we do not have sufficient data for national parameters. (snow, wind etc.).

Among 27 committees actively working in the area of civil engineering, the following ones have the importance for structural engineering:

- U104- Concrete and concrete products;
- U125-Masonry structures;
- U167- Structural bearings in civil engineering;
- Engineering structures management;
- Concrete structures designing;
- Designing of steel structures, steel-concrete composite structures and aluminium structures.
- Designing of timber and masonry structures.

In the area of geotechnics, each alternate year, in the organization of the Association of Civil Engineers of Serbia, a scientific-professional meeting is held where the topics important for structural engineering are discussed: Models of geomaterial and numerical methods; Prediction and observation results of structures, observation method; Soil consolidation, reinforcing, grouting, geotextile and other; deep excavations and tunnels; piles, diaphragm and other technologies of foundation engineering; micro-zoning and seismic risk for the purpose of structural analysis to seismic action.

The still standing concept of seismic protection is still based on the design of the structure for the action of design earthquake (return period $T_r \approx 500$ y.). The bearing capacity of the structure is determined for the seismic forces which correspond to the given design level, determined by the application of the reduction factor (dependent on the capacity of deformation (ductility). From the concept based on the force which was valid in the 90's of the previous century, the approach was changed to the displacements, and nowadays, the concept based on performances and structural damage is very topical. In the paper is formulated the mathematical model for static and dynamic analysis of horizontally loaded tall buildings [18]. In the area of earthquake engineering analysis of irregular structures of buildings and beam bridges is very topical. In order to avoid irregularities, it is insisted on conceptual designing and choice of regions for energy dissipation so that they are accessible for checks and repairs. In case of extremely long bridges which rest on non-homogeneous layers of soil, in order to avoid differential displacement, additional separation (expansion) joints are introduced.

Due to the small seismic resistance of masonry buildings, the issue of their reinforcement, pre-stressing and post-stressing is very topical. The latter method is very efficient for reinforcement of masonry buildings, which was used by B. Petrović on Kamchatka, Russia.

In interventions on concrete structures, very often are used authentic or repair materials which calls for formation of very narrowly specialized teams. Lately strengthening of reinforced concrete element's sections with externally bonded fiber reinforced-polymer (FRP) composite materials is very popular, and in which the bond of additional elements and the substrate is important [43]. The same path of application of NSM method and pre-stressing of is FRP reinforcement in order to strengthen concrete structures is topical in our country, lately. As there are no national or international standards for dealing with the design of such sections, recent guidelines given by American Concrete Institute, International federation for concrete (FIB) and Concrete Society Council (UK) can be used.

In the period **2011 – 2014** the Ministry financed the following projects related to structural engineering:

- Development and application of scientific methods in designing and building of highly economical structural system by implementing new technologies (M. Nestorović);
- Research of potential for application of waste and recycled materials in concrete composites, with the evaluation of their influence on the environment, for the purpose of promotion of sustainable civil engineering in Serbia (V. Radonjanin);
- Development and improvement of methods for analysis of structure and soil interaction on the basis of theoretical and experimental research (V. Prolović);
- Development and application of comprehensive approach to design of new structures and assessment of safety of existing structures with the aim of reducing the seismic risk in Serbia (Đ. Lađinović);
- Research of the impact of traffic vibrations on structures and people with the goal of sustainable development of cities (M. Petronijević);
- Research of status and methods of improvement of engineering structures from the aspect of serviceability, bearing capacity, cost-effectiveness and maintenance (Z. Mišković).

Topical themes dealing with seismic were present at XV World Conf. of earthquake engineering (EE) in Lisbon, Portugal, from 24th to 27th September and those are: Geotechnical EE; Seismic behaviour of engineering structures; Assessment and retrofitting; Lifeline systems; Social and economic aspects of earthquake. In the paper: Effects of multiple earthquakes on the seismic response of structures – Contemporary civil engineering practice, N. Sad (A. Liolios, 2012), it was demonstrated that apart from the action of an isolated earthquake, it is necessary to introduce the multiple earthquakes.

Numerous papers related to the seismic enhancement of the existing structures, mostly by applying FRP elements [27] and [34]. It is the topic of several journals, such as [2]: The overall performance of hollow clay tile infilled RC frames strengthened with carbon fibre/reinforced polymer (CFRP) materials is experimentally investigated in the paper: Seismic strengthening of infilled reinforced concrete frames with composite materials (S. Ozden, et al.) and others. The mentioned journals treat several other topical issues. Control of dynamic response of structure is the new philosophy of designing [7], with the potential that the structure is transformed from the passive status into active subject able to adapt to seismic action. Particularly topical are the problems of interaction structure-foundation-soil [32], and the meetings with such topics are organized [42] and tend to be organized in the future. The papers of soil-foundation interaction with introduction of viscous properties are still topical [22], and classification of damage and its causes as in the papers [16] and [17] which were the result of the work in the technical committee 104 DCC (Damage classification of concrete) of RILEM, where the author of this text worked with B. Zakić 1987-1992. In addition, B. Zakić worked in several other committees.

Improvement of Bridge Management System (BMS) is the subject of many researches such as [11], [13] and [50]. It is attested by the site http.://www.dot.wisconsion.gov (Wisconsin Department of Transportation, WisDOT Bridge Manual, Madison, WI, 2010). Very topical is the applicaton and improvement of orthotropic slabs in large span bridges, [36], and the meeting IDE was dedicated to innovation as an important factor of development, and it was held in Niš [35].

Publication of topics issues is active in the region, too, such as e.g. Analysis of effects of reduction of stiffness on the seismic resistance of structures [9], paper: Seismic dampers in engineering structures (A. Nižić and D. Meštrović) and several papers dealing with the same issue which are published in several last issues of the journal. The forecast model for determination of fire resistance [38] and numerical model for anticipation of structural behavior in fire [49] are also very topical and will continue to be such.

The journal GMiK (CBMS) published by DIMK Serbia is regularly published in Serbian and English. The journal has an international editorial board and it is open for the authors from the region and other countries.

7. CONCLUSION

From the list of doctoral dissertations in the field of structural engineering, it can be stated, that they are, in major part, the result of individual work and enthusiasm of the individuals. From the list and brief analysis of papers published in Bulletin/Journal MiK and presented at the Conventions and Congresses of DIMK, and the list of scientific-research projects at the faculties and institutes, it can be concluded that they represent a significant contribution to the structural engineering of Serbia. It is particularly important that these results were realized by the modest finances available to support science in Serbia.

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SOCIETY FOR MATERIALS AND STRUCTURES TESTING OF SERBIA
DRUŠTVO ZA ISPITIVANJE I ISTRAŽIVANJE MATERIJALA I KONSTRUKCIJA SRBIJE
International Conference "Marking 60 Years of Operation of DIMK
Belgrade, October 19, 2012.

Dragan Ivanov[1], Goran Markovski[2],
Toni Arangelovski[3], Darko Nakov[4]

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RESULTS FROM SOME OF THE RESEARCH PROJECTS AND EXPERIMENTS REGARDING STRUCTURES AND BUILDING MATERIALS IN REPUBLIC OF MACEDONIA

Abstract: *In this paper, written on occasion of 60 years anniversary of SJL-JUDIMK, knowledge from testing, research of material properties and behavior of structures done for the construction of large structures are given. Also results from theoretical and experimental research within the research projects are given selected by the authors. The results from the testing of 54 beams to determine the influence of nonprestressed steel reinforcement on the behavior of prestressed concrete elements are precious (Research project 1). In the Research project 2, 3 and 4 the continuous 15 years research on the influence of live load on the time-dependent prestressed concrete elements behavior, high strength concrete elements and steel fibre reinforced concrete elements is presented.*

Keywords: *testing, prestressed, high strength, steel fibre reinforced concrete beams, long term load*

NEKI REZULTATI ISPITIVANJA I ISTRAZIVANJA MATERIJALA I KONSTRUKCIJA U R. MAKEDONIJI

U ovom radu, sastavljen povodom 60 godina od osnivanja SJL-JUDIMK-a, prikazani su saznanja u vezi eksperimentalnih ispitivanja, istrazivanja materijala i konstrukcija za potrebe velikog broja objekata.Dati su i rezultati teoriskih i eksperimentalnih istrazivanja u ovkiru istrazivackih projekata po izboru autora. Dragoceni su rezultati ispitivanja 54 opitnih nosaca za izucavanje uticaja nezategnute armature na ponasanje prethodnonapregnutih elemenata. U istrazivackim projektima 2, 3 i 4 prikazani su rezultati 15 godisnjeg kontinuiranog istrazivanja uticaja promenljivog opterecenja na ponasanje prethodnonapregnutih elemenata, elemenata od betona visoke cvrstoce i elemenata armirani celicnim vlaknima.

Kljucne reci: *prethodnonapregnuti, visokojakostni, fiber betonski elementi, dugotrajni uticaji*

[1] Prof. Dragan Ivanov, ddivanovi@t-home.mk
[2] Prof. Goran Markovski, markovski@gf.ukim.edu.mk
[3] Doc. Toni Arangelovski, arangelovskitoni@gf.ukim.edu.mk
[4] Asis. Darko Nakov, nakov@gf.ukim.edu.mk
University Ss. Cyril and Methodius, Faculty of Civil Engineering, Skopje, Republic of Macedonia

1. INTRODUCTION

Scientific knowledge of the individual researches is the basis and condition for the progress of those activities. The progress of construction works depends on the level of scientific knowledge.

For efficient use of the scientific knowledge and organization of the research work, in 1952 Federal association of Yugoslav laboratories for testing and research of materials and structures was established, abbreviate SJL. Before the secession of Yugoslavia, change of the name was made from SJL to JUDIMK Yugoslav association for testing materials and structures. In the period of 40 years of existence SJL justifies its existence. Organized through the central laboratories-institutes in each republic it led and coordinate the research work in the field of construction materials and structures.

Accommodate in small offices, limited resource materials and staff, SJL posing to solve numerous problems in the field of construction economy. The need for the adoption of standards for testing materials and technical rules for the design and execution of buildings was a first priority. These issues were very important in this period of intensive construction works relating to various structures.

Activities to organize symposium on testing and research of materials and structures are worth to be mentioned here. These symposiums were organized every three years in different republican centers, where the results of performed tests and researches in the previous period were presented.

Thus professional public was informed for the scientific knowledge in the field of construction materials and structures. In the years of the formation of the Federal Association of Yugoslav laboratories SJL, the only laboratory in the Republic of Macedonia, supplied with minimal equipment, was laboratory at the Technical Faculty in Skopje. It was established primarily for the teaching activities of Civil Engineering and Architecture department at the faculty. Intensive construction works in Macedonia, especially after the 1963 earthquake, met the necessities for many laboratories. The existing laboratory was transformed into two departments: Laboratory of the Faculty of Civil Engineering in Skopje and Institute for testing materials. For the purposes of the construction economy, in the Republic of Macedonia, Civil Engineering Institute "Macedonia" GIM was established following the example of other republics, with a tendency to become a Republican institution. After the earthquake in Skopje, the Institute of Earthquake Engineering and Engineering Seismology was founded at the University "Ss. Cyril and Methodius". EEES Institute is widely acknowledged institution specializing in the field of earthquake engineering. Featuring high-quality personnel, contemporary equipment and technique, the institute can perform the most complex experiments on a seismic platform. Because this paper is written on the occasion of the 60-th anniversary of SJL, reminding of the terms of scientific research was necessary.

In the past period of activities, in laboratories in Macedonia, numerous tests were conducted for the needs of construction works on structures, as well as researches to clarify some behavior and phenomena in concrete that are still insufficiently studied.

In this paper some results of testing and research carried out in the Laboratory of Civil Engineering in Skopje are given.

For the purposes of the construction of various facilities, tests of various types of concrete were made with the use of different materials that have to meet a number of quality requirements: compressive strength at different age, tensile strength, freezing/thawing resistance, water permeability, thermal compatibility and other properties.

For the bridges with prestressed concrete beams mandatory were tested: shrinkage, creep and modulus of elasticity. Concrete with set of twelve quality requirements was used in the production of pretensioned prestressed concrete sleepers for Macedonian Railways.

For pretensioned prestressed concrete beams, despite the above mentioned tests, also tests were carried out to define adhesion between concrete and prestressed reinforcement in the form of wires or tendons. Adhesion is an important parameter for the transfer of forces from the reinforcement on concrete. Data base of results from "steel fiber reinforced concrete" and self-compacting concrete is quite small because the scope of the application is lower. Fiber concrete and self-compacting concrete were used in the rehabilitation and strengthening of bridges. It is worth to mention the activities of the inter-department team for bridges formed to meet the needs to strengthen bridges in Macedonia. A solution was chosen in which the lack of reinforcement in the cross-sections, carbon materials were added: strips or wraps made of carbon fiber. Using modest experience in this field, a mathematical procedure for proof of the necessary strengthening was established, the design of the required carbon materials, as well as precise technical conditions for execution of the construction works. During 2001 and 2002 the strengthening of the first series of 19 bridges wad done on the road section Petrovec-Bulgarian border. Procedure for the strengthening and first experiences was used for the bridges on other sections of the road network in the Republic of Macedonia.

Despite the wide use of prestressed concrete in different types of structures, it also represents a wide research field. In Republic of Macedonia, frequent utilization of prestressed concrete structures was registered in the years after the Skopje earthquake. Before the earthquake in 1963, according to the research of the authors, in R.Macedonia only a few buildings were built with prestressed concrete elements.

The use of prestressed concrete in these structures was not the cause of damage or destruction. Insufficient frequent use of prestressed concrete can be explained by the lack of education of the staff in that area. The number of the companies that have been trained to work on the IMS system at that time was insufficient. The state of scientific research is a factor on which the practical application of prestressed concrete depends. Shortly after the earthquake in Skopje, a number of authorities imposed an opinion that prestressed concrete structures are not suitable for use in seismically active regions. But during the years of intensive construction these opinions were rejected. But despite the wide utilization of prestressed concrete structures, it remains a vast field of research.

For investigation of the effects due to partially prestressing, as well as the effects of creep and shrinkage of different types of concrete and in different conditions, many research projects were done or are currently undergoing.

In this paper, results from the following research projects are presented:

Research project 1: Theoretical and experimental investigations of prestressed concrete elements. Author: Dragan Ivanov

Research project 2: Influence of live load on the time-dependent prestressed concrete elements behavior. Author: Goran Markovski

Research project 3: Time-dependant behavior of high-strength concrete elements under action of variable load. Author: Toni Arangelovski

Research project 4: Time-dependant behaviour of steel fibre reinforced concrete elements under sustained loads. Author: Darko Nakov

2. DRAGAN IVANOV: THEORETICAL AND EXPERIMENTAL INVESTIGATIONS OF PRESTRESSED CONCRETE ELEMENTS

2.1. The aim of the research

Partially prestressed concrete elements consist of three different materials: concrete, prestressed tendons and soft steel reinforcement. The bearing capacity and the deflections depend on the characteristics of the cross sections, as well as the mechanical characteristics of the materials: compressive and tensile strength, modulus of elasticity, characteristics of the prestressed and soft steel reinforcement and their arrangement in the cross sections. Experimental program was predicted, in order to find out the influence of all these parameters on the bearing capacity and the deflections of prestressed concrete elements.

2.2. Experimental program

The experimental program consists of 54 full scale beams from prestressed concrete. The beams have cross section dimensions 15/28cm and total length l=300cm, and are divided in series and 20 groups, each with few identical beams. Control specimens were casted, in order to test the compressive strength, tensile strength, modulus of elasticity, the deformations due to shrinkage, the compressive strength at the time of prestressing and the time of testing of the beams. In the beams from Series I and II, the prestressing force and the amount of prestressed reinforcement were varied. In the beams from series III, IV and V, the amount of soft steel reinforcement was varied. Smooth, ribbed and Bi reinforcement were used, according to PBAB 87. The characteristics of the beams are presented in Table 1.

Serija	Grupa	Broj opitnih nosača	Projektov. početna sila [kN]	Armatura za preth.naprezanje		Nezategnuta armatura		
				br. žica u kablu nø	početno napr. [MPa]	vrsta armature	dole nø	gore nø
II	A_2^*	3	0	0	0	GA 240/360	2ø12 G	2ø10 G
	B_2^{**}	3	140	6ø5	1200			
	C_2	3	210	9ø5	1200			
	D_2	4	280	12ø5	1200			
III	A_3	4	140	6ø5	1200	GA 240/360	–	2ø10 G
	B_3	3					2ø10 G	
	C_3^{**}	3					2ø12 G	
	D_3	3					2ø14 G	
	E_3	3					2ø16 G	
IV	A_4	3	140	6ø5	1200	RA 400/500	2ø10 R	2ø10 G
	B_4	3					2ø12 R	
	C_4	3					2ø14 R	
	D_4	3					2ø16 R	
V	A_5	3	140	6ø5	1200	BiA 680/800	2Bi 80	2ø10 G
	B_5	3					3Bi 80	
	C_5	3					2Bi 113	

Table 1. Characteristics of the beams

High strength concrete was used with compressive strength in the range of f_b=65-70 MPa. Agreggate with four fractions from the Vardar valley was used with D_{max}=31.5mm and the used cement was PC450 with amount of 420kg/m^3. The concrete is with plastic consistency, water cement ratio of w/c=0.35 and superplasticizer, type fluidal. The casting was done in special steel formwork made from steel „U“ profiles. High rate frequency formwork vibrators and internal vibrators were used during the casting. The prestressed steel was patented special high strength steel with diameter 5mm with the following mechanical caracteristics: $f_{0,2}$=1700MPa, f_p=1850MPa. In all beams one tendon wich contain 6.9 or 12 steel wires was inbuilt. The tendons were stressed with the system IMS using hydraulic complet BC-30. The protection of the tendons was done with injection of cement paste under pressure.

The soft steel reinforcement in the upper compressive zone of the cross sections in all beams is 2Ø10, made of smooth steel GA 240/360. The soft steel reinforcment in the beams from group III, IV and V is from three types of reinforcement defined in PBAB 87: GA 240/360, RA 400/500 and BI 680/800. The soft steel reinforcement in these three groups is different not only due to the different mechanical caracteristics but also due to the bond with the concrete. The arrangement of the reinforcement in the cross sections is presented in Figure 1.

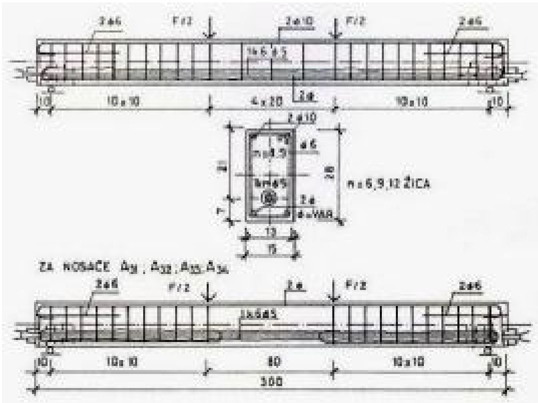

Figure 1. The arrangement of the reinforcement

After the injection of the tendons and at least 7 days for achievement of the needed strength of the cement paste used for injection, ultimate load testing was performed of all beams. The beams were tested in bending with two concentrated forces , as it is presented in Figure 2. The intensity of the forces was increased in many cycles up to failure. The application of the load was done with special portal frame and using hydraulic piston, type „AMSLER“ with capacity of 500kN.

During testing of the beams, deflections, strains in the prestressed tendons, in the soft steel reinforcement as well as in the concrete on many measurement points were measured. Measurements were done in all loading and unloading cycles. Measurement of the strains was done with strain gages and with mechanical deformeter „Hugenberger“. Also the appearance and the widths of the cracks were measured in all loading and unloading cycles. The location of the measurement points is given in Figure 2.

In the same time when the full scale beams were tested, the casted specimens for the concrete, prestressed and soft steel reinforcement characteristics were also tested.

„The success of one experimental investigation depends on the quality of the registered results. Good prepared experiment is a very big thing.“

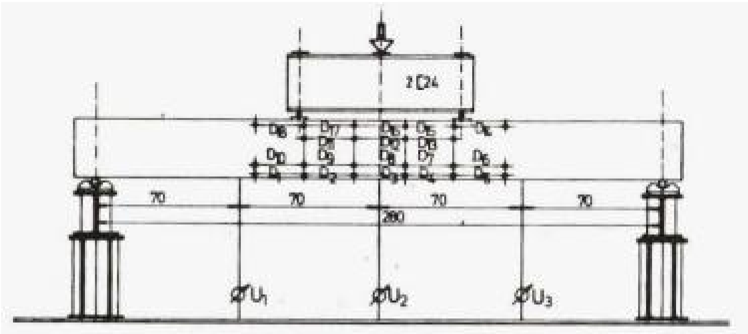

Figure 2. Location of the measurement points

2.3. Results

During testing of the beams according to the experimental program, a lot of data were registered which enabled forming of many relationships between the loading and the response of the structure, as F-d. With the analysis of the registered curves of loading and unloading, the effect of the level of prestressing, prestressing tendons and soft steel reinforcement on the bearing capacity and the deflections can also be obtained. With the choosing of the percentage of reinforcing with prestressed and soft steel reinforcement the bearing capacity and the deflections can be controlled. The experimentally obtained relationship F-d for the beams A_{32} and A_{43} are presented in Figure 3 and 4. From these relationships the energy absorption capacity of the cross sections at the moment of failure can be obtained.

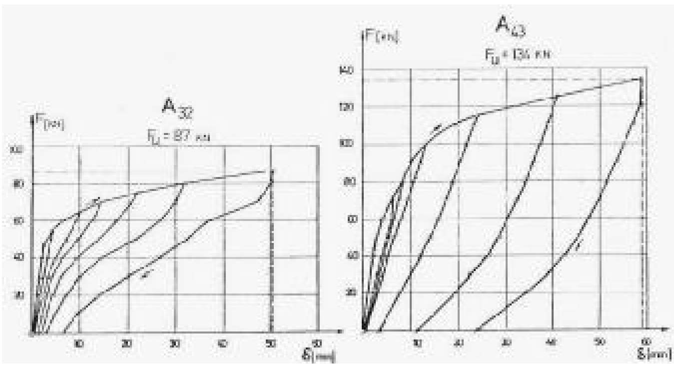

Figure 3-4. Experimentally obtained relationship F-d, for the beams A_{32} and A_{43}

The registered results from the measurements enabled forming of the following relationships: F-$\varepsilon_{s'}$, F-$\Delta\varepsilon_p$, F-ε_b.

The results from the measurements of the crack widths enabled forming of the relationship $\Delta\sigma_p$-k, as well as, the relationship between the characteristic bending moments M_r, $M_{0,1}$, $M_{0,2}$ and $M_{0,3}$ and the mechanical coefficient of reinforcing ω, where: M_r is the cracking moment, $M_{0,1}$, $M_{0,2}$ and $M_{0,3}$ are the moments at which the crack widths are 0.1, 0.2 and 0.3mm, while ω is the mechanical coefficient of reinforcing which can be calculated with the following expression:

$$w = r_p \frac{f_{yp}}{0.83 f_b} + r_s \frac{f_{ys}}{0.83 f_b} \tag{1}$$

The influence of the soft steel reinforcement on the crack widths distribution along the length of the beams A_{31} and A_{41} is presented in Figure 5 and 6.

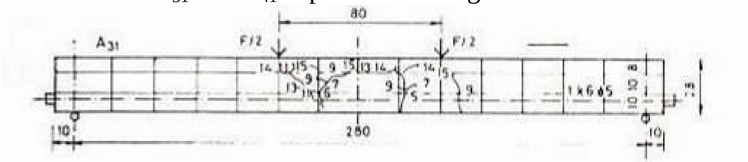

Figure 5. Crack widths distribution along the length of the beam A_{31}

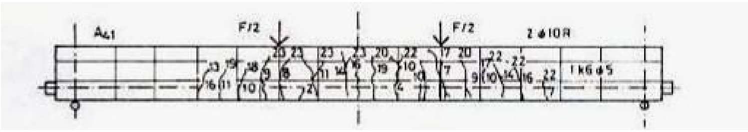

Figure 6. Crack widths distribution along the length of the beam A_{41}

The relationship M- ω for the beams from series III is presented in Figure 7. The relationship between the variations of the stresses in the prestressed reinforcement $\Delta\sigma_p$ and k, the level of prestressing, defined as $k=M_d/M_s$, where M_d is the moment of decompression, and M_s is the service moment is presented in Figure 8.

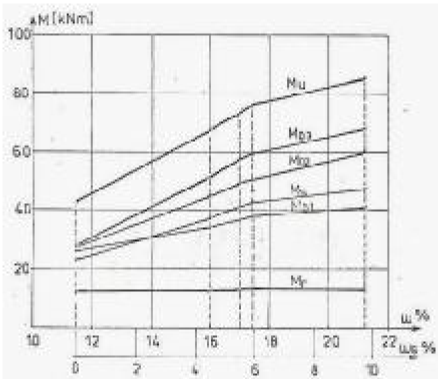

Figure 7. Relationship M- ω for the beams from series III

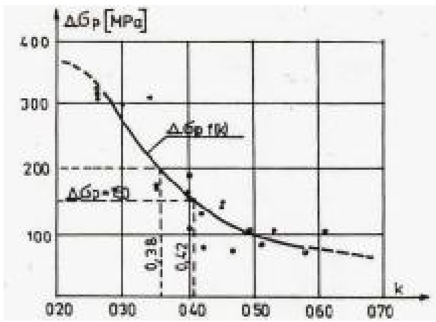

Figure 8. Relationship $\Delta\sigma_p$ - k

2.4. Conclusions

From the results obtained in this investigation, the following conclusions can be addressed:

– With the choosing of the geometrical characteristics of the cross sections and mechanical characteristics of the materials, the bearing capacity and the capacity of deflections can be controlled.

– With the choosing of the characteristics of the cross sections and the level of prestressing, the variations of the stresses in the prestressed reinforcement $\Delta\sigma_p$ and the crack widths can be controlled, which is especially important for the partially prestressed structures.

3. GORAN MARKOVSKI: INFLUENCE OF LIVE LOAD ON THE TIME-DEPENDENT PRESTRESSED CONCRETE ELEMENTS BEHAVIOUR

3.1. The aim of the research

On the basis of more numerous knowledge about the behavior of real structures during time, as on the basis of certain number of experimental researches it can be determined a conclusion that beside dead load, proper influence on the behavior of some reinforced concrete and prestressed concrete elements and structures has live-variable load during time. This is the main purpose of this research carried out at the Faculty of Civil Engineering, Skopje, Macedonia - on the base of the results of many experimental investigations and adequate theoretical analysis to give contribution on defining the influence of variable loads during the process of determining the stress-deformation state of sections and elements in connection to long-term action in concrete.

Therefore a „real" history of loading was simulating for structure like city bridges, where during the day (approximately 12 hours) live load permanently exist on the structure, and where during the night only the dead load exist.

3.2. Experimental program

Experimental testing was consisted of making, observation and testing of 24 trial elements-beams with dimension of cross section width/high/length 15/28/300cm (Fig. 9) and 171 trial specimens with different shape and size to determine mechanical and reological properties of hardened concrete.

Elements are divided in three series: A, B and C. Each series consist of four groups (1 to 4). In each group there are two fully identical beams. Beams are made of concrete grade MB-40 (f_{ck} = 40 MPa). They were prestressed at age of 28 days by one cable D15.2mm. In the tension and compresion part of the section in the beams soft (untensioned) reinforcement was embedded (Fig. 9). External loads with a a special gravitational lever were applied on beams.

In the framework of the experiment following parameters were varied:

– Ratio between dead and total load (Mg/Mq=0.55; 0.70; 0.85).

– Type of load (short-term, constant long-term, constant long-term plus cyclic variable short-term load).

Elements of the first group of each series (A_1, B_1 and C_1) were tested until they achieved the ultimate state of capacity from short-term load at age of t=40 days.

The trial beams from the second group (A_2, B_2 and C_2) were tested to the ultimate state of capacity at age of concrete t=430 days. Until the moment of testing they were subjected to no any type of load (except, of course only on prestressing force). Obtained results from the testing of this group of beams were compared with results obtained from testing of the first group of elements to determine the influence of age and reological properties of concrete in the ultimate state of capacity and in the exploitation ultimate state.

Elements from the third group of each series (A_3, B_3 and C_3) at age of t=40 days were subjected to constant dead load till the day of testing to ultimate state of capacity at age of t=430 days. This type of loading was chosen so that in the beams there were no appearance of cracks.

Elements of the fourth group of each series (A_4, B_4 and C_4) were loaded with equal dead loads as same as the beams from the third group of each series. The difference between two of them is that in the elements of this group, beside the dead load applied during the 390 days and they were subjected to certain variable load periodically (in cycles of 12 hours) with intensity different for every series. Intensity of dead and variable load, as and the prestressing force are chosen to manifested appearance of cracks in the elements of the fourth group of series A and B, until in the elements from series C there is no appearance of cracks.

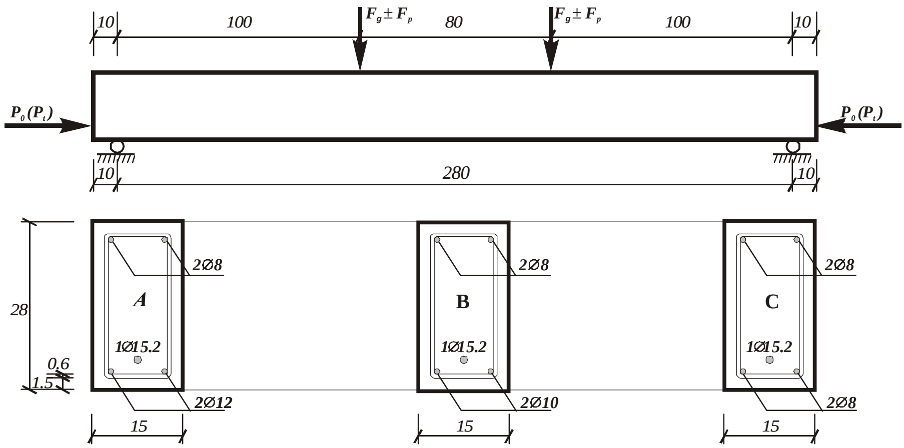

Figure 9. Trial elements - Series A; B & C

3.3. Results

During the experimental research a large number of data were obtained which generally can be divided in two groups:

Data obtained during long-term testing

Data obtained from testing elements until ultimate state capacity was achieved (at t=40 and t=430 days)

In this paper, a part of the results, obtained from long-term testing of the groups A3 and A4 of series A, will be presented. After the time period chosen for long-term testing (at the age of concrete of 430 days), beams A3 and A4 were tested until they reach the ultimate state of capacity, where the successive increment of the force has been started at the level of dead load.

Time dependent progress of the deformations in the middle of the span of beams from groups A3 and A4 are given in Figure 9, and final values are given in Table 2.

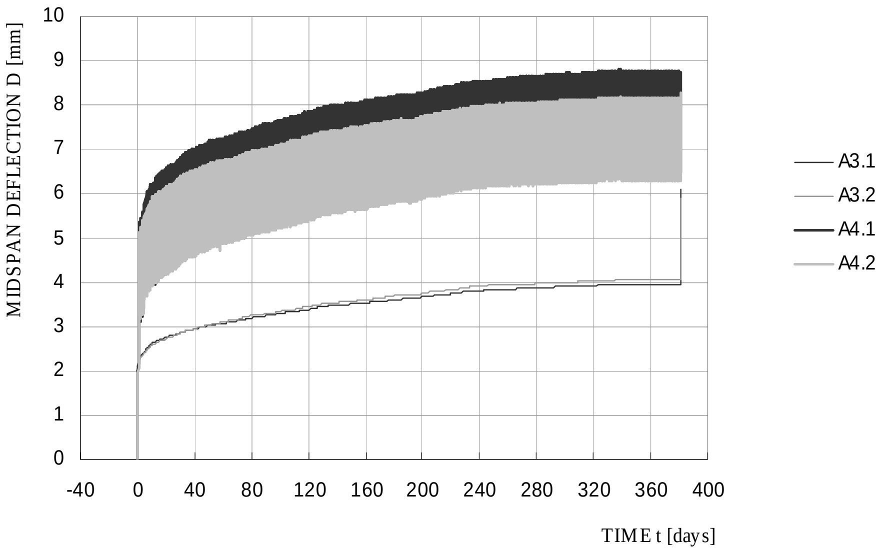

Figure 10. Trial elements A - Instantaneous and long time deflections

BEAM	A 3.1	A 3.2	A 4.1	A 4.2
$\Delta\ t_{40}$ [mm] Fg	2,11	2,08	2,01	1,95
$\Delta\ t_{430}$ [mm] Fg	3,95	4,05	6,48	6,29
$\Delta\ t_{430}$ [mm] Fg+Fp	6,11	5,89	8,75	8,27
$\Delta\ t_{430} - \Delta\ t_{40}$ [mm] Fg	1,84	1,97	4,47	4,34

Table 2. Final values of deflections of elements A

Under the influence of dead and live-cycle load the average value of the increase of time-dependent deformation of beams A4.1 and A4.2 was 131% greater than that of the beams A3.1 and A3.2. One part of this difference is because of irreversible (permanent) deformation of the beam, which was created during the first cycle of unloading the live load when in the beam for the first time a crack occur.

The modeling and the analysis of the behaviour of the prestressed beams under different loading histories during the time are carried out with the computer program Sofistik, which provides a nonlinear analysis of the beams with the finite element method. In the model only material nonlinearity is taken under consideration. The material is treated as incrementally linear and elastic (for one loading increment, it is assumed that the material behaves elastically). Among the increments, the material stiffness is corrected, so it can reflect the last change in the stresses and strains of the body. The concept of the one-axial dilatation is developed to provide a track of the degradation of the stiffness and the strength of the concrete and to provide that the actual stress-strain curve will be expressed according the one-axial dilatation.

The concrete under tension is modeled as a linear elastic material. When the maximum main stresses overcome the tension strength, a crack normal to the main stress is formed. The behaviour of the concrete is not isotropic anymore; it becomes orthotropic material with local material axes that are same as the directions of the main stresses. As a consequence on the reinforcement, it is assumed that the concrete between the cracks still takes on itself a main part of the tension stiffening, which contributes to increasing in the stiffness. The fracture energy concept is used, according which the released energy at the moment of the crack appearance in the material, G_f, is a material constant. The behaviour of the tensioned concrete after the reaching of the tension limit is in the direct relation to G_f.

The reinforcement is modeled according the smeared model. It is discretised with four levels with equal width. A one-axial stress state is in each of them. Usually it is used in the models with more layers, when the constitutive matrix is obtained by addition of the matrices of the concrete and steel layers.The standard three-linear stress-strain curve is used for the steel.

The length of the testing beams is divided into 30 finite elements, each of them long 10cm (3x10=300 cm) and divided in 20 layers (Fig.11). The reinforcement is modeled with a so-called Smeared model.

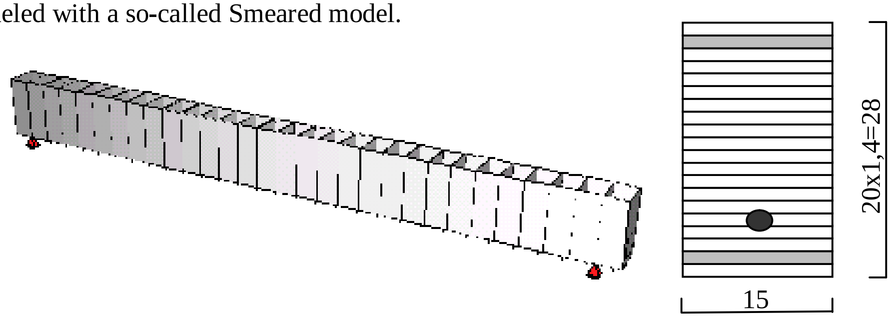

Figure 11. Discretisation of the beam in finite elements and section in layers

The axial tension strength of the concrete is assumed to be ½ of the experimentally obtained flexural tension strength (EC 2; Partial prestressing - Report of a Concrete Society Working Party). The energy at the crack's appearance G_f=0,12 N/mm, is calculated in terms of the characteristics of the used materials, according the expressions given in the Model Code MC 90. The compression crack energy G_c is a material parameter also, which can be 10-25 N/mm (adopted 20 N/mm).

In the creep analysis, the assumption for a linear relation between the creep strains and the stresses is assumed. The Age adjusted effective modulus method by Trost, actually Bazant, is used. The program provides using a step-by-step approach. Every step begins with a memorized deformation state from the previous step. For every step the appropriate creep coefficient $\Delta\varphi$ for the time interval can be introduced, as well as the shrinkage dilatation and the relaxation of the prestressing reinforcement.

The external load can be applied in unlimited number of phases – loading cases. In each of them new values for the materials can be introduced.

In the realized model analysis of the beams in the experimental program, few characteristics phases can be pointed out:

– From the moment of the releasing of the form at t=7 days up to the prestressing moment t=30 days.

– From the prestressing moment t=30 days up to the moment of the testing of the first group of elements, i.e. loading the third and fourth group t=40 days. At this age of the concrete, the compression force is applied on the prisms, which were used for the determination of the creep deformation of the concrete.

– From t=40 days up to the end of the long-term following of the elements t=430 days.

The measurement of the strains of the beams (20 points) started immediately after they were released from the form (t=7 days).

With the analysis, carried out using the program Sofistik, for every characteristic loading phase, parameters which are defining the complete behaviour of the beams are obtained: deflections, cracks and strains

All the obtained values are compared with the corresponding ones from the experimental testing. Furthermore, a part of the characteristic results will be shown.

The adopted model showed good results for the long-term deflections of the elements from the group A3. Figure 12 shows the experimental and calculated instantaneous and long time deflections at the mid-span cross section of the beams A3.

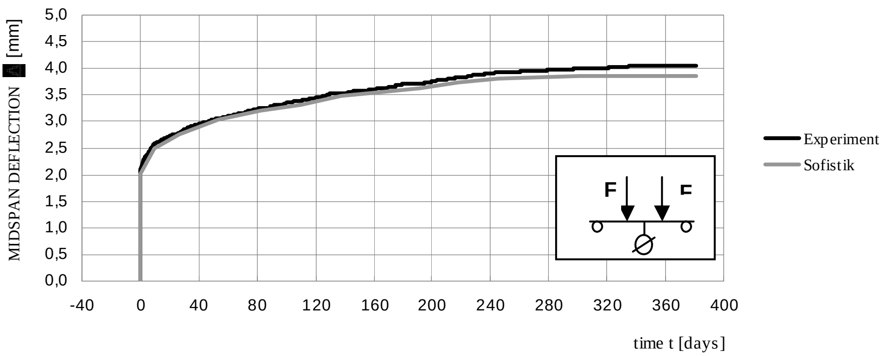

Figure 12. Instantaneous and time dependent deflection D for trial elements A3

The withdrawn of the final calculated values (t=40+390 days) from the measured is 4.90%. It is worth to be mentioned that the deflections were calculated in 57 time intervals, and each of them was started with the values for the creep coefficient and dilatations of the concrete shrinkage measured on the corresponding testing samples.

Respectively, a good agreement is obtained for the strains at the mid-span of the beams, (Fig.13).

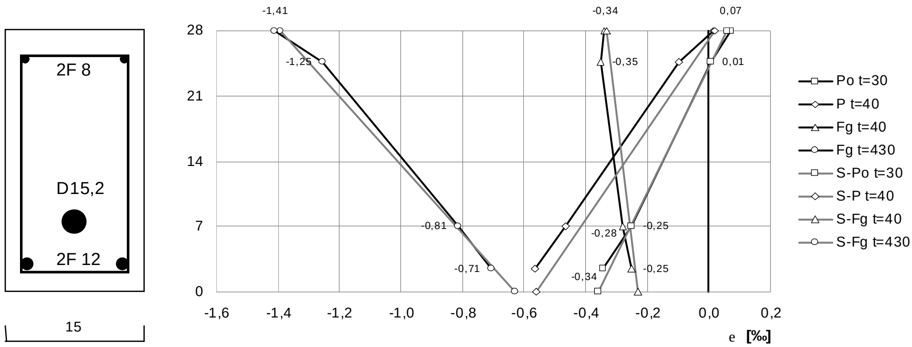

Figure 13. Instantaneous and time dependent strains (experimental and calculated with Sofistik-S) at midspan cross section for trial elements A3

As a base to define the influence of live load to behaviour of prestressed concrete elements during the time the results of executed experimental program were taken. For this purpose the next methodology was used:

– Certificate of validity of adopted accounting model through comparing theoretically and experimentally obtained results.

– A solution proposition of the problem using the previously adopted accounting model taking in calculations experimentally obtained results for the elements exposed to action of dead and live load during the time (group of elements 4).

As was mentioned above, modeling and analysis of the behaviour of prestressed concrete beams exposed to different history of loading during the time were performed with help of the computer software program SOFISTIK which enable nonlinear analysis of the beams using finite element method. Bazants'Age adjusted effective modulus method was used:

$$E_{adj,ef.}=E(t_o) / [1+\chi\varphi(t,t_o)] \quad (2)$$

To decide this problem, an approach where analysis of long-term actions uses correction of dead load was adopted, or with other words said by quasi-permanent load, which was consisted of dead and a part of the live load.

On Figure 14 experimentally determined development of the deformation in the middle of the span of the beams loaded only by dead load and the one of dead and live load (cycle in time of 12 hours) was shown. First applying of the load was done at the age of concrete t=40days. On t=430days, until now the beam loaded only by the dead load, live load was applied (jump in the diagram).

Four different approaches were examined (Figure 14), but approach 1 and 3 are taken for further consideration.

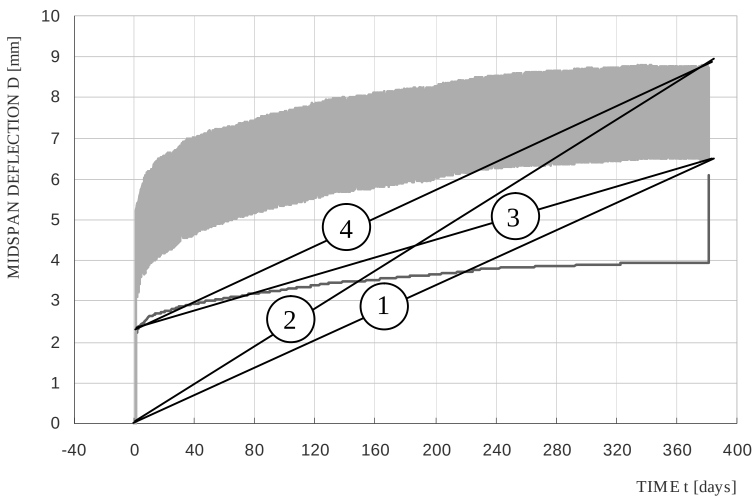

Figure 14. Four evaluated approaches

From practical view of point first approach is most suitable. According to this, with this intensity of the load (quasi-permanent) we can determined first the instantaneous deformation (of course in this case this is unreal, what is lack in this variation) and the deformation during time. In this way the procedure for determining the deformation stays identical to the usual one.

$$\Delta_{texperiment}\,(G+P_{cycl.})= \;\Delta_o\,(G+\Psi_1 P)+\,\Delta_t\,(G+\Psi_1 P) \qquad (3)$$

Approach under the number 3 was chosen because with this one, the prediction of quasi-permanent load has "smallest" participation in the total amount of the deformation. In this variation, undependably were determined the instantaneous deformation from dead load (Δt=40days) and from the live load (Δt=430days). Quasi-permanent load participate only to define the increment of the initial deformation during the time (Δt=40-430).

$$\Delta_{texperiment}\,(G+P_{cycl.})= \;\Delta_o\,(G)+\,\Delta_t\,(G+\Psi_3 P) \qquad (4)$$

On the basis of the experimentally determine values of the deformations during the time and the adopted approach to find out the level of quasi-permanent load ($G+\Psi_i P$) the following values for the coefficients Ψ_1 and Ψ_3 were obtained (Table 3).

SERIES	DEAD LOAD G [kN]	LIVE LOAD P [kN]	TOTAL LOAD G+P [kN]	COEFICIENT Y_1	Y_3	QUASI PERMANENT LOAD [kN] G+Y_1P	G+Y_3P
A	18	15,53	33,53	0,40	0,52	24,22	26,08

Table 3. Determine values of coefficients Y_1 and Y_3

3.4. Conclusions

With the defined values for the coefficient for quasi-permanent loads, a compare between the calculated and experimentally obtained values for the deflection (Fig. 15), opening of the cracks and the strains for the elements were made.

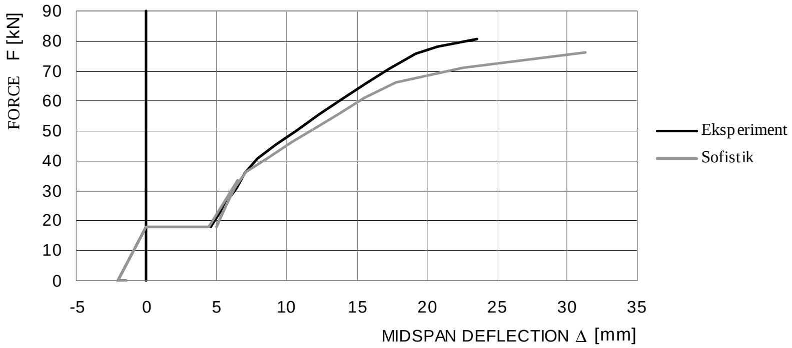

Figure 15. Force F vs. deflection D relation (experimental and calculate with Sofistik) for trial elements A4

Good matching of the results certify the validity of the adopted procedure and accounting model that give us the right the same to be proposed for this kind and similar analysis.

It can be concluded that up to the level of the force equal to half of the failure force, there is good agreement of the measured deflections at the mid-span.

The employed mathematical model gives good results for the progress of the time-dependent strains at the mid cross-section of the beams A4 (using quasi-permanent load), (Fig. 16), as well as for the dilatations for the testing up to failure at 430 days (Fig. 17).

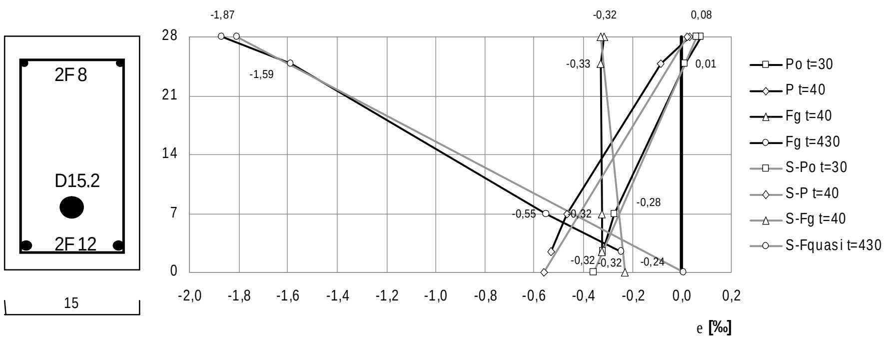

Figure 16. Instantaneous and time dependent strains at midspan cross section (experimental and calculated with Sofistik-S) for trial elements A4

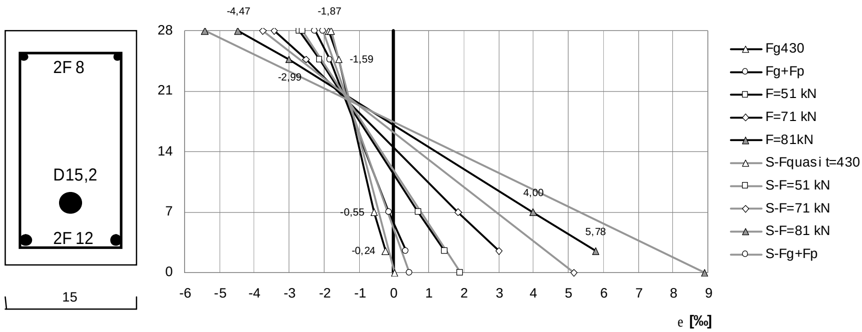

Figure 17. Strains at midspan cross section for trial elements A4 (experimental and calculated with Sofistik-S) until breaking test at t=430 days

On the basis of the obtained results from the nonlinear analysis, performed with the program Sofistik, and their comparison with the experimentally determined ones, the conclusion is that the employed model represents a good fundament for numerical definition of the behaviour of the prestressed beams during the time.

4. TONI ARANGELOVSKI: TIME-DEPENDANT BEHAVIOUR OF HIGH-STRENGTH CONCRETE ELEMENTS UNDER ACTION OF VARIABLE LOAD

4.1. Aim of the research

The long-term effects due to creep and shrinkage under repeated variable actions could overcome serviceability limit state criteria of concrete structures. According to Eurocodes EN1990-EN1999, in serviceability limit design, assessment of effects due to creep and shrinkage of concrete caused by repeated variable load are taken into consideration using quasi-permanent combination of actions (also used for reversible limit states). The level of quasi-permanent load was defined by the quasi-permanent factor ψ_2 [4].

The factor ψ_2 is defined as a nationally determined parameter and a value could be proposed in the National annex to Eurocode EN1990 [11]. This gives opportunity for research of effects of variable load and its real presentation by experimentally defined quasi-permanent factor ψ_2 for certain loading history.

Extensive experimental program and analytical research using model B3 and AAEM method was performed in order to define factor ψ_2 for two specific loading histories. In this research, besides loading histories, one parameter more, strength of concrete was included in the analysis. Two types of concrete were analyzed: ordinary concrete with a strength class C30/37 and high-strength concrete of class C60/75.

In the experimental test of reinforced concrete beams, deflections, crack width and strains in concrete and reinforcement were measured. In this paper the results from measured deflections are analyzed to propose factor ψ_2 for quasi-permanent combination of actions and to verify criteria requirements for the control of deflection in serviceability limit state.

4.2. Experimental programe

Experimental programme was proposed to analyze time-dependant behaviour of reinforced concrete elements under action of different types of loading histories. For the tests following loading histories were defined:

- Short-term load to failure at concrete age of 40 days and 400 days.
- Long-term permanent load intensity of “G” which do not cause cracking in the reinforced concrete beam applied for period of 400 days.
- Long-term load with intensity equal to the sum of permanent load “G” and variable load “Q/2” i.e. 50% participation of the variable load that acts like quasi-permanent load (ψ_2=0.50). This load “G+Q/2” was applied for period of 400 days.
- Combination of action of long-term permanent load with intensity “G” and repeated variable load “Q” which was applied in cycles of loading/unloading for 24 hours. This load “G±Q/2” was acting on the beam for period of 400 days.
- Combination of action of long-term permanent load with intensity “G” and repeated variable load “Q” which was applied in cycles of loading/unloading for 48 hours. This load “G±Q/2” was acting on the beam for period of 400 days.

The experimental program is shown in Table 4.

Series	Group	Elements	Concrete class	Type of load	Cycles of loading	Time of testing
A	A_1	2	C30/37	Short-term load	/	t=40
	A_2	2	C60/75			t=40
B	B_1	2	C30/37	Permanent load "G"	/	t=400
	B_2	2	C60/75			t=400
C	C_1	2	C30/37	Permanent load "G+Q/2"	/	t=400
	C_2	2	C60/75			t=400
D	D_1	2	C30/37	Permanent load "G" and Variable load "Q"	Loading/ unloading Δ_{t1}=24 hours	t=400
	D_2	2	C60/75			t=400
E	E_1	2	C30/37	Permanent load "G" and Variable load "Q"	Loading/ unloading Δ_{t2}=48 hours	t=400
	E_2	2	C60/75			t=400
F	F_1	2	C30/37	Short-term load	/	t=400
	F_2	2	C60/75			t=400

Table 4. Experimental program

In the experimental program 24 reinforced concrete beams were made. The dimensions of the reinforced concrete beams and the position of the reinforcement are shown on Figure 18.

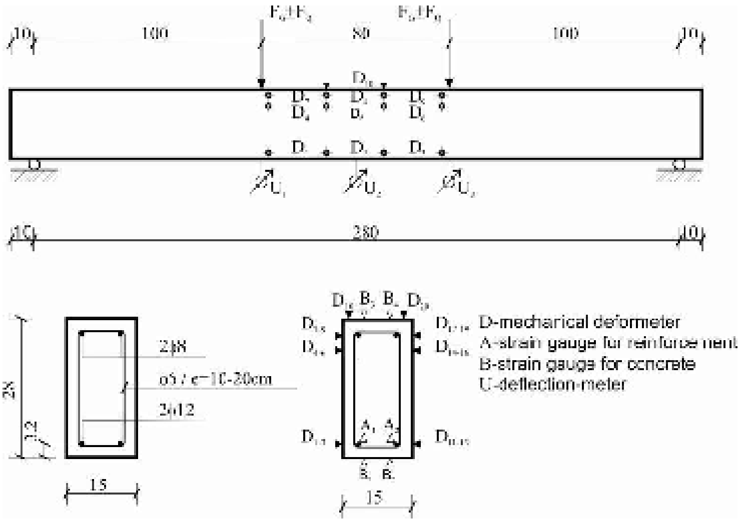

Figure 18. Reinforced concrete beams for testing and position of the measuring points

4.3. Results

Influence of the permanent action and repeated variable actions on long-term behavior of reinforced concrete beams was analyzed measuring the development of long-term deflections in period of 400 days. Experimental results for the deflections within the beams series "D" and "E" are given in Figures 22, 23, 24, 25 and 26 by the relations of deflection (a)- time (t) together with the results from the analytical analysis.

For analytical analysis of the experiment data first evaluation of autogenous shrinkage, drying shrinkage and creep strains were performed by using B3 Model (Bazant & Baweja. 2000) [4]. This analysis provide data to define improved B3 model compliance function J(t,t'), aging coefficient $\chi(t,t')$ and relaxation function R(t,t'). Previously mentioned parameters were used to calculate and to verify long-term deflections by age adjusted effective modulus method (AAEM method). To calculate effects from variable action on long-term deflections of concrete elements quasi-permanent load procedure and the principle of superposition were used.

The simplest solution of the problem was taking into consideration one load, as sum of permanent and quasi-permanent load, from which we can obtain initial and time-dependant deflection. This solution can be written with the following equation [5]:

$$a_{t,exp}(G+Q) = a_0(G+\psi_2 Q) + a_t(G+\psi_2 Q) \tag{5}$$

Where: $a_{t,exp}(G+Q)$ - experimentally obtained deflection under the effect of long-term permanent load G and repeated variable load Q; $a_0(G+\psi_2Q)$ - analytically obtained initial deflection under the effect of long-term permanent load G and quasi-permanent load ψ_2Q; $a_t(G+\psi_2Q)$ - analytically obtained time-dependant deflection under the effect of long-term permanent load G and quasi-permanent load ψ_2Q, as a part of the variable load.

This solution has one imperfection in calculation of initial deflection, which for the real loading history used in the experiment, is not correct.

Analysis using B3 model shows that a good agreement is achieved between the experimental and analytical results especially for the creep compliance. Better results were obtained by using improved B3 model calculating updated parameters p_1 and p_2. For concrete class C30/37, on the basis of linear regression, following values were obtained p_1=0.9088 and p_2=0.998 to adjust the creep compliance. Updating of shrinkage predictions was done only by using scaling parameter which for ordinary concrete C30/37 was p_6=0.82. For high strength concrete C60/75, on the basis of linear regression, p_1=1.4264 and p_2=0.3319 were obtained to adjust the creep compliance in B3 model (Figure 19) and p_6=1.0955 to adjust drying shrinkage (Figure 20). Analysis of autogenous shrinkage is given on Figure 21.

Analysis of autogenous shrinkage using B3 model shows that proposed formula for autogenous shrinkage predictions is not valid for use neither for ordinary concrete and neither for high-strength concrete. In the analysis of experimental data it was tried to use the original formula for calculation of autogenous shrinkage by the model B3 [2] (eq. 6):

$$\varepsilon_a(t) = \varepsilon_{a\infty}(0.99 - h_{a\infty}) \cdot S_a(t) \tag{6}$$

Where: $\varepsilon_a(t)$ is autogenous shrinkage, $\varepsilon_{a\infty}$ final value of autogenous shrinkage, $h_{a\infty}$ is the final self-desiccation humidity (it may be assumed to be 80%) and $S_a(t)$ is time dependence of ultimate autogenous shrinkage.

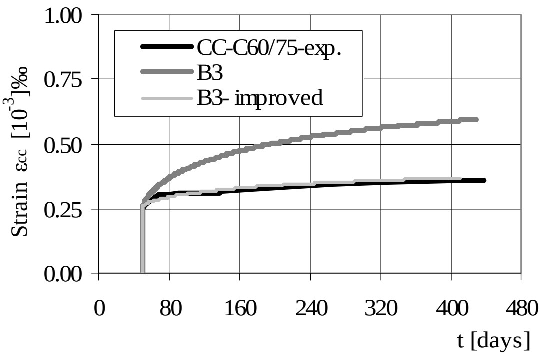

Figure 19. Diagram creep of concrete-time ε_{cc}-t for class C60/75: experimental results, B3-model and B3-model improved

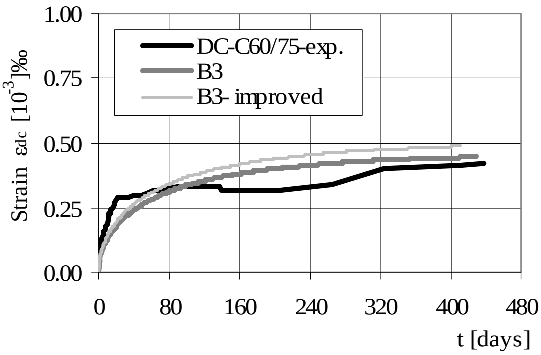

Figure 20. Diagram creep of concrete-time ε_{dc}-t for class C60/75: experimental results, B3-model and B3-model improved

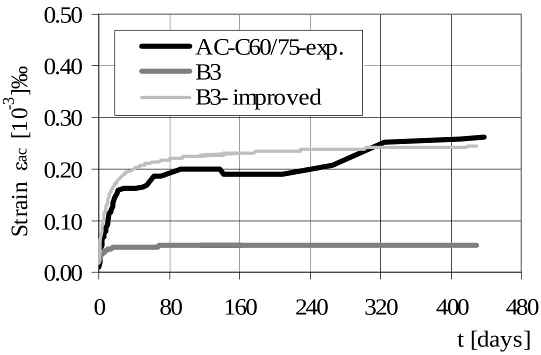

Figure 21. Diagram creep of concrete-time ε_{ac}-t for class C60/75: experimental results, B3-model and B3-model improved

During the calculation a convergence problem arises when for $t \to \infty$ $\varepsilon_a(t \to \infty) = \varepsilon_{a\infty}(0.99 - h_{a\infty})$, because $S_a(t) \to 1$. If $h_{a\infty} = 80\%$, in that case we have $\varepsilon_a(t \to \infty) = 0.19 \cdot \varepsilon_{a\infty}$ or with other words it doesn't converge in $\varepsilon_a(t \to \infty) = \varepsilon_{a\infty}$.

Analytical analysis of the autogenous shrinkage was done in accordance to updated formula based on experimental results:

$$\varepsilon_a(t) = \varepsilon_{a\infty}(0.99 - h_{a\infty})^{\frac{1}{\sqrt{t-t_s}}} \cdot S_a(t) \tag{7}$$

Using AAEM method, effects from variable actions were calculated to obtain the long-term deflection using quasi-permanent combinations of actions defining factor ψ_2. Results are given on the basis of experimental data and analytical results by AAEM method for the development of deflections during time using data from previous analysis by the B3 model and the improved B3 model.

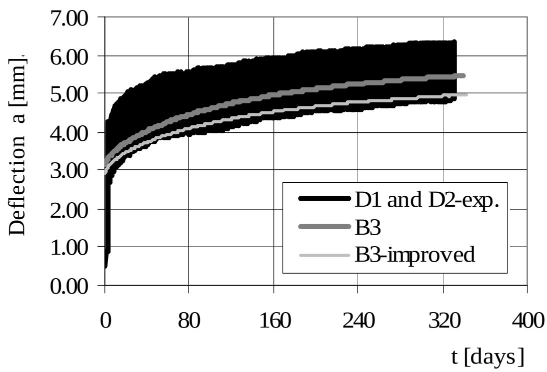

Figure 22. Diagram deflection-time a-t for series D beams concrete class C30/37

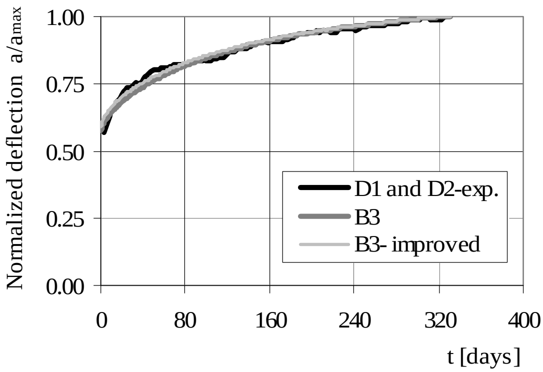

Figure 23. Diagram normalized deflection-time a/a_{max}-t for series D beams concrete class C30/37

According to the analysis presented within the figures 22, 23 and 24 for series D beams best results were obtained when quasi-permanent coefficient was set by the factor ψ_2=0.49 for concrete beams class C30/37 and ψ_2=1.0 for concrete beams class C60/75. This means that, in the specific beam set tested, approximately 50% of the variable load should be taken as quasi-permanent load for ordinary concrete beams and full value of variable loads for high-strength beams.

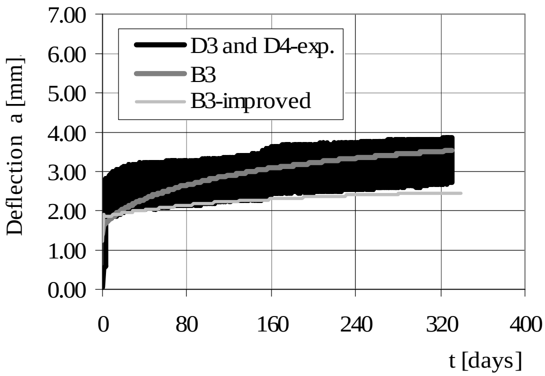

Figure 24. Diagram deflection-time a-t for series D beams concrete class C60/75

For series "E" beams according to the diagrams presented in Figures 25 and 26 best results were obtained when the quasi-permanent coefficient was set to ψ_2=0.66.

In the series “D” and “E” with beams made of high-strength concrete, the analysis of the experimental results and the results obtained with the analytical AAEM method analysis shows that for the determination of the coefficients ψ_2 the stress state should be included in the analysis when cracks appears.

The chosen loading histories for these series of high-strength concrete beams were determined in order to compare the behavior with reinforced concrete beams made of ordinary concrete.

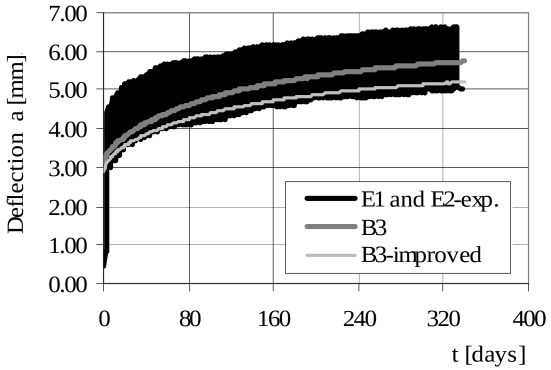

Figure 25. Diagram deflection-time a-t for series E beams concrete class C30/37

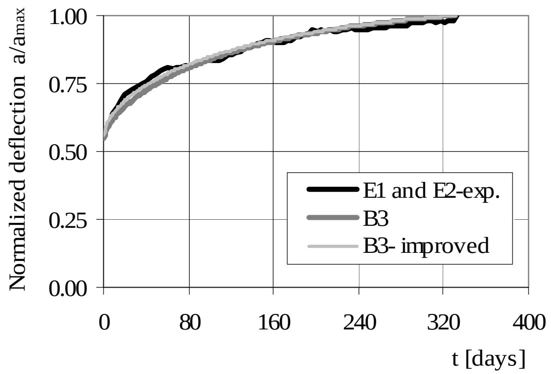

Figure 26. Diagram normalized deflection-time a/a_{max}-t for series E beams concrete class C30/37

But higher mechanical properties of high-strength concrete enable cracks appearance practically at the level of service load. At this level due to the approximately equal cracking moment M_{cr}=11.6kNm and moment of the service loads M_s=12.6kNm, a stabilized state of cracks was formed. Thus, the coefficient of participation for defining of the variable load as a quasi-permanent action is approaching to $\psi_2=1$ in the analytical solution in order to obtain the experimentally measured deflections. However, this does not coincide with usual practical cases. In this way replacing variable load by permanent load to calculate time-dependant deflections for elements made of high-strength concrete wrong solutions are obtained.

For the elements of high-strength concrete it is necessary to define higher values of the stresses, i.e. state of stress that causes cracks from the action of load less than the value of service load. This is a precondition for proper analysis of variable loads effect on the behavior of the reinforced high-strength concrete elements to obtained values for the quasi-permanent coefficient ψ_2. Generally, a decision must be made between cracked and uncracked sections.

4.4. Conclusions

From the experimental and analytical analysis of time-dependant behavior of reinforced concrete beams, made of ordinary and high-strength concrete, subjected to long-term permanent load and repeated variable load following conclusions can be received:

1. Actions of long-term permanent load and repeated variable load have significant influence on the time-dependant behavior of concrete beams made of ordinary and high-strength concrete.
2. Time-dependant behavior of concrete elements under action of long-term permanent load and repeated variable load depends from loading history, i.e. especially from the cycles of loading/unloading by variable load (Δt_1=24h; Δt_2=48h).
3. Analytical analysis of drying shrinkage and creep using B3 model shows good agreement with experimental results using update parameters for improved estimations. On the basis of regression analysis, for ordinary concrete following values of update parameters p_1 and p_2 were obtained: $p_1=0.9088$ and $p_2=0.998$, to adjust the creep compliance. For the improved estimation of the drying shrinkage coefficient p_6 is set to $p_6=0.82$. These values for high-strength concrete are $p_1=1.4264$, $p_2=0.3319$ and $p_6=1.0955$ to adjust creep compliance and drying shrinkage strain.
4. Analysis of autogenous shrinkage strain according to B3 model gives incorrect estimations of strains and that's why new formula was proposed to obtain analytical solution using experimental results:

$$\varepsilon_a(t) = \varepsilon_{a\infty}(0.99 - h_{a\infty})^{\frac{1}{\sqrt{t-t_s}}} \cdot S_a(t) \tag{8}$$

5. Using Age Adjusted Effective Modulus Method and principle of superposition analysis quasi-permanent coefficients ψ_2 were obtained to determine influence of variable load on the time-dependant behavior of concrete for control of the deflections. According to experimental and analytical results, repeated variable load was replaced by quasi-permanent load defining it through the quasi-permanent coefficient ψ_2. For the concrete class C30/37 and considered loading histories by repeated variable loads coefficient $\psi_2=0.50$ was obtained for cycles of loading/unloading Δt=24h. Coefficient $\psi_2=0.65$ was obtained for cycles of loading/unloading Δt=48h for beams made of ordinary concrete class C30/37. For the same loading histories at high strength concrete beams coefficient $\psi_2=1$ was obtained.

5. DARKO NAKOV: TIME-DEPENDANT BEHAVIOUR OF STEEL FIBRE REINFORCED CONCRETE ELEMENTS UNDER SUSTAINED LOADS

5.1. The aim of the research

One of the materials that were used from the beginning of the last century, but still has not found its place in practice is steel fibre reinforced concrete. There is no doubt about the positive influence of the steel fibre reinforcement on some of the properties of concrete. One of the main improvements is the decreasing of crack widths. With this improvement, durability of the concrete elements and structures is increased, which is one of the main goals in the current research in up to date civil engineering.

The crack widths and their further opening are in direct correlation with the long term effects in the concrete, creep and shrinkage. The experimental investigations in this area are still rare, so every contribution in increasing the data base in this area is a big step further. With the proposed experimental program, detailed investigations in both areas and their correlation are planned.

The aim of this still ongoing research is to contribute in creation of data base which will help in definition of computational model that presents the influence of creep and shrinkage on the behavior of steel fibre reinforced concrete elements. As it was expected, smaller creep strains were obtained due to the addition of fibers to the concrete matrix, which already contains aggregate as a restraint. In this area improvement of the newest codes is predicted so that they can be adjusted and used for this material also. At the end of the research a value for the coefficient ψ_2, which is used for part of the variable load to be added as a permanent load, for steel fibre reinforced concrete will be proposed.

5.2. Experimental program

Because of the importance of the experimental results in this type of investigations, an experimental program was planned. It consists of 24 full scale beams from reinforced concrete and steel fibre reinforced concrete with additional reinforcement. The beams have cross section dimensions 15/28cm and total length l=300cm, Figure 27. Together with each series of beams, control specimens were cast, in order to test the compressive strength, flexural tensile strength, splitting tensile strength, elastic modulus and the deformations due to creep and shrinkage. Beside the concrete mechanical and time dependant characteristics, the used reinforcement was also tested.

All 24 beams were manufactured with concrete class C30/37, and according to the type of material they were divided in three series:

– Series A, reinforced concrete (RC);

– Series B, SFRC with 30 kg/m^3 steel fibres and additional reinforcement (SFRC1);

– Series C, SFRC with 60 kg/m^3 steel fibres and additional reinforcement (SFRC2).

In order to find out the influence of the different fibre dosages on the behavior of the elements with time, the investigated parameter is the fibre dosage. Steel fibres were hooked-end HE1/50, Arcelor Mittal with diameter 1mm and length of 50mm.

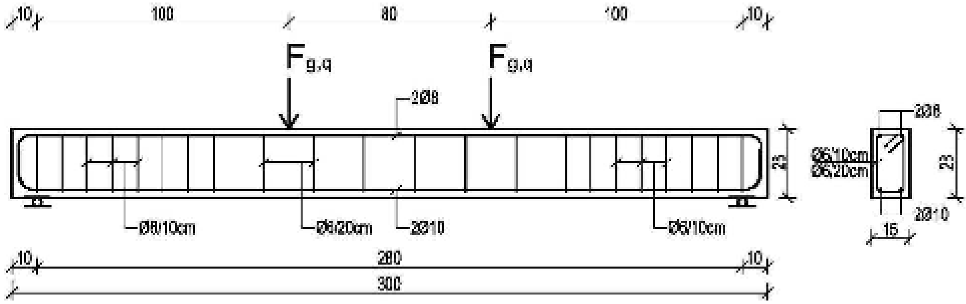

Figure 27. Geometry, reinforcement and loading scheme of the full scale beams

Regarding the loading history the beams were divided in four groups:

1. The beams from all three series from group "1" (A_1, B_1, C_1) have been tested under short term ultimate load at the age of concrete of 40 days. With this testing, relevant dependences should be found out for this age of concrete and the behavior of the reinforced concrete and two types of steel fiber reinforced concretes should be compared.

2. The beams from all three series from group "2" (A_2, B_2, C_2) will be tested also under short term ultimate load, but at the age of concrete of 400 days. This will be done in order to find out the influence of the age of concrete on the behavior of the beams.

3. The beams from group "3" (A_3, B_3, C_3) have been pre-cracked with permanent and variable load "g + q", and afterwards a long term permanent load with intensity "g" was applied on the age of concrete of 40 days, to be held up to 400 days, when a short term ultimate load testing will be performed. In the meantime the strains, deformations and crack widths will be measured.

4. On the beams from group "4" (A_4, B_4, C_4) long term permanent load with intensity "g" has been applied on the age of concrete of 40 days, to be held up for one year as a long term load. On the fortieth day, variable cyclic repeated load “± q” was also applied in an interval of 8 hours +q and 16 hours –q, for one year. It means that for 8 hours every day the beams will be loaded additionally with load “q” and the strains, deformations and crack widths will be measured. After 8 hours the beams will be unloaded from the load “q” and all the measurements will be performed again. This will be repeated every day for one year in order to simulate a realistic load history.

The mixture proportioning was done so that is the same for the three types of concrete, except the aamount of the fibres. The experimental program is presented in detail in Table 5. The long term load, which consists of permanent sustained load “g” and variable repeated load “q”, was applied by gravitation lever, which enabled an increase of the load for 13 times. The permanent load acts all the time, while the variable load was applied and removed each day by secondary hand gravitation lever. The bending moments are as follows: from self weight of the beam, M_{sw}=1kNm, from permanent load “g”, M_g=5.0kNm, from variable load “q”, M_q=3.1kNm, from self weight, permanent and variable load (service) M_{sw+g+q}=9.1kNm. The bending crack moment was M_{cr}=6.1kNm, while the ultimate bending moment M_d=15.6kNm. The intensity of the load was chosen so that the M_{cr} is bigger than M_{sw+g} and smaller than M_{sw+g+q}. The permanent load is 0.39 times the flexural strength, while the service load is 0.58 times the flexural strength of the beam without fibres.

Series	Group	Number of elements	Type of concrete (C30/37)	Steel fibres (kg/m^3)	Tensile Reinforcement μ (%)	Type of long term load	Time of ultimate load testing	Time of observing of the elements
A	1	2	RC	0	0.37	/	t=28	
	2	2	RC	0	0.37	/	t=428	t =400
	3	2	RC	0	0.37	"g"	t=428	t =400
	4	2	RC	0	0.37	"g ±q"(Δ_{tl}=8h)	t=428	t =400
B	1	2	SFRC 1	30	0.37	/	t=28	
	2	2	SFRC 1	30	0.37	/	t=428	t =400
	3	2	SFRC 1	30	0.37	"g"	t=428	t =400
	4	2	SFRC 1	30	0.37	"g ±q"(Δ_{tl}=8h)	t=428	t =400
C	1	2	SFRC 2	60	0.37	/	t=28	
	2	2	SFRC 2	60	0.37	/	t=428	t =400
	3	2	SFRC 2	60	0.37	"g"	t=428	t =400
	4	2	SFRC 2	60	0.37	"g ±q"(Δ_{tl}=8h)	t=428	t =400

Table 5. Experimental program

5.3. Results

The mixture proportioning was done according to all the recommendations, [9], [16] and [14] in the up to date literature, so the slump of the concrete without fibres was 120mm. Since the fibres are decreasing the workability, the slump decreased to 75mm and 50mm with addition of 30 and 60 kg/m^3.

Mechanical properties at the age of 40 days were tested on 3 specimens for the compressive strength, splitting tensile strength and Modulus of Elasticity and 6 specimens for the flexural tensile strength, according to RILEM TC 162-TDF, [19].

Since the experiment is still ongoing, only the results from the first 150 days for the drying shrinkage and creep are presented in this paper (Figure 28 and Figure 29).

The measured drying shrinkage strains after this time period are: 664[10^{-6}]μs for RC, 650[10^{-6}]μs for SFRC1 and 662[10^{-6}]μs for SFRC2.

The measured instantenous strains are: 302[10^{-6}]μs for RC, 261[10^{-6}]μs for SFRC1 and 247[10^{-6}]μs for SFRC2, while the creep strains after this time period are: 561[10^{-6}]μs for RC, 529[10^{-6}]μs for SFRC1 and 493[10^{-6}]μs for SFRC2.

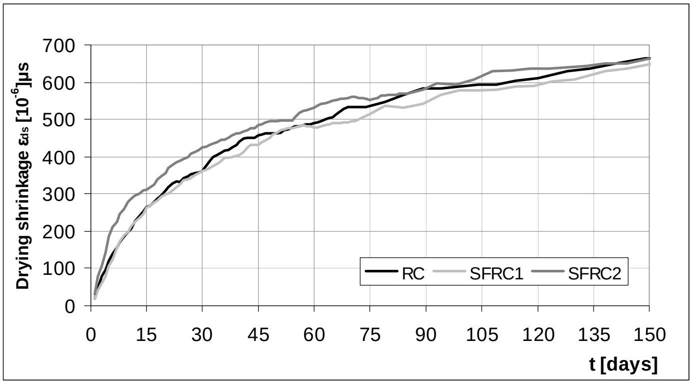

Figure 28. Drying shrinkage strain in the first 150 days for RC, SFRC1 and SFRC2

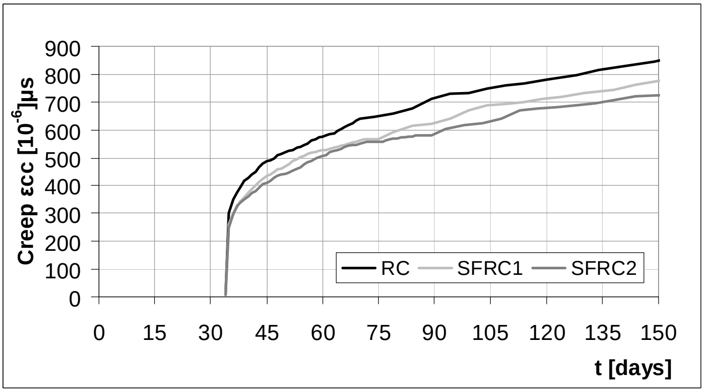

Figure 29. Creep strain in the first 150 days for RC, SFRC1 and SFRC2

The time dependant deflections, as well as the time dependant crack widths for the beams from group 3 and 4, on which specific loading history is applied, are presented for the first 100 days in Figure 30 to Figure 33.

The instantenous deflections at the load level of “g” for the beams from group 3 are: 1.94mm for RC, 1.56mm for SFRC1 and 1.31mm for SFRC2, while the deflections after 100 days of loading at the same load level are: 2.99mm for RC, 2.46mm for SFRC1 and 2.16mm for SFRC2.

Time dependant crack widths are presented for load level “g” and the values for the beams from group 3 are: 0.12mm for RC, 0.11mm for SFRC1 and 0.07mm for SFRC2.

The instantenous deflections at the load level of “g+q” for the beams from group 4 are: 1.91mm for RC, 1.77mm for SFRC1 and 1.48mm for SFRC2, while the deflections after 100 days of loading at the same load level are: 4.05mm for RC, 3.39mm for SFRC1 and 2.96mm for SFRC2.

Time dependant crack widths are presented for both load levels “g” and “g+q” and the values at the load level “g+q” for the beams from group 4 are: 0.17mm for RC, 0.13mm for SFRC1 and 0.07mm for SFRC2.

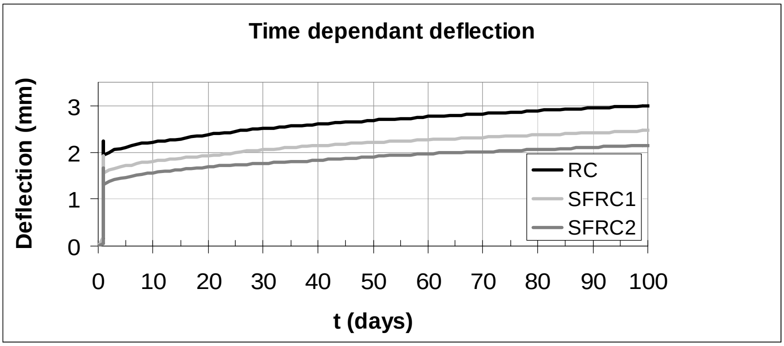

Figure 30. Time dependant deflection for the beams from group 3 for RC, SFRC1 and SFRC2

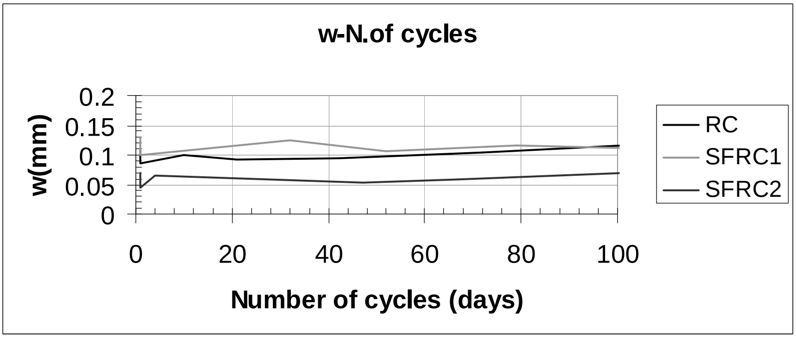

Figure 31. Time dependant crack widths for the beams from group 3 for RC, SFRC1 and SFRC2

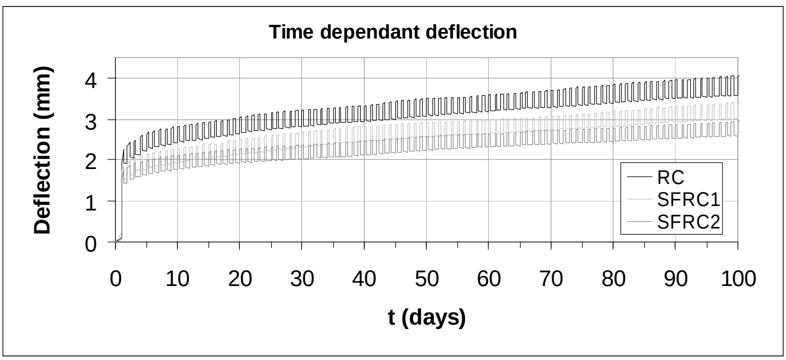

Figure 32. Time dependant deflection for the beams from group 4 for RC, SFRC1 and SFRC2

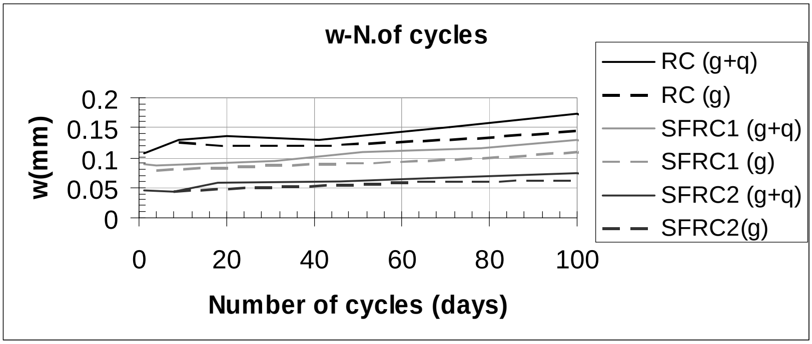

Figure 33. Time dependant crack widths for the beams from group 4 for RC, SFRC1 and SFRC2

5.4. Conclusions

In order to obtain the mechanical characteristics of the three different concretes, tests were done and the results show that with addition of 30kg/m^3 fibres there is no significant improvement, except for an increase in toughness. With 60 kg/m^3 fibres, the compressive strength was already increased for 4%, the splitting tensile strength for 14% and flexural tensile strength for 2.3%. On the other hand, the slump decreased from 120mm of RC to 75mm of SFRC1 and 50mm of SFRC2.

After measuring of the drying shrinkage strains in the first 150 days, it can be noticed that the addition of 30kg/m^3 or 60 kg/m^3 steel fibres does not have any influence on the drying shrinkage. On the other hand, the addition of 30kg/m^3 steel fibres reduced the creep strain for 5.7%, while the addition of 60 kg/m^3 reduced the total creep strain for 12.1% when compared to ordinary concrete.

After obtaining the final results from the drying shrinkage and creep, based on the Theory of the creep of steel fibre cement matrices under compression, developed by Mangat and Azari [15], it is expected that a certain improvement of the viscous (flow) compliance in the Model B3 will be made, which is Creep and shrinkage prediction model for an analysis and design of concrete structures, developed by Prof. Bazant.

The time dependant deflections on the beams from group 3 after 100 cycles of loading are for 17.7% smaller for SFRC1, while 27.8% smaller for SFRC2 when compared to RC. The time dependant crack widths on the beams from group 3 after 100 cycles of loading are 8.3% smaller for SFRC1, while 41.7% smaller for SFRC2 when compared to RC.

The time dependant deflections on the beams from group 4 after 100 cycles of loading are for 16.3% smaller for SFRC1, while 26.9% smaller for SFRC2 when compared to RC. The biggest improvement up to now, was achieved at the time dependant crack widths on the beams from group 4, which are 23.5% smaller for SFRC1, while 58.8% smaller for SFRC2 when compared to RC.

At the end of the experimental program, using the AAEM method, a certain value for the coefficient ψ_2 for Steel fibre reinforced concrete will be proposed, defining the quasi permanent value of variable load according to Eurocode 2, for structures like warehouses, parking garages or bridge girders.

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SOCIETY FOR MATERIALS AND STRUCTURES TESTING OF SERBIA
DRUŠTVO ZA ISPITIVANJE I ISTRAŽIVANJE MATERIJALA I KONSTRUKCIJA SRBIJE

International Conference "**Marking 60 Years of Operation of DIMK**
Belgrade, October 19, 2012.

Azra Kurtović[1] UDK:691.21.059.4

DURABILITY AND DECOMPOSITION OF NATURAL STONE

Summary: *Although it is disregarded as the construction material, nowadays the need arises more and more to interpret the natural stone destruction forms and its time-related durability increase. In the paper, the overview is given of the accepted descriptions of natural stone destruction forms as well as the research results for the existing stone of the Hellenistic urban complex on the ruins of the fort of Ošanići near Stolac – Bosnia and Herzegovina.*

In order to enjoy centuries-old monumental beauty of the natural stone, it is necessary to stick to the new guidelines for the conduct of preliminary and control tests according to CPD (Construction Products Directive).

Key words: *decomposition forms, durability, natural stone properties*

TRAJNOST I RAZGRADNJA PRIRODNOG KAMENA

Rezime: *Premda je zanemaren kao konstruktivni materijal, danas se sve više javlja potreba za tumačenjem oblika razaranja prirodnog kamena i povećanja njegove vremenske trajnosti. U radu je dat prikaz prihvaćenih opisa oblika razgradnje prirodnog kamena kao i rezultati istraživanja postojećeg kamena Helenističkog urbanog kompleksa na gradini Ošanići kod Stoca-BiH.*

Da bismo uživali u vjekovnoj monumentalnoj ljepoti prirodnog kamena neophodno je pridržavati se novih smjernica za provođenje prethodnog i kontrolnog ispitivanja prema CPD (Direktiva o građevinskim proizvodima).

Ključne riječi: *oblici razgradnje, trajnost, svojstva prirodnog kamena*

[1] Ph.D., Faculty of Civil Engineering Sarajevo, Ass. professor

1. INTRODUCTION

The time-related durability of the stone is the resistance offered by the stone against the physical and chemical influence of the environment in which it is, and in which process of the time-related course of destruction depends on the petrographic nature of stone and on the nature of the influences it is exposed to.

It does not matter if they are of chemical or physical nature, the destruction processes can be very fast and noticeable in the time period of the total of several years and even in shorter periods.

The main cause of the stone decomposition is the water (damp in stone, humidity in atmosphere) which is responsible for the chemical and physical degradation. All the basic components of the stone are soluble in the water; it happens in some of them after relatively short exposure to water influence, and in some of them after geologically long time period.

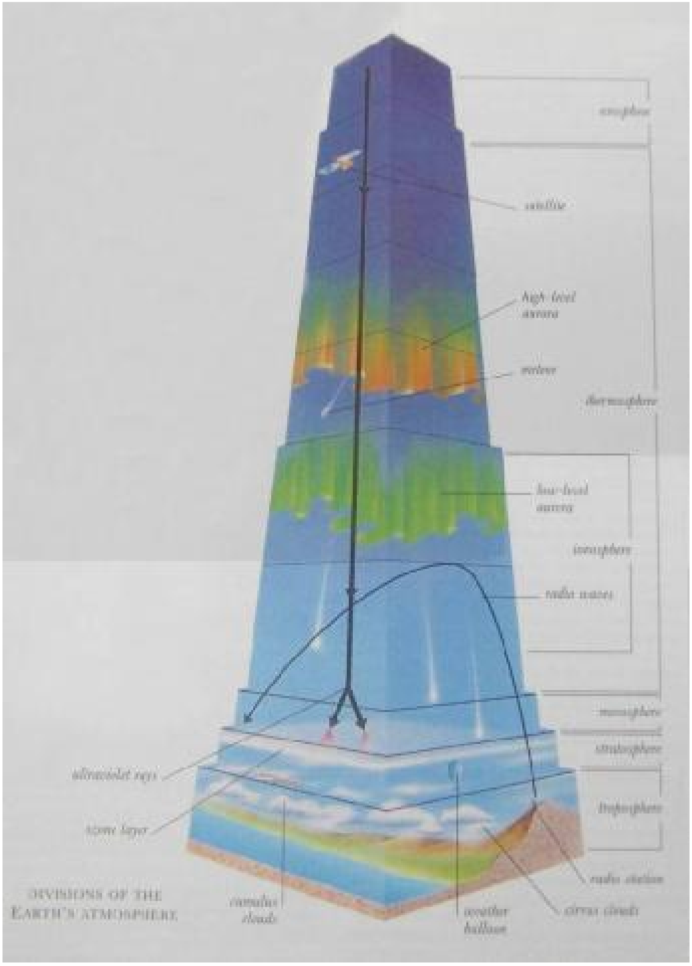
The climatic and microclimatic differences are the main factor of the different behavior of same stone in various milieus (natural exogenous factors – sun, frost, rain, water, wind and urban complex aggressive products – exhaust gases, nitric, carbonic and other acid agents).

Knowing the facts relating to the durability of stone is the precondition for the evaluation of its application and way of dressing.

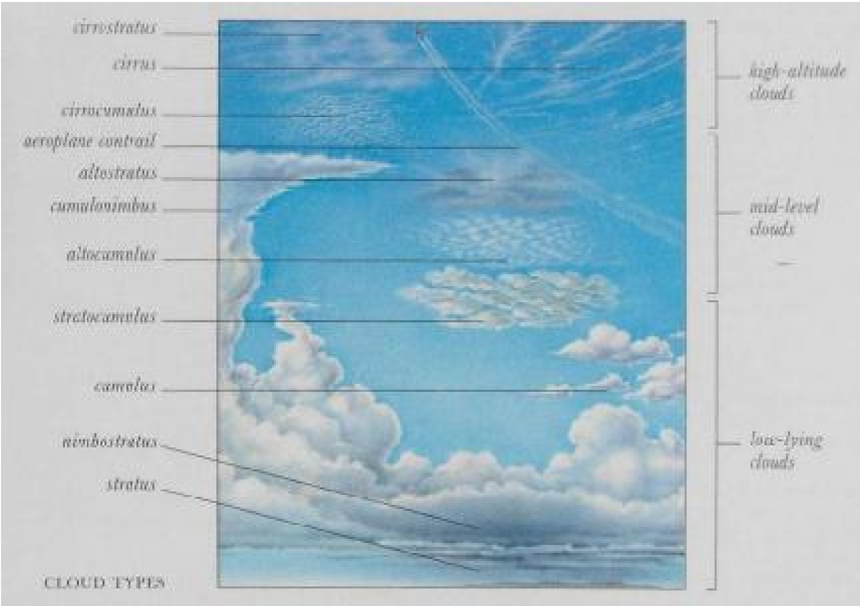
At the current degree of the laboratory technique development, there is no absolutely accepted test method which completely reproduces the natural conditions in which the architectural-constructional stone can be found in the free atmosphere.

The physical destruction caused by the frost effects is the most prominent factor of stone physical destruction in the areas with humid climate. This influence can be such that the stone, which is characterized by the pores of certain size and their mutual connection, in the conditions of being filled with water, is destructed in a very short time period (during one winter).

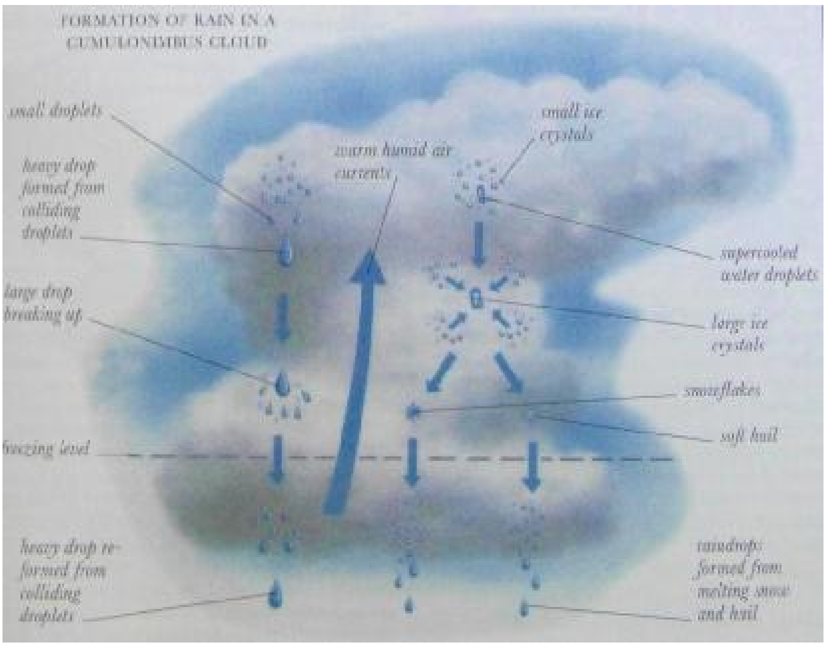
For the evaluation of the susceptibility to the frost, nowadays there are used the direct methods (freezing test, crystallization test) where physical destruction developments are registered and which are manifested in the form of fragmenting or attrition and the indirect methods (establishing weakening of internal connection between components by measuring fall of firmness due to pressure, elasticity module and so on after conducted freezing test or crystallization test), and which are applied in cases when no visual manifestations of destruction exist.



Picture 1: devisions of Earth's atmosphere



Picture 2: Cloud types



Picture 3: formation of rain in cumulonimbus cloud

2. FORMS OF NATURAL STONE DESTRUCTION

A huge problem in defining the destruction forms is to find generally accepted, corresponding expressions when describing them. Various descriptions of decomposition forms can be found in the literature.

In studies on stone deterioration and conservation, terminological confusions lead to major communication problems between scientists, conservators and practitioners. In this context, it is of primary importance to set up a common language; if degradation patterns can be shown, named and described, then they can be recognised and compared with similar ones in a more accurate way in further investigations.

The ICOMOS (International Council on Monuments and Sities) International Scientific Committee for stone (ISCS) is providing a forum for the interchange of experience, ideas, and knowledge in field of stone conservation. ISCS aims at facilitating the publication, dissemination and presentation of state of the art reviews on pre-identified issues.

The ISCS glossary constitutes an important tool for scientific discussions on decay phenomena and processes.

Seven documents were identified as a basis for collecting and combining useful terms into a generalised glossary:

1. Unpublished list of 21 terms written by A.Arnold, D.Jeannette and K.Zenhder (1980)

2. Compilation of 24 English terms with related definitions published by Grimmer (1984) of the U.S.National Park Service

3. Italian Standard NORMAL 1/88 (NOrmativa MAnufatti Lapidei) published in 1990 . This glossary is illustrated by photographs. Proposal for a terminology of stone decay forms on monuments, written by Jose Delgado Rodrigues from LNEC (Lisbon, Portugal), was largely inspired in internal documents produced in the framework of the Petrography Group of the ICOMOS Stone Committee and published in its newsletter in 1991.

4. This proposal was used as a basis for the publication by LNEC 2004, of a glossary with short definitions in Portuguese language, including terms related to stone, masonry and render deterioration.

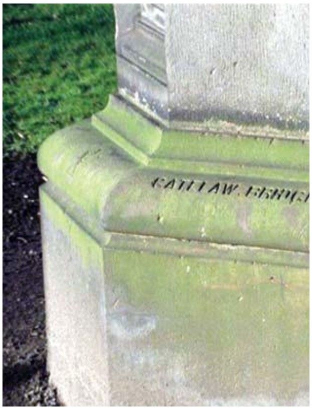
5. Detailed contribution by B.Fitzner, K.Heinrichs, R.Kownatzki (1995) on classification and mapping of weathering forms., which was update in 2002 . This document presents as well definitions of terms which are found in a slightly altered form in the present glossary, as an introduction into the mapping of stone damages.

6. Multiauthored book (Franke et al. 1998) published as a deliverable of a FP5 European Commission research program. The document is an Atlas and a classification of brick masonry deterioration. It deals both with deterioration of the material (brick, joint and pointing mortars) and with degradation of the whole masonry. It was developed together with an expert system, of which the acronym is MDDS, which stands for „Masonry Damage Diagnostic System".

The ISCS glossary only contains terms related to stone material as an individual element within a built object or sculpture. As a consequence, the terms do not relate to the description of the deterioration of a stone masonry structure as a whole.

The ISCS glossary is arranged into 6 families :

- General terms (Alteration, Damage, Decay, Degradation, Deterioration, Weathering)



Common alteration of architectural mouldings by algae.

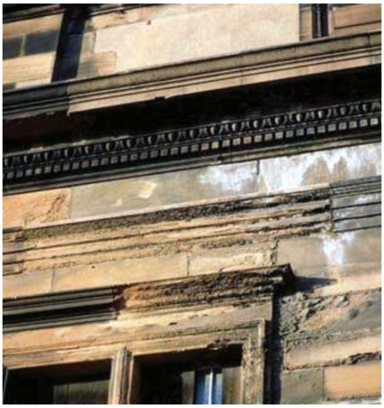
***Alteration** – modification of the material that does not necessary imply a worsening of its characteristics from the point of view of conservation. For instance, a reversible coating applied on a stone may be considered as an alteration.*

Damage to the lower part of a sandstone grave slab resulting in loss of value.

***Damage** –human perception of the loss of value due to decay.*

Limestone relief showing advanced decay.

***Decay** – any chemical or physical modification of the intrinsic stone properties leading to a loss of value or to the impairment of use.*



Degradation of red sandstone masonry due to defective rainwater gutter behind parapet.

***Degradation** – decline in condition, quality, or functional capacity.*

Deterioration of a Carboniferous sandstone masonny.

***Deterioration** – process of making or becoming worse or lower in quality, value, character, etc....; depreciation.*

Weathering of a Lewisian Gneiss monolith resulting from long term exposure to the elements.

***Weathering** – any chemical or mechanical process by which stones exposed to the weather undergo changes in character and deteriorate.*

• Crack and deformation (Fracture, Star crack, Hair crack, Craquele-crack network, Splitting, Deformation).

Cracking may be due to weathering, flaws in the stone, static problems, rusting dowels, too hard repointing mortar. Vibrations caused by earth tremors, fire, frost may also induce cracking.

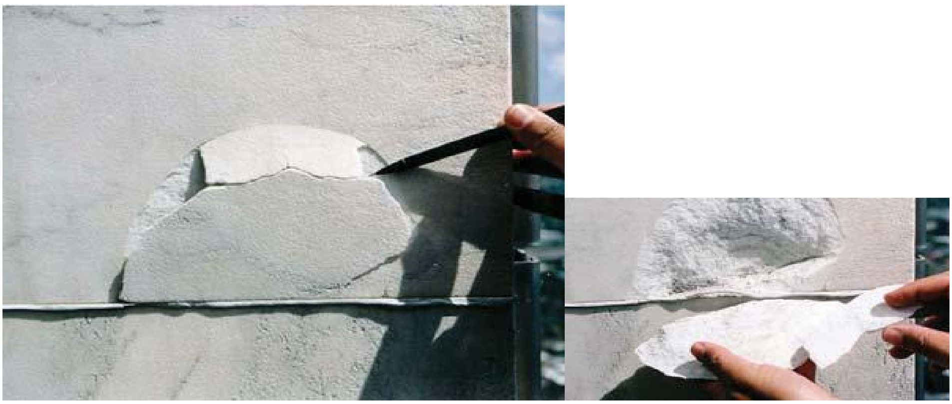
Crack and fractures occuring on rock carved surfaces are usually named after the geological terminology; Joint if there is no displacement of one side with respect to the other; Fault if there is a displacement.

Deformation is change in shape without loosing integrity, leading to bending, buckling or twisting of a stone block.

• Detachment (Blistering, Bursting, Delamination, Disintegration, Fragmentation, Peeling, Scaling)

Blistering of sandstone masonry caused by expansion of the weathered surface layer leading to loss of the stone surface.

***Blistering** – separated, air-filled, raised hemispherical elevations on the face of stone resulting from the detachment of an outer stone layer. This detachment is not related to the stone structure.*



*Typical **bursting** at flat wall marble panel.*

***Bursting** – local loss of the stone surface from internal pressure usually manifesting in the form of an irregularlysided crater.*

sandstone **exfoliation.**

*This subtype of **delamination** is characterised by a detachment of multiple thin stone layers sub-parallel to the stone surface.*

***disintegration- sanding** of a coarse grained granite.*

Detacment of single grains or aggregates of grains.

In the case of crystalline marbles, thermal stresses are known to be one of the main causes of granular disintegration, thus leading occasionally to deformation patterns.

Stones may display deterioration patterns intermediate between granular disintegration and crumbling, scaling or delamination.

Contour scaling, developing on a magmatic stone element (Kersanton).

Contour scaling- scaling in which the interface with the sound part of the stone is parallee to the stone surface. In the case of flat surfaces contour scaling may be called spalling.

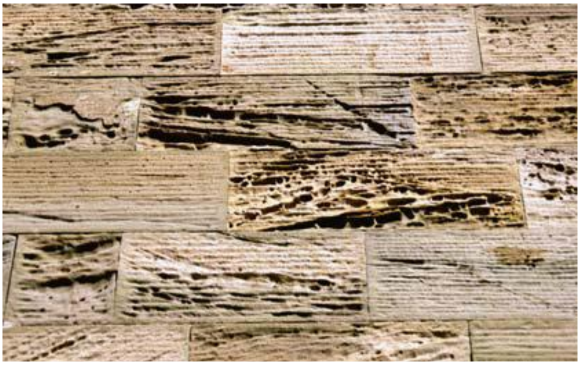
Spaling

Sandstone block contaminated with sodium chloride. Salt crystallization induces granular disintegration and scaling of the stone. As scales are very thin, the degradation pattern is also called flaking.

- Features induced by material loss (Alveolization, Erosion, Mechanical damage, Microkarst, Perforation, Pitting)

Alveolization – formation, on the stone surface, of cavities (alveoles) which may be interconnected and may have variable shapes and sizes (generally centimetric, sometimes metric).

Alveolization is a kind of is a defferential weathering possibly due to inhomogeneities in physical or chemical properties of the stone. Alveolization may occur with other degradation patterns such as granular disintegration and/or scaling.



***Alveolization** develops here as cavities illustrating a combination of **honeycombs** and alignments following the natural bedding planes of the sandstone.*

***Microkarst** – network of small interconnected depressions of milimetric to centimetric scale, sometimes looking like hydrographic network. Microkarst patterns are due to a partial and/or selective dissolution of calcareous stone surfaces exposed to water run-off.*

Microkarst developed on the base of a chalk column particularly exposed to weather.

Pitting – point-like millimetric or submillimetric shallow cavities. The pits generally have a cylindrical or conical shape and are not interconnected, although transitions patterns to interconnected pits can also be observed.

Pitting developping on a marble sculpture. Microbiological origin is probable.

• Discoloration and deposit (Crust, Deposit, Discolouration-colouration, bleaching, moist area, staining, Efflorescence, Encrustation, Film, Glossy Aspect, Graffiti, Patina, Soiling, Subflorescence)

Efflorescence - generally whitish, powdery or whisker-like crystals on the surface. Efflorescences are generally poorly cohesive and commonly made of soluble salt crystals.

Efflorescence is commonly the result of evaporation of saline water present in the porous structure of the stone. Efflorescence are often constituted of soluble salts such as sodium choride ($NaCl$) or sulphate (Na_2SO_4), magnesium sulphate ($MgSO_4$ $7H_2O$), but they may also be made of less soluble minerals such as calcite ($CaCO_3$), barium sulphate ($BaSO_4$), and amorphous silica (SiO_2nH_2O).

*Formation of salts forming **efflorescence** on the surface of sandstone masonry, focused at joints between masonry blocks.*

Encrustation – compact, hard, mineral outer layer adhering to the stone. Surface morphology and colour are usually different from those of the stone.

Encrustations on monuments are frequently deposits of materials mobilized by water percolation and thus coming from the building itself: Carbonates, sulphates, metallic oxides and silica are frequently found.

Calcite **encrustation** linked to water leached from joints, on a granite, sandstone and schist ashlar.

- Biological colonization (Alga, Lichen, Moss, Mould, Plant)

*Chalk sculpture, showing **mosses**, which appear brownish (typical aspect during the dry season), and are developed on the upper part of the figure.*

Mosses often change morphology and colour under lack or excess of water. During dry periods of the year the cushions shrink become harder and brittle and their colour turns to brown.

3. TESTS OF EXISTING STONE SAMPLES FROM LOCATION „HELLENISTIC URBAN COMPLEX OF RUINS OF FORT OF OŠANIĆI NEAR STOLAC, BIH“

The tests of the existing stone from the archeological location at the ruins of fort of Ošanići near Stolac have been conducted in order to evaluate quality and stone destruction cause.

Taking stone samples was carried out at the following positions:

- Gate – entry into town – stone from restoration in 1974
- Megalithic wall – internal side at beginning of gate – right stretch
- Entrance gate, „filling“ – pile – internal side right stretch
- Megalithic wall – inside – where it is green
- Southern staircase below town towards Radimlja – stone from restoration
- Southern staircase below town towards Radimlja

- Underpinning and staircase at foot of huge mass, south-west
- Northern wall (monolithic solitary one)
- Eastern wall next to huge mass
- Quarry – visually compact stone
- Quarry – stone with visual destruction
- Quarry – stone with visible destruction (monolithic solitary one)
- On way to quarry – megalithic wall – stone with visible destruction (monolithic solitary one)

After the detailed visual examination and the comparison of samples formed at the mentioned positions, the selection of representative samples started in order to study the stone decomposition causes and to determine the stone type.

On the occasion of forming the representative samples, the care was taken that all visually different samples are comprised by tests.

Picture 1.

Sample 1 – quarry and on way from quarry – megalithic wall → stone with visible destruction (monolithic solitary one)

Picture 2.

Sample 1 – quarry and on way from quarry – megalithic wall stone with visible destruction

Picture 3.

Sample 2 – gate – entrance into town – stone from restoration in 1974

Picture 4.

Sample 3 – eastern wall next to huge mass

Picture 5.

Sample 4 – megalithic wall – inside – Biological colonization

The aim of forming the representative samples and work program of research was to establish whether the existing stone which is built into the megalithic wall (at all positions), the stone from the potential site – quarry, as well as the stone used on the occasion of restoration of individual constructional elements belong to the same stone type, in petrographic sense and in view of physical and physical-mechanical properties. Furthermore, on the basis of such data, further analysis would start in order to evaluate the destruction causes.

The mineralogical-petrographic analysis and the chemical analysis were carried out on characteristic samples which were formed on the following positions:

- Quarry and on way quarry-megalithic wall → stone with visible destruction (monolithic solitary one)
- Gate –entrance in town –stone from restoration in 1974
- Eastern wall next to huge mass

The mineralogical-petrographic analysis established that all three tested samples belong to sedimentary carbonate rocks and they are limestones. These are the varieties with fine rich fossil detritus, which is well sorted and very well packed in carbonate (calcite) matrix and it is microcrystalline and partly finely recrystallized crystalline. The fossil detritus is represented mainly by fine pelagic foraminiferal fauna and it is represented more than 30%. Based on composition, structure and texture, they are identified as biogenic limestones and they are biomicrites and fossil microsparites.

The rock texture is massive. The rock texture is microcrystalline and partly fine-grained psammitic.

The basic mass is created of microcrystalline and partly of cryptocrystalline calcite and it is finely dispersed with a small quantity of marly-clayey substance. In this fine micrite matrix, different remains of fine pelagic organogenic detritus (geol. – stone fragments which emerged from stone disintegration, friction and stripping) are disposed and the remains of fine pelagic detritus is represented by fine roundish forms.

Nearly all the fossils which build this rock are the pelagic foraminifers represented by globigerine, less frequently by globotruncana and other tiny microfauna. The fossil detritus is represented more than 30%.

In the chemical analysis, the following is carried out:

– quantitative determination of main stone components which are expressed in the form of oxides (analysis results are given in the table 2.1. – contents of incombustible substances),

– quantitative analysis of hazardous soluble salts in stone – anions (chlorides, sulfates, nitrates, carbonates and phosphates) and cations (calcium ion, magnesium ion, sodium ion and potassium ion), evaporation remnants of the total quantity of soluble salts, pH value and electric conductivity of their water extract (analysis results are given in table 2.2.).

Table 2.1. Chemical analysis results

Symbol	SiO_2	Al_2O_3	TiO_2	Fe_2O_3	CaO	MgO	MnO	P_2O_5	SO_4	K_2O	Na_2O
Sample 1 (%)	< 0,02	< 0,02	< 0,02	0,11	99,22	0,45	< 0,02	0,04	~ 0,01	0,02	0,07
Sample 2 (%)	< 0,02	< 0,02	< 0,02	0,18	99,12	0,43	< 0,02	0,07	~ 0,01	0,05	0,07
Sample 3 (%)	< 0,02	< 0,02	< 0,02	0,14	99,42	0,20	< 0,02	0,04	~ 0,01	0,04	0,07

The contents of the parameters which are determined in the eluate, in which process the analytic sample was treated by boiling in redistilled water, are given in the table 2.2.

Table 2.2. Chemical analysis results – eluate

Symbol	Dry remnant (mg/g)	Cl^- (mg/g)	SO_4 (mg/g)	Ca (mg/g)	Mg (mg/g)	Na (mg/g)	K (mg/g)	NO_3 (mg/g)	El. conductivity (S)	pH
Sample1	1,20	< 0,01	0,1	0,06	0,003	0,01	0,003	0,01	$1,0x10^{-4}$	6,5
Sample2	1,30	< 0,01	< 0,1	0,08	0,003	0,01	0,015	0,01	$1,4x10^{-4}$	6,6
Sample3	1,30	< 0,01	< 0,1	0,07	0,003	0,01	0,004	0,07	$3,0x10^{-4}$	6,0

The pH value of the eluate of limestone reference sample is 8,5, while the electric conductivity of redistilled water is $3,3x10^{-6}$ S.

Lower pH value and higher electric conductivity of water extract are noticed in all the tested samples.

The chemical analysis results are quite aligned.

The pH value decrease, i.e. stone acidity increase, is concluded in all the tested samples.

The stone acidity increase occurred by the joint bacteria effects and rinsing solved mineral substances.

The flora which is composed of very simple organisms (bacteria, simple algae, mold, lichens and moss) can colonize on all stone type surfaces, by destroying it primarily by biochemical effects; by producing carbonic, nitric, sulfuric and some other weaker acids, and by attacking the present minerals. The organic acids emerge from the decomposition of dead vegetation, and by their getting amongst mineral grains and inside the pores, the flora expands the interstices and so opens the path for the other destruction agents. Compact and fresh stone is not easily attacked by the side of bacteria, but the stone which is affected by the time-related alteration can be the habitat of large bacterial colony along all the surfaces, and along the cracks.

On the other side, the water which, by recurring moistening, gets into the porous limestone during its stay in its interior dissolves the soluble substances. In the desiccation phases together with the humidity movement towards the surface, also the dissolved substances move and they are deposited on the surface parts of the pores or on the surface itself. The transfer of the mineral substance from the internal parts towards the surface, leads to the weakening of connection between undissolved mineral components, as well as the decrease of firmness and even disintegration, while at the same time in the surface parts the density increases by forming carbonized layer of hardly soluble $CaCO_3$, which can be disintegrated, due to the atmospheric agents, waters containing the carbonic acid ($CO_2 + H_2O$), into easily soluble calcium bicarbonate.

Fine grayish surface film – patina or "calcina" of formed carbonized layer of width of about 0,5-2 mm with improved mechanical properties, stormily reacts when cold diluted hydrochloric acid is poured onto it.

The obtained values of anion concentration (of chloride, sulfate and nitrate) are classified according to the following table (in accordance with Kerasan Informatons Broschüre, Wien, 1994):

Table 2.3. Classification of anion concentration harmfulness

Anions	Harmless concentration (%)	Possible harmful concentration (%)	Harmful concentration (%)	Contents (mg/g)		
				Sample 1	Sample 2	Sample 3
Cl^-	< 0,03	0,03-0,09	Greater than 0,09	< 0,01	< 0,01	< 0,01
SO_4^{2-}	< 0,08	0,08-0,24	0,24	0,1	< 0,1	< 0,1
NO_3^-	< 0,05	0,05-0,15	Greater than 0,15	0,01	0,01	0,07

The harmless concentration of anions, as well as of alkalis was concluded in all the tested samples.

The soluble salts are known as the important agents in physical destruction of solid rocks in the nature as well as of stone in the constructions. In the nature, they are characteristic for the areas with arid climate. In the areas with moderately humid climate, the appearance of soluble salts on the rock surfaces is less frequent and they can be seen at quarry open surfaces or natural outgrowths.

The water is the medium in carrying over the soluble salts, thus their appearance is connected with ways of water movements through the stone. The salt efflorescences occur mainly at the borders between damp and dry wall zones, such as upper border of the damp from the ground or dampened surfaces in their drying process.

The nitrates are present in the traces in many cases of efflorescence, and as prevailing they are only in the environments where decomposing organic material exists (sample 3). The chlorides, although very little, are brought in by the activities of sea water and they are brought by south wind.

On the occasion of mineralogical-petrographic analysis of the sample which is formed on the eastern wall next to the mass, the presence of yellowish-brown patina from iron hydroxide on the surface is concluded, and which is the most probably the consequence of the effects of the iron bacteria – Ferrobacillus ferroxidans and Thiobacillus ferrooxidans, which play very important role in the iron oxidation process.

The location of the Hellenistic urban complex at the ruins of fort of Ošanići is at the altitude of about 300 meters above sea and air distance of about 25 km from Neum, with very characteristic wind of north direction, which is known by local name "Šever", with very strong, even stormy squalls mostly in the second half of December and in January.

It is unquestionable that such circulations had impact on the time-related changes on the stone, and especially having in mind the position of megalithic wall and town in relation to wind direction, as well as position of individual constructive parts in relation to solar radiation.

It is very interesting to notice that it is possible that the difference in temperature of external and internal surface had influence on the stone destruction.

Namely, considering that it is about porous fossiliferous limestone, the damp accumulation is prominent in the porous area. On the occasion of warming up which occurs by insolation, the water in the pores expands, and if that insolation occured directly after the dampening (which is exactly the characteristic of these areas), when the pores are saturated by water, significant stretching strains appear in the stone.

Due to the effects of north (continental) wind which has low relative humidity and lower temperature, significant evaporation from the stone surface occurs during constant long lasting cycles of alternating drying and humidifying. Also, knowing north wind characteristics in that area, it is probable that there appeared also great differences in temperature of warm heated interior of stone facets, which were heated by west and south-west Mediterranian solar radiation (to the position of stone site – quarry, the mouth of the Radimlja in Mediterranian Sea was navigable, so that the influence of the Mediterranian climate along the river valley prevailed), and cold surface dried by the wind. It is very important to highlight that there met the influences of south wind from the sea, which are milder, warmer and humid, and cold north wind, which is cold, strong and stormy.

Although by the position of archeological location more significant presence of chloride was expected because of the vicinity of the sea, very low contents of these soluble salts were obtained, and it was established in the tested samples, it most probably reflects the natural circulation in the interweaving of strong north wind and milder south wind.

The other deciding climatic factors in the time-related decay of Hellenistic urban complex at the ruins of fort of Ošanići are quantity and pattern of precipitations during year, differences in air temperature during day and year, water temperature and contents of atmosphere, climatic and microclimatic differences (the following is prominent: change of water course of the Radimlja, withdrawal of navigable part of the Radimlja, changes occurred in the atmosphere composition due to industrialization of the region and more and more influence of acid rains), and they had effects on porous fossiliferous limestone.

The presence of lichens is very prominent on the stone location, since they are the first colonists of bare stone surfaces and they participate in the beginning of formation of leaf mold which enables the appearance of the higher plant types.

The calcareous lichens are formed at great extent on the carbonate rocks. It is considered that the lichens are the most fruitful producers of stone eroders, they stand the extremes in temperature and can survive for a long time without damp, but sulphur, nitric oxide and increased quantity of carbon dioxide in atmosphere make their development difficult, thus it is hard for them to develop in zones of polluted atmosphere.

Within the frame of the planned standard tests of natural stone, the tests are carried out in relation to apparent density with pores and cavities, to real density without pores and cavities, to porosity, to density coefficient and to values of water absorption.

The other tests have not been carried out, since on the occasion of taking the samples, the total destruction occurred, due to the enormous number of cracks, even in the stone which appeared during the visual examination as being a healthy mass.

Based on the information which was obtained on the occasion of taking samples and forming experiment for testing, it is concluded that, on the stone, which is built into all the construction elements of the archeological site, and which visually gives impression of a healthy facet, under the carbonized and mechanically improved patina surface, there is the interior which is affected by the destruction (uncemented open cracks, destruction and partition).

The apparent density of stone with pores and cavities is approximately 2630 kg/m^3, while real density is approximately 2740 kg/m^3.

The density coefficient ($i = \frac{g}{g_S}$) is 0,960, and general (absolute) porosity is 4,0 %.

The value of water absorption is established by method of sinking and it is within the limits from 0,4 to 1,0 %.

The differences between true (absolute) porosity and relative (open) porosity, which shows which part of pores and cavities is available for water, they appear because the pores in the stone are not mutually connected, either because they do not keep the water because of their size and/or because at atmospheric pressure they cannot be filled with the water. The absolute porosity represents the limit of relative porosity.

Based on calculated values of porosity and water absorption, tested existing stone at the archeological site falls into the category of porous stone with moderate water absorption.

In accordance with the tests conducted on the samples of stone existing at the location of Hellenistic urban complex at the ruins of fort of Ošanići near Stolac, and in order to evaluate quality and cause of stone destruction, it was established that the existing stone which is built into the megalithic wall (with great probability at all positions), the stone from the potential site-quarry, as well as the stone used on the occasion of restoration of individual construction elements belong to the same stone type and these are sedimentary carbonate rocks, limestones.

The protection of the existing stone from further forthcoming definitive destruction implies the priority and complete protection from the influences of atmospheric agents.

4. DIRECTIVE 89/106 CONSTRUCTION PRODUCTS (CPD) IN STONE SECTOR

For the elimination of the technical barriers, the European Community took various initiatives, amongst them the maximum development of the CEN (European Standardization Committee) activity for the issuing of European Standards to be obligatory accepted by all the European Member States.

The Directive 89/106/CEE on the construction products (CPD) is one of the first document prepared for this scope. The main aim of the CPD is to remove the barriers of a techical nature which impede the free circulation of construction products on the commnunity market.

According to the CPD the construction products can be placed on the market only if they aresiutable for the intended use. That is to say if they have such characteristics that the works in which they are to be incorporated, assembled, applied, can, if properly designed and built, satisfy the following six essential requirements:

1. Mechanical resistance and stability
2. Safety in case of fire,
3. Hygiene, health and the environment,
4. Safety in use,
5. Protection against noise,
6. Energy economy and heat retention

As the essential requirements refer to works and not to products, the CPD requested a series of explanatory documents (one for each essential requirement). These have the aim of establishing the link between the essential requirements of the works and the performance characteristics of the products belonging to them and constitute the basis of the mandates which the European Commission confers upon the European Standard Bodies for the drawing up of technical harmonised specifications.

4.1. Standards of reference

In the range of the Construction Products Directive the Technical Committees of CEN that are elaborating the Standards relative to stone material are:

- The CEN/TC 246 „Natural Stone“, which deals with construction products of stone for internal and external cladding, internal flooring and stairs, modular tiles and dimensional stone work;
- The CEN/TC 178 „Units for external paving and kerbs“, which Work Group 2 deals in products in natural stone for external paving
- The CEN/TC 128 „Products for discontinuous roofing“ which under Commission 8 treats slates and other stone materials for roofing,

- The CEN/TC 125 „Masonry“ which Work Group 1 has a Task Group (TG 7) which deals with natural stone products used in masonry.

Table 3.1. – Classification of Standards regarding natural stone products

1.	**Product Standards**
1.1	**Semi-finished products**
1.1.1.	Rough blocks (EN 1467)
1.1.2.	Rough slabs (EN 1468)
1.2.	**Finished products**
1.2.1.	Slabs for cladding (EN 1469)
1.2.2.	Slabs for flooring and stairs (EN 12058)
1.2.3.	Modular tiles (EN 12057)
1.2.4.	Slabs for external paving (EN 1341)
1.2.5.	Setts for external paving (EN 1342)
1.2.6.	Kerbs for external paving (EN 1343)
1.2.7.	Products for discontinuos roofing (EN 12326-1)
1.2.8.	Masonry units (EN 771-6)
1.2.9.	Dimensional stone work (standard project in elaboration phase)
2.	**Terminological Standards** EN 12440 Denomination EN 12670 Terminology
3.	**Test methods** *
3.1.	**Geometrical**
EN 772-16	Determination of dimensions of masonry units
EN 772-20	Determination of flatness of faces of masonry units
EN 13373	Determination of geometric characteristics on units
3.2.	**Petrographic**
EN 12407	Petrographic examination
3.3.	**Physical-mechanical**
EN 772-1	Determinatio of compressive strength of masonry units
EN 772-11	Detrmination of water absorption of masonry units due to capillary
EN 1925	Detrmination of water absorption coefficient by capillarity
EN 1926	Determination of compressive strength
EN 1936	Detrmination of real density and apparent density and of total and open porosity
EN 12372	Detrmination of flexural strength under concentrated load
EN 13161	Detrmination of flexural strength under constant movement
EN 13364	Detrmination of the breaking load at dowel hole
EN 13755	Detrmination of water absorption at atmospheric pressure
EN 14146	Determination of the dynamic modulus of elasticity (by measuring the fundamental resonance frequecy)
EN 14158	Determination of rupture energy
EN 14205	Determination of Knoop hardness
EN 14579	Determination of sound speed propagation
EN 14580	Determination of static elastic modulus
EN 14581	Determination of thermal expansion coefficient

3.4.	**Durability**
EN 12370	Determination of resistance to salt crystallisation
EN 12371	Determination of frost resistance
EN 13919	Determination of resistance to ageing by SO_2 action in the presence of humidity
EN 14066	Determination of resistance to ageing by thermal shock
EN 14147	Determination of resistance to ageing by salt mist
3.5.	**Technological Testings**
EN 14157	Determination of the abrasion resistance
EN 14231	Determination of the slip resistance by means of the pendulum test
* **For Slate for discontinuous roofing only a single Standards exits with all the testings methods of EN 12326-2**	

All the product standards have the same structure, in that three parts can be distinguished: introduction, specification of requirements, and evaluation of conformity.

The attestation of conformity can be issued by a recognised Certification Body or by the manufacturer and it is based on the results of the initial type testing of the product and on the Factory Production Control. Since for natural stone the intervention of a recognised Certification Body is not envisaged, not the initial type testing on product which the Factory Product Control are effected by the manufacture under his responsibility (system 4 of attestation of conformity). Only in the case in which resins are added to the products or in the case in which the use envisaged is the ceiling cladding, is it necessary to carry out the initial type testing of reaction to fire or of flexural strength respectively by a notified laboratory (system 3 of attestation of conformity).

5. CONCLUSION

The steadiness of natural stone is the function of its mineralogical-petrographic composition, its structure and texture; compactness, structural homogeneity and thermal stability.

In exceptionally huge number of negative effects, it is necessary to, for every individual case of stone application, identify those which are necessary: sun, wind, water, frost, chemical agents of human environment.

In order to prevent the destruction and to meet the esthetic criteria, it is necessary to provide the monitoring program and to maintain the stone which is built in.

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Mihailo Muravljov[1]

UDK:624.971.059.2

ONE CASE OF REHABILITATION OF REINFORCED CONCRETE CHIMNEY USING CFRP STRIPS

Summary: This *paper presents solutions of reinforced concrete chimney core repair where the degree of damage necessitated undertaking radical remedial interventions. Damages to the chimney are manifested in the form of very pronounced vertical cracks and fissures formed as a result of damage to the insulating lining inside the flue duct and reinforced concrete, i.e. as a result of exposing the concrete core to the temperatures near 200* C^0*. A more or less cosmetic repair of surface damages present on the outer surface of the chimney was done but the chimney was structurally strengthened by installing reinforcement rings of CFRP strips glued to the outside of the chimney. The decision to use CFRP strips is based on the fact that the applied heat calculations showed that even in the critical summer operating conditions and in cases where there is damage inside the insulating wall of the chimney the temperature on the outer side of the concrete core never exceeded 60°C. The paper pays special attention to extensions of carbon strips, and in that sense, the appropriate extension verified by experimental tests was applied.*

Keywords**:** *chimney mantle, thermal insulation masonry, cracks,* segregation *of concrete, grout,* carbon *strip, epoxy glue, strip extension*

JEDAN SLUČAJ SANACIJE ARMIRANOBETONSKOG DIMNJAKA PRIMENOM KARBONSKIH TRAKA

Rezime: *U radu se daje prikaz rešenja sanacije plašta armiranobetonskog dimnjaka blokova* A_1*,* A_2 *i* A_3 *TENT - Obrenovac, kod koga je stepen oštećenja nalagao preduzimanje radikalnih sanacionih intervencija. Oštećenja na dimnjaku su se manifestovala u vidu vrlo izraženih vertikalnih prslina i pukotina nastalih kao posledica oštećenja termoizolacione obloge u unutrašnjosti dimnjačke cevi, odnosno kao posledica izloženosti armiranobetonskog plašta temperaturama veličine i blizu 200°C. U sklopu sanacije izvedena je manje-više kozmetička popravka površinskih oštećenja prisutnih na spoljašnjoj površini dimnjaka (sanacija prslina, segregacija betona i dr.), ali je izvedeno i ojačanje plašta dimnjaka putem prstenova od karbonskih traka zalepljenih za dimnjak sa spoljašnje strane. Odluka o primeni karbonskih traka, između ostalog, zasnovana je na činjenici da su sprovedeni termički proračuni pokazali da čak i u letnjim uslovima eksploatacije kao kritičnim, a u slučajevima kada dođe do oštećenja termoizolacionog ozida u unutrašnjosti dimnjaka, na spoljnoj površini betonskog plašta temperatura nikada nije veća od 60°C. U radu je posebna pažnja posvećena nastavcima karbonskih traka, pri čemu je u tom smislu primenjen detalj nastavka verifikovan odgovarajućim eksperimentalni ispitivanjem.*

Ključne reči: *plašt dimnjaka, termoizolacioni ozid, prslina, segregacija betona, injektiranje, karbonska traka, epoksidni lepak, nastavak trake*

[1] Professor emeritus, PhD, Grad. Civ. Eng, the Faculty of Civil Engineering, Belgrade, Bulevar kralja Aleksandra 73, Belgrade

1. INTRODUCTION

The reinforced concrete chimneys built in the second half of the 20th century, usually as a part of newly built coal-fired power plants were designed on an more than optimistic assumption that the concrete core (wall) of the chimney will be efficiently and durably protected from the effects of high flue gases temperature by the thermal insulation masonry lining. On the basis of this assumption, the temperatures measured in the middle of the reinforced concrete core walls of chimneys seldom exceeded 30°C, which was a ±10°C do ±20°C boundary variation in relation to the stated mean temperature, depending on the thickness of the chimney core at the location where the measure was taken, thus on the basis of such temperature effects, very small quantities of horizontal - ring reinforcement were adopted. However, in the situations when the thermal insulation would be damaged at some place in the chimney interior, the reinforced concrete core would at this place be exposed to the internal temperature reaching environs of 200°C, so as the chimney was not designed for such temperature, quite logically, its core would sustain very pronounced, mostly vertical cracks and fissures. Such defects, which were accompanied by reaching the tensile limits in horizontal – ring reinforcement, could be registered and possibly rehabilitated only at the occasion of control inspections of the chimneys. Those inspections, according to the standing codes and standards, were performed on alternating years, at which occasions, and it should be specially emphasized, only the repairs of localized damages of the chimney linings were performed. For those reasons, the external surfaces of a large number of chimneys exhibited very pronounced network of mostly verticals cracks and fissures along their entire heights. They were the result of multiple damages and rehabilitations of thermal insulation brickwork, and the result of long lasting fissure formation of the chimney core.

All the mentioned factors lead to compromising of bearing capacity and stability of chimneys which are specific reinforced concrete structures, so it is totally logical that an increased presence of cracks and fissures on the external surfaces of the chimney requires appropriate rehabilitation interventions. These interventions, in principle, should not comprise only the “cosmetic” rehabilitation of the mentioned defects, but should include the appropriate reinforcement of the structure in terms of rings, so as to enable the chimney in the future to resist the increased temperature effects occurring when the thermal insulation brickwork is damaged.

This paper will present the design of remedial works on the core of the reinforced concrete chimney of the A_1, A_2 and A_3 blocks of TENT (Nikola Tesla Power Plant) - Obrenovac, whose level of damage required undertaking very radical remedial measures.

2. BRIEF DESCRIPTION OF THE CHIMNEY STRUCTURE AND DAMAGE

The chimney that will be discussed is an reinforced concrete structure 150 m in relation to the surface of the surrounding terrain. It is founded on the circular reinforced concrete slab dug-in 6m below the surface level, with the main structure of the chimney starting from the foundation slab having the shape of a hollow truncated cone. The thickness of the chimney walls ranges between 18cm to 45cm, becoming thinner with the increasing height. The outer diameter of the chimney at the ground level is 15.20m, while the diameter at the top is cca 11,00m. At the height of 8,50m above the ground, three flue ducts are introduced in the chimney, having the openings 5,0x7,5m. In figure 1, basic geometrical characteristic of the chimney have been schematically presented, as obvious, defined in sections I – XV, which correspond to the phases of construction.

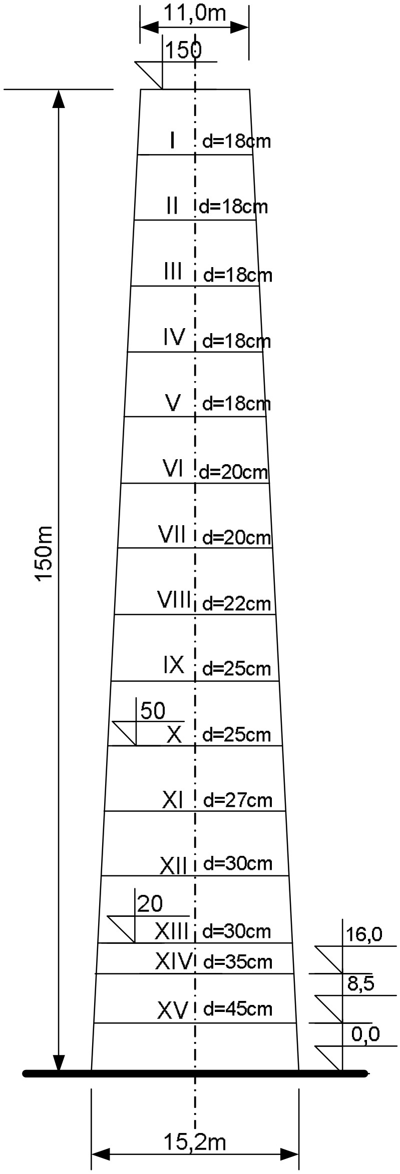

Figure 1. Schematics of basic geometrical characteristics of the chimney

For the purpose of protection of concrete from the direct thermal and chemical effects of flue gases, a fireclay masonry wall is constructed inside the chimney, lining the reinforced concrete core wall. This lining is placed on the consoles – shelves at every 10m of height.

In table 1 there is a presentation of the chimney core reinforcement, according to the main design used for construction of the chimney.

Table 1. Reinforcement in the chimney core – cross section surface area

S e k t o r	HORIZONTALNA ARMATURA - prisutna samo uz spoljašnju površinu -	VERTIKALNA ARMATURA - prisutna samo uz spoljašnju površinu -
(1)	(2)	(3)
Lamela XV	2Ø10/10cm → **A_a= 15,70cm^2/m**	9Ø18/m → **A_a= 22,89cm^2/m**
Proširenje XV - XIV	2x14Ø10/1,25m → A_a = 17,58cm^2/m	9Ø18/m → **A_a= 22,89cm^2/m**
Lamela XIV	2Ø10/10cm → **A_a= 15,70cm^2/m**	9Ø18/m → **A_a= 22,89cm^2/m**
Proširenje XIV - XIII	2x14Ø10/1,25m → A_a = 17,58cm^2/m	9Ø18/m → **A_a= 22,89cm^2/m**
Lamela XIII	Ø10/10cm → **A_a= 7,85cm^2/m**	6Ø16/m → **A_a= 12,00cm^2/m**
Proširenje XIII - XII	14Ø10/1,25m → A_a= 8,79cm^2/m	6Ø16/m → **A_a= 12,00cm^2/m**
Lamela XII	Ø10/10cm → **A_a= 7,85cm^2/m**	6Ø16/m → **A_a= 12,00cm^2/m**
Proširenje XII - XI	14Ø10/1,25m → A_a= 8,79cm^2/m	6Ø16/m → **A_a= 12,00cm^2/m**
Lamela XI	Ø10/15cm → **A_a= 5,23cm^2/m**	7Ø14/m → **A_a= 10,77cm^2/m**
Proširenje XI - X	14Ø10/1,25m → A_a= 8,79cm^2/m	7Ø14/m → **A_a= 10,77cm^2/m**
Lamela X	Ø10/20cm → **A_a= 3,92cm^2/m**	7Ø14/m → **A_a= 10,77cm^2/m**
Proširenje X - IX	14Ø10/1,25m → A_a= 8,79cm^2/m	7Ø14/m → **A_a= 10,77cm^2/m**
Lamela IX	Ø10/20cm → **A_a= 3,92cm^2/m**	7Ø14/m → **A_a= 10,77cm^2/m**
Proširenje IX - VIII	14Ø10/1,25m → A_a= 8,79cm^2/m	7Ø14/m → **A_a= 10,77cm^2/m**
Lamela VIII	Ø10/20cm → **A_a= 3,92cm^2/m**	8Ø12/m → **A_a= 9,04cm^2/m**
Proširenje VIII - VII	14Ø10/1,25m → A_a= 8,79cm^2/m	8Ø12/m → **A_a= 9,04cm^2/m**
Lamela VII	Ø10/20cm → **A_a= 3,92cm^2/m**	7Ø12/m → **A_a= 7,91cm^2/m**
Proširenje VII - VI	14Ø10/1,25m → A_a= 8,79cm^2/m	7Ø12/m → **A_a= 7,91cm^2/m**
Lamela VI	Ø10/20cm → **A_a= 3,92cm^2/m**	7Ø12/m → **A_a= 7,91cm^2/m**
Proširenje VI - V	14Ø10/1,25m → A_a= 8,79cm^2/m	7Ø12/m → **A_a= 7,91cm^2/m**
Lamela V	Ø10/20cm → **A_a= 3,92cm^2/m**	6Ø12/m → **A_a= 6,78cm^2/m**
Proširenje V - IV	14Ø10/1,25m → A_a= 8,79cm^2/m	6Ø12/m → **A_a= 6,78cm^2/m**
Lamela IV	Ø10/20cm → **A_a= 3,92cm^2/m**	6Ø12/m → **A_a= 6,78cm^2/m**
Proširenje IV - III	14Ø10/1,25m → A_a= 8,79cm^2/m	6Ø12/m → **A_a= 6,78cm^2/m**
Lamela III	Ø10/20cm → **A_a= 3,92cm^2/m**	6Ø12/m → **A_a= 6,78cm^2/m**
Proširenje III - II	14Ø10/1,25m → A_a= 8,79cm^2/m	6Ø12/m → **A_a= 6,78cm^2/m**
Lamela II	Ø10/20cm → **A_a= 3,92cm^2/m**	6Ø12/m → **A_a= 6,78cm^2/m**
Proširenje II - I	14Ø10/1,25m → A_a= 8,79cm^2/m	6Ø12/m → **A_a= 6,78cm^2/m**
Lamela I	Ø10/20cm → **A_a= 3,92cm^2/m**	6Ø12/m → **A_a= 6,78cm^2/m**

In case when the lining inside the chimney is damaged, the concrete core at this point is exposed to the temperature of cca 190°C. Since the chimney is not designed for such temperature effects, its core exhibited during service (since the seventies of the 20th century) very pronounced, primarily vertical cracks and fissures (figures 2 and - 3 left). Namely in the past period, there were multiple damages of the lining, thus the cracks and fissures on the outer surface of the chimney, prior to remediation, were the result of a cumulative process lasting many years.

Figure 2. Vertical cracks on the chimney core

Apart from the mentioned cracks and fissures, on the exterior of the chimney core, other kinds of damage were observed, whereby that damage were mostly the result of certain errors made during the construction, and due to weathering. With regard to that, it can be considered that there were the following types of more or less surface damage on the outer surface of the chimney:

- concrete segregation (figure 3 – right), blistering of the surface layers of concrete due to the reinforcement corrosion, and spalling of thin layers of concrete along the rebars, which was a consequence of the reinforce corrosion and expansion of corrosion products,
- cracks with gaps up to 0,4mm,
- cracks with gaps larger than 0,4mm up to the maximum of 0,8mm,
- cracks-fissures with the gaps larger than 0,8mm.

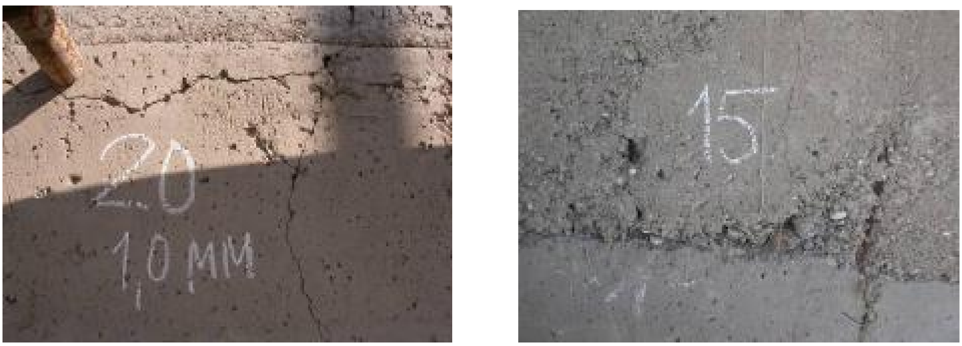

Figure 3. Left: vertical and horizontal crack:; right: concrete segregation zone

Apart from the remediation of the described superficial loads, it was decided to perform also the structural strengthening of the chimney core, in terms of ring reinforcement.

3. TEHNICAL REMEDIAL MEASURES DESIGN

3.1. Remediation of superficial damage

After an analysis of relevant techno-economic parameters, it was decided not to undertake any special interventions regarding the cracks up to 0.4 mm wide, so only the following procedures were performed on the damages observed on the external surface of the chimney core:

a. Rehabilitation of superficial damage (defects) such as concrete segregation, blistering of the surface layers of concrete due to the reinforcement corrosion, and remediation of spalling of thin layers of concrete along the rebars (which was a consequence of the reinforce corrosion and expansion of corrosion products) were done after prescribed preparatory works both on concrete and on reinforcement, applying appropriate mending plaster, a manufactured product, and previously applying the primer coating over the damaged place to create the bond between the old and new concrete.

b. Sealing of cracks with the width from 0,4mm to maximum 0,8mm applying appropriate epoxy resin injected in the previously grip-cut cracks.

c. Grouting of cracks-fissures with the widths larger than 0,8mm by an appropriate low-viscosity resin, with the previous preparation, i.e. working of the cracks.

3.2. Chimney core strengthening

<u>3.2.1. Adopted design</u>

Analyzing the relevant techno-economical parameters, it was concluded that it is possible to statically strengthen the chimney core implementing rings formed by carbon strips, which will be glued to the external concrete surface of the chimney. This design concept proved to be optimal during the rehabilitation works, and for it the carbon laminated strips *Sika CarboDur* S1214 were adopted, which necessitated the usage of the epoxy adhesive of the same manufacturer, and execution of a number of prescribed preparatory procedures regarding the concrete surface on which the strips were to be glued.

The decision to implement the carbon strips was, among other factors, based on the fact that the conducted thermal analysis demonstrated that even in the critical summer season service conditions, and in the cases when the of thermal insulation lining in the chimney would be damage, the temperature on the outer side of the concrete core would never exceed 60^{o}C. And that temperature represents the upper limit for the safe application of epoxy adhesive used for the adopted type of carbon strip *Sika CarboDur* S1214.

3.2.2. Temperature analysis

In order to obtain proofs that even in the most adverse case, the temperature on the external side of the concrete core of the chimney would never exceed 60°C, and also for the purpose of obtaining the input parameters for dimensioning of carbon strip rings which would encircle the structure on the external side, a thermal design with the following input data was made:

- internal temperature in the chimney: T_i = 190°C (this is the maximum value related to the lower zone of the chimney, while the upper zone can be assumed to have T_i ≈170°C);
- ambient temperature: Te = -10°C in winter regime, Te = 30°C in summer regime;
- $1/\alpha_i = 0{,}12$ ($m^2 \cdot {}^oC/W$);
- $1/\alpha_e = 0{,}04$ ($m^2 \cdot {}^oC/W$);
- for concrete λ= 2,00W/(m·°C).

With these parameters the temperatures in the chimney concrete core were obtained and presented in table 2 (see also figure 4).

Table 2. Temperatures in the chimney core

Lamela	**Temperature na betonskom zidu - slučaj neoštećene termoizolacije (°C)*)**			**Temperature na betonskom zidu u slučaju oštećene termoizolacije – letnji režim (°C)**				**Temperature na betonskom zidu u slučaju oštećene termoizolacije – zimski režim (°C)**			
	t_u	t_s	t_{sred}	t_u	t_s	Δt	t_{sred}	t_u	t_s	Δt	t_{sred}
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
I	32	11	21,5	113	56	57	84,5	94	22	72	58,0
II	32	11	21,5	113	56	57	84,5	94	22	72	58,0
III	32	11	21,5	113	56	57	84,5	94	22	72	58,0
IV	32	11	21,5	113	56	57	84,5	94	22	72	58,0
V	32	11	21,5	113	56	57	84,5	94	22	72	58,0
VI	34	10	22,0	119	55	64	87,0	98	21	77	59,5
VII	34	10	22,0	119	55	64	87,0	98	21	77	59,5
VIII	36	10	23,0	120	54	66	87,0	101	20	81	60,5
IX	39	10	24,5	126	53	73	89,5	104	19	85	61,5
X	39	10	24,5	126	53	73	89,5	104	19	85	61,5
XI	41	10	25,5	128	51	77	89,5	110	17	93	63,5
XII	44	9	26,5	128	51	77	89,5	113	16	97	64,5
XIII	44	9	26,5	128	51	77	89,5	113	16	97	64,5
XIV	47	9	56,0	134	49	85	91,5	119	14	105	66,5

*) Vrednosti iz GLAVNOG PROJEKTA ARMIRANOBETONSKOG DIMNJAKA TE "OBRENOVAC" (Zenica, juli 1967.)

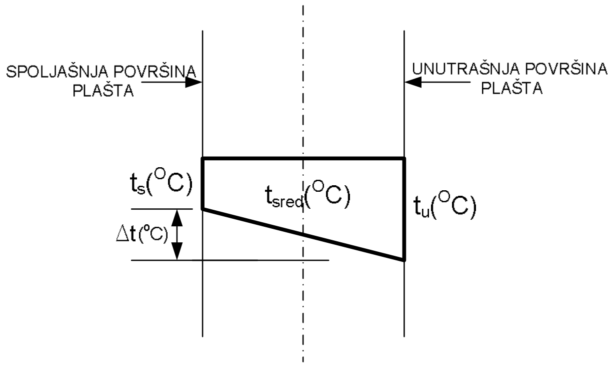

Figure 4. Explanation of temperature values displayed in table 2

The following conclusions were drawn on the basis of the analysis of temperature values displayed in the table 2:

- the highest temperatures on the exterior surface of the concrete core of the chimney (t_s) occur when thermal insulation is damaged in the summer regime, whereby those temperatures in all sections of the chimney are lower than 60°C (see the column (6) in table 2);

- the highest temperatures in the middle of the concrete core (t_{sred}) occur in the cases when thermal insulation is damaged in summer regime of chimney service (see column (8) in table 2); however, such case of temperature is not relevant from the point of view of the stress-strain state, that is from the aspect of designing the carbon strip rings which will encircle the chimney structure on the outer side, since for this case the case of damage of thermal insulation in winter regime of service is relevant, as the highest values of Δt are obtained (see the column (11) in table 2).

3.2.3. Temperature effects in the concrete core

The analysis of temperature effects was conducted applying the computer software TOWER 6 - RADIMPEX. The analysis was conducted assuming that in one of the sections according to the figure 1, there was a total devastation of internal thermal insulation – lining, while the other sections (those below and above) have the intact thermal insulation. This means that in one section, the temperatures presented in the columns (9), (10) and (12) of table 2 will be present, while in other sections the temperatures as in the columns (2), (3) and (4) will be present.

It is stressed here that in the further text, as characteristic and relevant, only the calculation results referring to the cases of total devastation of thermal insulation in the following sections are presented: section III (core thickness 18cm) , section VII (core thickness 20cm), section X (core thickness 25cm) and section XI (core thickness 27cm).

In the calculation of the reinforcement required to compensate for the effects of temperature, it was assumed that the concrete core was made of a concrete class MB 20, which is a safety providing assumption. However, for the part of calculation of cross section forces, the modulus of elasticity of concrete of 27GPa was taken (corresponding to the concrete class MB 15), and which is less than the modulus of elasticity for the concrete class MB 20. This assumption can be considered appropriate, regarding the cracked state of the core surface, which certainly reduced the structural stiffness in global terms.

Table 3. Required additional horizontal reinforcement in individual sections

SECTION NUMBER	**Core thickness (cm)**	$\mathbf{A_{a,potr}}$ **(cm²/m)**	$\mathbf{A_{a,post}}$ **(cm²/m)**	$\mathbf{\Delta A_a}$ **(cm²/m)**
Section III	18	29,73	3,92	25,81
Section VII	20	32,91	3,92	28,99
Section X	25	44,37	3,92	40,45
Section XI	27	50,00	5,32	44,68

In table 3 the results of calculation of the necessary quantities of horizontal – ring reinforcement required for compensation of temperature effects are presented. The calculation was performed according to the ultimate limit state, the results referring to the smooth reinforcement GA 240/360 used for construction of the chimney. Apart from that, in table 3 is presented the quantity of existing horizontal – ring reinforcement on whose basis the difference is obtained which should serve for the calculation of strengthening the chimney via horizontal rings made of carbon strips.

3.2.4. Calculation of reinforcement by the rings made of carbon strips

On the basis of the conducted calculation of the effects of temperature in the core, as well as on the basis of the values presented in table 3, carbon strips necessary for compensation of the missing horizontal – ring reinforcement in the core can be defined. As it has already been said, in relation to this, the laminated carbon strips *Sika CarboDur* S1214 were adopted, with the following characteristics:

- width b = 120mm,
- thickness d = 1.4mm,
- tensile strength 3100MPa,
- ultimate strain 1,8%,
- modulus of elasticity E = 165000MPa.

Ultimate bearing capacity of such a strip will be defined according to an approximate but fully acceptable procedure, so on these grounds it can be considered that the limit value of tensile force in one strip which will be glued on the surface of the chimney core amounts to

$$Z_u = \mathbf{0{,}8}\cdot 0{,}120\cdot 0{,}0014\cdot 3100000 \approx 417\text{kN}.$$

If it is assumed that across the width of the core of 1.0m "n" strips are glued, the ultimate limit value of tensile force in these strips will be

$$Z_{u,1} = n\cdot 417\ (\text{kN/m}).$$

Regarding that the strips in questions should be applied for compensation of the missing reinforcement GA 240/360 with the cross section surface of ΔA_a (cm^2/m), the limit value of the force to provide this compensation is

$$\Delta Z_{u,1} = 24{,}0\cdot\Delta A_a\ (\text{kN/m}).$$

Accordingly, the missing reinforcement will be compensated with the following number of strips:

$$\mathbf{n} = (24{,}0/417)\cdot\ \Delta A_a \approx 0{,}058\cdot\Delta A_a\ (\text{kom./m}),$$

which means that the strips in question should be glued at he axial distances

$$\lambda = 100/n = 100/0{,}058 \cdot \Delta A_a \approx 1724/\Delta A_a \text{ (cm)}.$$

On the basis of the previously derived expressions for the required number and carbon strips and their mutual distance, the following results are obtained:

<u>Section XI:</u> $\mathbf{n} = 0{,}058 \cdot 50{,}00 = 2{,}90$ pcs./m, $\boldsymbol{\lambda} = 100/2{,}90 \approx 35$cm.
<u>Section X:</u> $\mathbf{n} = 0{,}058 \cdot 44{,}37 = 2{,}57$ pcs./m, $\boldsymbol{\lambda} = 100/2{,}57 \approx 40$cm.
<u>Section VII:</u> $\mathbf{n} = 0{,}058 \cdot 32{,}91 = 1{,}91$ pcs./m, $\boldsymbol{\lambda} = 100/1{,}91 \approx 50$cm.
<u>Section III:</u> $\mathbf{n} = 0{,}058 \cdot 29{,}73 = 1{,}72$ pcs./m, $\boldsymbol{\lambda} = 100/1{,}72 \approx 60$cm.

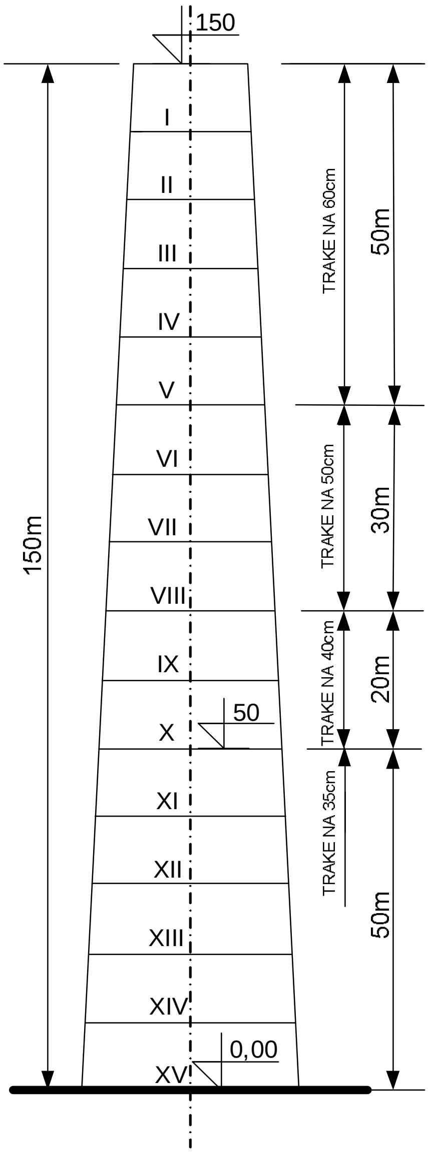

Figure 5. Distribution of carbon strips – rings along the chimney height

On the basis of the previously presented calculations, laminated carbon strips *Sika CarboDur* S1214 were adopted as elements for strengthening glued along the circumference of the chimney, at the mutual distance as in figure 5.

It is stressed here that construction of strengthening by carbon strips followed the completion of works on remediation of superficial damage on the chimney core, that is, after the interventions describe in item 3.1. (works under **a.**, **b.** and **c.**).

3.3. Decorative – protection coating over the exterior surface of the chimney core

As part of the works on the mentioned chimney, as a last phase of remedial intervention, a decorative – protective coating for concrete *SIKAGARD ELASTOKOLOR W* – vapor permeable membrane by *Sika*, was applied over the entire surface of the chimney. This protection was applied in two colors (red and white) alternating along the height of the chimney, this procedure being applied, among other reasons, for the purpose of sealing all the 0.04 mm cracks on the exterior surface of the chimney. In figure 6 there is an image of the appearance of the chimney after the remedial intervention, finishing in application of the decorative – protective coating.

Figure 6. Appearance of the rehabilitated chimney

4. EXECUTION OF REMEDIAL WORKS

The described remedial works of the chimney were executed by the company *Glečer d.o.o.* from Belgrade, specialized in works on high-rise structures. The works up to the height of cca 50m were done from the tubular scaffold mounted around the entire chimney, while above this height, the works were done using a special scaffold suspended from the top of the chimney. In this manner, using the given technology, an access to all the places where the chimney was rehabilitated was provided.

Figure 7. Left: appearance of the part of surface with the glued carbon strips; right: detail of the glued carbon strips, where one of the strip extensions is visible

In figure 7 – left – the appearance of the part of the surface with the glued carbon strips is presented, while in figure 7 – right, there is a detail of glued carbon strips with one of the strip extensions.

5. CARBON STRIP EXTENSIONS AND CONDUCTED EXPERIMENTAL TESTS

Regarding that the laminated carbon strips were mounted along the height of the chimney were placed in the form of rings, due to the great length of certain rings, they had to be constructed out of several segments (2 to 3). This means the for every ring, in order to extend the strips, there had to exist extension in the form of overlapping strips 100 cm long. Even though in the concrete case it was considered that the mentioned overlapping length was sufficient to transfer the tensile force, it was proposed by *Sika* – the manufacturer and supplier of materials for remedial works (carbon strips and appropriate adhesive) to fix the steel plates on both ends of designed strip overlappings as connectors using the HILTI HST M-12 anchor screws to fasten them to the concrete surface of the chimney. This suggestion was adopted, so each of the mentioned anchor screws (a total 4 pieces on each overlapping section) was secured using tightening torque of 7,5kN, so that the total tightening torque on each end of the overlapping section would be 15kN. In this manner, at the ends of overlapping sections, a certain pressure force on the strips was exerted by tightening the anchor screws, and this increased the safety of the constructed extension.

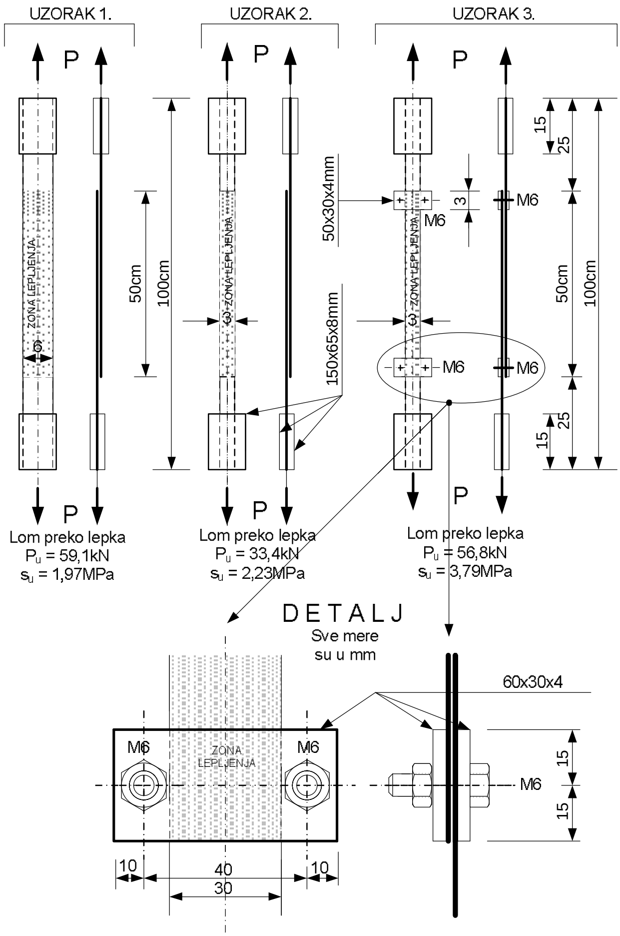

Figure 8. Testing of overlapping strip extensions

It is stressed here that in the places where two *Sika CarboDur* S1214 were extended by gluing by an epoxy adhesive, with the overlapping of 100cm, the average value of ultimate shear stress of that bond was

$$\tau_{u,tr} = 417/(1{,}00x0{,}12) = 3475kN/m^2 = 3{,}48MPa.$$

Despite the fact that the suggestion of *Sika* to reinforce the overlapping sections – extensions of carbon strips, at the Faculty of Civil Engineering of the University of

Belgrade – at the Institute for materials and structures (IMK) – laboratory tests were conducted in order to verify such design.

In relation to this, three samples presented in figure 8 were made and tested. Those are 30mm and 60mm wide samples cut out from the *Sika CarboDur* S1214 strip 12cm wide, which were exposed to the tensile forces P, whereby two samples (specimens 1 and 2) had only glued overlapping sections, while one sample (specimen 3), also was additionally reinforced with steel clamps.

Table 4 presents the results of conducted tests, whereby their analysis among other things, was based on the calculated value of the ultimate tensile force Z_u = 417kN which refers to the strip *Sika CarboDur* S1214 120mm wide. In case those are strips 60mm and 30mm wide, those forces will respectively be $Z_u \approx$ 208,5kN, and $Z_u \approx$104,25kN.

Table 4. Test results of the bearing capacity of extensions-connectors of carbon strips realized using glued overlapping bonds

Sample Number	Failure force of extension – connection P_u(kN)	Connection shear strength τ_u(MPa)	Force ratio P_u/Z_u	Failure type
1	59,1	1,97	0,283	Adhesive failure
2	33,4	2,23	0,320	Adhesive failure
3	56,8	3,79	0,545	Adhesive failure

On the basis of the test results displayed in table 4, it is obvious that by applying additional reinforcement in the form of steel clamps presented in figure 8, the bearing capacity of strips extended by overlapping is significantly increased. This increase is such, that in the concrete case, the tensile strength of the connection is τ_u = 3,79MPa, which is practically of the same order of magnitude as the ultimate shear stress $\tau_{u,tr}$ = 3,48MPa in the connection of the strips forming rings around the chimney in question.

The stresses τ_u at which the samples 1 and 2 failed are only conditionally accurate values, that are based on the assumption of even distribution of shear stress along the observed strip connection (bond). However, this distribution, as in case of the similar connections in steel structures (done by the rivets, bolts or welding) and in timber structures (done by nails, scres etc) is not uniform, but in principle corresponds to the case presented in figure 9. However, in case of steel and timber, and also the connecting devices used for these structures, those materials and elements are ductile, so this phenomenon has no particular importance in respect to the ultimate bearing capacity of the mentioned connections. Namely, due to the mentioned ductility, in the ultimate stress states cases, the external force P_u is always relatively evenly distributed on all the used connecting devices.

However, in case of the carbon strips and adhesives used for them, the ductility assumption is not valid. Namely, in both cases, the materials are not ductile, so in the bond presented in figure 9 enormously high stresses τ_{max} always occur at the ends of the strips, and there is no possibility to redistribute the stress from those stress points, along the bonded connection. In such cases, as soon in the narrow zones around the ends of the strips, in the zones of characteristic stress concentrations τ_{max} – the ultimate tensile strength of the adhesive used is reached, the entire connection fails.

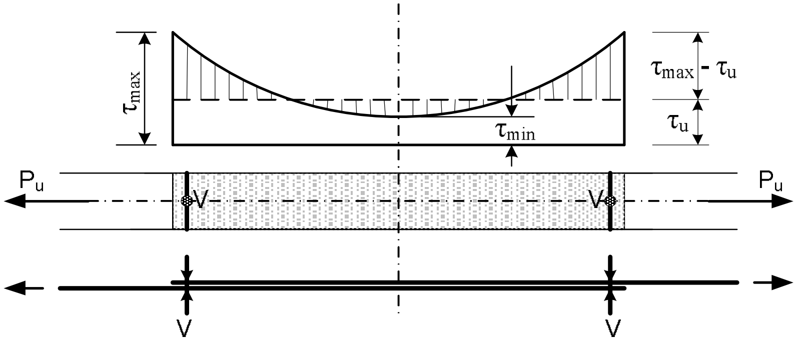

Figure 9. Shear stress distribution in the glued strips connections

When at the ends of the glued strips the reinforcements in the form of steel clamps are applied, as in the case of sample 3 presented in figure 8, and this means when in the zones of the ends of glued strips certain pressure forces V are introduced (see figure 9), those forces will in a certain way eliminate the present stress concentrations, so it can be reasonably considered that the shear stress will be relatively evenly distributed along the bond of glued strips. For this reason, in the given case 3 the obtained stress was τ_u = 3,79MPa, which is 3,79/2,23 ≈ 1,7 times higher than the stress obtained in the case 2 – without steel clamps.

SOCIETY FOR MATERIALS AND STRUCTURES TESTING OF SERBIA
DRUŠTVO ZA ISPITIVANJE I ISTRAŽIVANJE MATERIJALA I KONSTRUKCIJA SRBIJE

International Conference "**Marking 60 Years of Operation of DIMK**
Belgrade, October 19, 2012.

Radenko Pejović[1]

UDK:620.1:624.011/014

RESEARCHES IN THE AREA OF BUILDING STRUCTURES AT THE FACULTY OF CIVIL ENGINEERING IN PODGORICA

Summary: *The subject of this paper are the researches from the area of building structures, which are performed at the Faculty of Civil Engineering in Podgorica. Researches which are performed are from the areas: timber, steel and concrete structures. Although this Faculty is very young, it was founded thirty years ago, over 20 research projects have been realized there. All realized projects had large experimental and theoretical researches. Using the results of the performed researches, a large number of doctoral and master theses are prepared.*
A large number of scientific papers are published in the referent modern and domestic magazines as well as in the domestic and international conferences.
According to the mentioned researches, this Faculty began recognized in the region, as well as further. Especially we point out the good cooperation with the Faculty of Civil Engineering in Belgrade and the University from Granada in Spain.
In future, this Faculty plans to nourish research work and especially experimental work because there are real conditions for that.

Key words: *experimental researches, structures, timber structures, steel structures, concrete structures*

ISTRAŽIVANJA U OBLASTI GRAĐEVINSKIH KONSTRUKCIJA NA GRAĐEVINSKOM FAKULTETU U PODGORICI

Rezime: *Predmet ovog rada su istraživanja iz oblasti građevinskih konstrukcija, koja su obavljena na Građevinskom fakultetu u Podgorici. Obrađena su istraživanja iz oblasti drvenih, čeličnih i betonskih konstrukcija. Iako je ovaj fakultet relativno mlad, osnovan je prije tridesetak godina, na njemu je iz ove oblasti realizovano preko 20 naučnoistraživačkih projekata. Svi realizovani projektu su imali obimna eksperimentalna i teorijska istraživanja. Koristeći rezultate provedenih istraživanja urađen je veliki broj magistarskih teza i doktorskih disertacija.*
Takođe je objavljen veliki broj naučnih radova u referentnim svjetskim i domaćim časopisima kao i na domaćim i međunarodnim konferencijama.
Po navedenim istraživanjima ovaj Fakultet je postao prepoznatljiv u regionu, a i šire. Posebno ističemo dobru saradnju sa Građevinskim fakultetom u Beogradu i Univerzitetom iz Granade u Španiji.
I ubuduće ovaj fakultet planira da njeguje naučno-istraživački, a posebno eksperimentalni rad jer za to postoje uslovi.

Ključne riječi: *eksperimentalna istraživanja, konstrukcije, drvene konstrukcije, čelične konstrukcije, betonske konstrukcije*

[1] PhD full professor, Faculty of Civil Engineering Podgorica, mob: ++382 69 012 802, e-mail: radenko@ac.me

1. INTRODUCTION

The Faculty of Civil Engineering in Podgorica is relatively young. It was founded in 1980. Beside teaching-educational processes at the Faculty, research work is also nourished. Beside very difficult material conditions, over 20 larger experiments have been realized. According to the performed experimental researches the Faculty is recognized in the region and further. Using the results of the received experimental researches, a lot of doctoral and master theses are prepared at the Faculty.

The performed researches refer to building materials and structures. The subject of this paper is the researches from building structures.

In this paper, researches from the area of timber, steel and concrete structures are performed. One part of researches from the area of concrete and timber structures was performed at the Faculty of Civil Engineering in Sarajevo. These researches are presented in this paper, because on of the researcher is the author of this paper, and now he is the professor at this Faculty.

Figure 1. Laboratory of the Faculty of Civil Engineering

2. TIMBER STRUCTURES

At the Institute for materials and structures of the Faculty of Civil Engineering in Sarajevo 1984. the researches of engineering structures were performed, made of lignified wood, prestressed with the aim of rationalization during the high load and spans. Researches were performed for the needs of Institution “Krivaja” from Zavidovići. Researches were performed by Ladislav Lesić, Krivaja Zavidovići, Davorin Lončarević, mr Srđan Kisin and Radenko Pejović, Faculty of Civil Engineering from Sarajevo.

This investigation was the pioneer intake because, the prestressed wood was performed for the first time in ex-Yugoslavia. Researches were performed in two phases.

The first phase of investigation includes the testing of bonded lignified girders without prestressing of the dimensions 240x1000x16000 mm. Three girders of the system of the simple girder were tested, which were loaded by the concentrated forces in the third spans which was L=15750 mm. During the testing, the deformations were measured for each phase of load up to the girder fracture.

The second phase of researches includes the testing of two prestressed girder, made of bonded laminated wood. The dimensions of the girder, the way of load, number and spacing of measured spots, that is instruments, remain the same as in the first phase of researches.

Testing of prestressed girders was divided in three phases. In the first phase, the measuring of deformations from the cable effects was performed. The second phase includes the time following of effects of cable relaxations, wood creeping and effects of the proper girder weight. The third phase is identical to the girder testing without prestressing, that is it includes the testing under the test load up to the fracture.

Technology of production of lignified girders for the second phase of research differs because of the cable placing for prestressing. These girders are done from two parts of the dimensions 120x1000x1600 mm where notches were placed for cable incorporation.

In these notches there are deformed plastic pipes and the jointing of the girder parts are done by sticking and bolting. The spacing of cables is given in the figure 2.

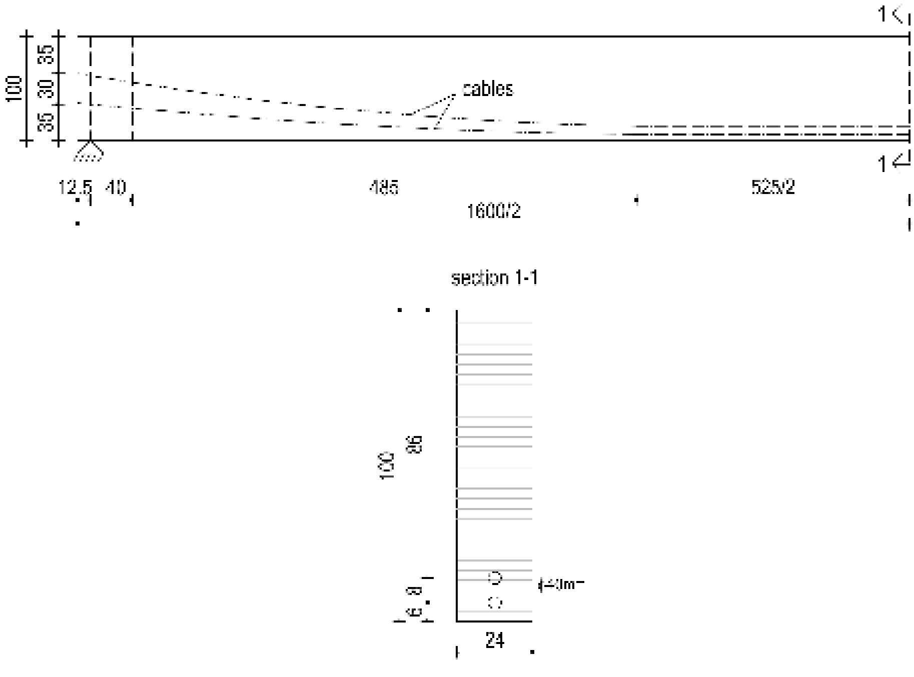

Figure 2. Glue-lam girder

The bolts had the structural intention, that is, to make impossible the separation of the girders part along the bolded joints and especially on the spots of fracture of the cables line, that is, appearance of deviated forces. During the choice of the distribution and the number of bolts, it took care about that. Beside that, at the ends of girders, on the place of cables, because of the large local stresses of wood, the special insurance was done, made of steel profiles and tins, which are also jointed by the bolts.

In order to obtain the correct adhesiveness of the steel insurance on the girder front, on the place of cable heads, partially free space among steel insurance and the girder head is filled up by the resin on the base of epoxide. In this way, the local overload and wood compression are avoided, as well as the external transmission of the stretching force to the girder.

Prestressing of girder was performed according to the system "IMS" with two cables which nominal force is 600 kN (60 Mp). Each cable was consisted of per 12 wires ø7mm, steel quality was $s_m/s_{0,2}$= 1700/1500 N/mm^2.

During the testing, the deformations were measured in the wood by cables and the flexion of girders in order to analyze their stress-deformed state in details. The spacing of measured spots and the way of testing was the same in both girders, which made possible to do the correct comparative analysis of the test results. In that way the economical analysis was done. All mentioned made possible to come to the adequate conclusions regarding the rationalization of the previous stress.

According to the performed researches, it is surely established characteristic increases at the prestressed girders and they are:

For failure force according to the experimental research 64%;

– For permitted force according to the permitted stress on the base of the calculated analysis 31%;

– For the permitted force according to the permitted flexion, on the base of the calculated analysis 103%.

At the same time it is stated the approximate flexion reduction at the prestressed girder for 52%.

Comparison of girders with and without prestressed stresses is completed parallel by the economical analysis. The basic results of this analysis are:

– Girders to the span 20 m are more expensive than the girders without prestressing;

– Prestressed girders of the spans more than 20 m are more economical with the tendency of more savings during the further increase of spans (exp. for L=30 m the savings are 34%, and for L=40 m the savings are 40%).

With the respect to the presented techniques and economical indicators, it can be generally concluded that the girders, made of lignified wood, prestressed, with the aim of rationalization, have the advantage in regard to the girders, without prestressing for spans more than 20 m.

3. STEEL STRUCTURES

3.1. Introduction

Very large researches from steel structures have been performed at the Faculty of Civil Engineering in Podgorica, beginning from 1995. The subject of researches has been, so called, thin-wall I profile loaded by close-divided or concentrated load. This load means divided load which acts locally on the small part of area or along some structural element.

It is very interesting when such defined load acts along the upper edge of I profile, so the web, under the load, is locally compressed. It is about very complex and provocatively problem, which have paid attention of many researchers. The problem of the ultimate bearing capacity is very interesting – as it originated, under which conditions, which parameters have decisively influence. Namely, it is about the problem of much expressed elastic-plastic stresses and deformations, beside that, it can be seen the geometrical nonlinearity on the loaded elements during the least forces. In the figure 3., characteristic cases are presented when I profile is loaded by close-divided load.

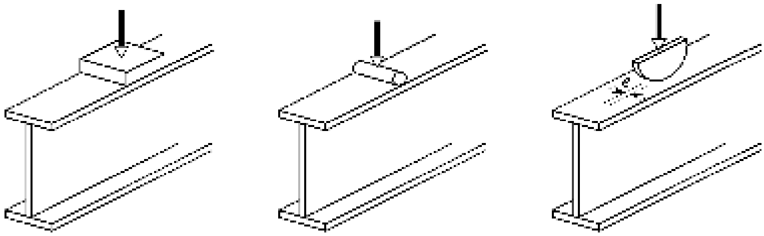

Figure 3. I profile is loaded by close-divided load.

Under the load, in the web, locally, stresses and deformations appear in the vertical direction. These stresses have no influence on the global stress – deformation state in the structure and they are not taken into account in the linear elastic theory of girder bending. The given relations in the theory of girder bending are valid, if they are considered on the enough distance from the place of the load effect (Saint-Venant's principle). However, regarding the ultimate bearing capacity, these stresses have the decisively role, because they can cause the local instability of elements, and at the same time, very often, the collapse of the whole structure.

The load can increase to the value, which many factors have influence on. In the literature the classification is the most frequent on three groups of the ultimate cases. Frequent names for these groups of limit states are: yielding, buckling and crippling (fig.4.). Yielding, as a limit state, appears in cases with the thicker web, when it can be seen quite web squashing and flange bending much earlier than the collapse load is reached. This phenomenon is especially expressed if web is not only thick, but also has small slenderness. For bigger web slenderness, characteristic form of bearing capacity loss is buckling. In these cases, the limit state is characterised by the web buckle, which is obvious along the web height. Finally, for the cases of very slender girders, the buckle appears only in the zone of load application, while the large part of the web remains, at sight, non-deformed even after reaching the collapse load.

Beside two basic parameters: web thickness and web slenderness, several parameters contribute to the form and the character of the buckle. They are: the flange stiffness, the relation of the flange thickness towards the web thickness, the width of load application, the spacing between the vertical stiffeners, the position of horizontal stiffeners, initial geometrical imperfections, load eccentricity etc. The level of the collapse load is mostly depended on the web thickness. It is shown that dependence is approximately close to the square of the web thickness. Other parameters which have influence on the form and character of the limit state, have also influence on the collapse load, but much less than the web thickness. The web slenderness, almost have no influence on the intensity of the collapse load.

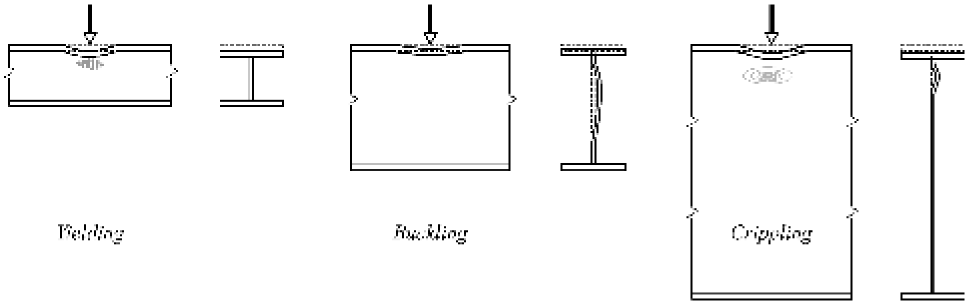

Figure 4. Characteristic limit states

In the engineering practice, this problem is presented. It is very frequent cases when some I profile is loaded by the close-divided load: the main one, loaded by secondary elements (binding rafters rested on the roof trusses); booms, loaded by the reactions of roof fastenings, boom or sheeting rail loaded by the columns reactions; non-stiffened edges of girders loaded by the supporting reactions; the structure column loaded (over the rails) by the crane wheels; the main bridge girders in cases when the assembly is performed by hauling; longitudinal and transversal girders of the road structure which are loaded indirectly (over orthotropic or composite road plate) by the vehicle weight; transversal girders above the support in case of the bridge raising (revision or overhaul of steel structure).

According to the concept, rationality and economy in designing of steel structures ask from the designer to obtain, regarding the possible stability loses (globally and locally), that each structural element sustains the stresses which are closer to the material endurance. Concretely in this case, when the engineer in practice finds the problem of entering of strong concentrated load in the structure, it is mostly solved by the simple web stiffening under the place of concentrated load effects. In this way local stresses are transmitted in vertical direction beside the web and also on the stiffener plates, and the stiffener plates by themselves obtain the web from possible buckling. At very slender girders, the only small part of the web is critical under the place of the force effects, so it is enough to place the stiffnesses only in that zone.

However, the tendencies of the modern designing go toward the omission of the stiffness of the vertical tin, where it is possible. The placing of stiffness disorders the process of automation of the production of the structural elements, which increase the spending of working hours and the price of products. Also, by placing the stiffener, the areas are increased which should be protected anti-corrosively. In cases of live load (crane paths, bridge assembly by hauling), it is impossible to obtain each critical spot by the vertical stiffener.

In any case, resistance of some structural elements on the effects of concentrated or close-divided load must be famous.

Five research projects are done from this area, which are shortly commented in the further text.

For these researches in the Faculty lab, special plant is made which obtained prescribed applying of adequate loads and precise deformations measuring of the samples during the tests.

3.2. Contribution to the stability analysis of thin-wall girders

These researches were performed during the preparation of the doctoral dissertation of mr Duško Lučić, grad.civ.eng.

Mentors for the preparation of this dissertation were academician prof. dr Nikola Hajdin, and prof. dr Branislav Ćorić from the Faculty of Civil Engineering from Belgrade.

In this paper, the problem is analyzed when thin wall I profile is loaded by close-divided and concentrated load. The paper is conceived that in the first part all more important researches are presented about these issues. In the second part of the paper, the most important results are presented of the researches themselves.

The analyzed problem is very current, in the last forty years. It is about very complex stress-deformation state of the local character where the collapse acts locally. It is about very complex stress-deformation state of the local character where the collapse acts locally.

Large number researchers tried to explain this problem in the whole mathematically, and to develop the procedure for the calculation of the force fracture. The problem is very complex. Geometrical imperfections causes that the girders often behave differently from case to case with the presented geometrical nonlinearity from the very beginning of the force apply. The girder fracture appears by the propagation of the web chamber, which is specific by the lines of plastification, so very complex elastic-plastic state is presented in the girder. From these reasons, it is very difficult to obtain all parameters, which acts to the girders behavior and on the fracture force itself, by one unique mathematical model. The suggested expressions are mostly empirical or semi-empirical character and they are rested on large number of experimental researches. Through more than 30 up to now finished experiments, rich experience is received about the girder behavior up to fracture. Very complex analyses are done where it is concluded how particular parameters have influence on the girder behavior under the load.

Although, a large number of researchers worked on this field, in mathematical modeling of the limit load, also on the experimental analysis, it can not be said that the problem is totally solved. Several problems remained, which ask the further analyses. These were the belonging reasons for starting the realization of this research.

The research is divided in two parts. The first experimental part and the second one where the work started on making the new mathematical procedure for calculating the fracture force.

Experimental research, generally, is divided in two phases, with two groups of girders which are tested. The first group of twelve girders is divided in two series of tests, while there were six girders in the second group, which were tested through the third series of testing.

The first two series are tested with centrally placed load in relation to the web plane, while in the third series the girders are loaded with certain eccentricity.

The aim of investigation in the first and second series was, to get clearer picture about the deformation, with the stress states in the girder, fracture form, and the height of the limit load through the variation of the web thickness and the thickness (stiffness) of the flange. In the preparatory research it can be seen that the chamber form on the web and its size are not expected, also the doubt about the flange plastification remain very interesting issue. These are the most important reasons why the test was performed in the first two series. The thickness of the web and the thickness of the flange are chosen, because it is famous that they have influence on the fracture form and the limit bearing capacity.

In the group of twelve girders, the web thickness was changed four times, and for each web thickness, the flange thickness was changed three times. The first series is consisted of twelve tests up to fracture. In this series, the flections of the loaded flange were measured, then the flections of the web chamber and also the height of the applied load. In the second series the girders are turned over and the tests are repeated by applying the load along the second flange. In this series, the deformations of the flange and web were measured in characteristic spots.

The second series is conceived on the base of the experience from the preparatory research which confirms the suggestion that the form of the web chamber and the form of the deformation line on the flange will be the same when the experiment is repeated twice with the girders of the same characteristics. From this reason, the experiment is repeated through researches in the second series, and the measuring tapes are arranged in the second series, according to the web and on the flange. It was possible to register deformations on the most interesting spots – correctly along the lines of plastification.

It was considered that in this way it got the most real picture about the stretching in the web and in the flange. By bonding of measuring tapes on the same place with both parts of the tin and in the same direction, it is possible to separate membrane deformations from the deformation bending.

The third series were planned so the testing was applied on three different girders with four changes of eccentricity of the applied load. According to that, three times per two girders are done. On two girders of the same characteristics, by loading on one, and on the other side, it is possible to test with four variations of eccentricity. For each of these three girders there was a girder of the same characteristics in the series I and II, which had a function as comparative, centrally loaded girder. Characteristics of the tested girders are chosen in a way, that in analysis there are different relations of the web thickness towards the flange stiffness.

Dimensions of the tested panels are b x d_w=700 x 700 mm, while the flange width in all cases is b_f = 150 mm (fig.5.).

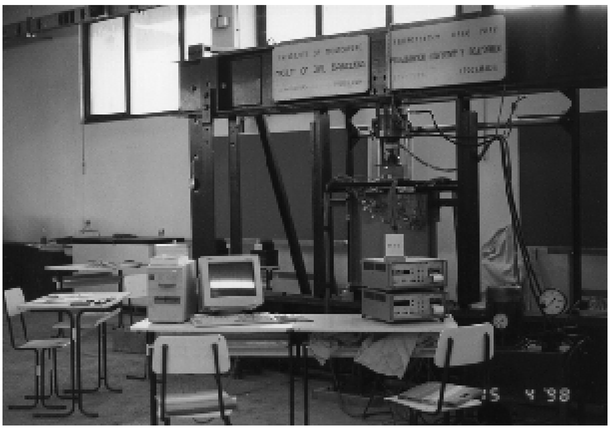

Figure 5. Fragment from experimental researches by D. Lučić

Experimental part is organized with the aim to get the response to some questions which are not solved for the further researches. Before all, very little, almost nothing was analyzed at the stress state in the loaded flange. The question about the plastification was asked in the flange, plastic joints – if they are formed and if they are formed where it is? Also, there was a question, if in the flange, regarding the local fracture, so called tensile field is formulated? The attention was especially paid to the formulation of the tensile fields in the web, along the lines of plastification. At the end, the parameter which is quite little examined, and which is very interesting from the engineering point of view, is the eccentricity of load in relation to the web plane. Some of these questions are answered, through the tests, while some other are opened more and they require the further analyses.

In the second part the mathematical model is suggested for the calculation of the fracture force at centrically loaded girders. The problem is asked in some unusual way, and what is the consequence of the experience, gained on the experimental research. In the foundation, the model is rested on the adopted fracture mechanism. However, the model is not still defined and it asks harder working in the further improvement.

Procedures for the calculation of the fracture force, beside the whole inclusion of all parameters acting the fracture force, should have more simple form, in order to be applied in the engineering practice. The aim is to check the insurance of some structural element very fast under the effects of concentrated load. Regarding the technical regularity, it is said that Robert's procedure is accepted as the best and it is introduced in Eurocode 3, Part I. However, this procedure can not be accepted as final.

Before all, some corrections of the fracture force are necessary, regarding the parameters which are not included by mathematical modeling, and they are: global stresses of bending, horizontal buckling, starting geometrical imperfections, and eccentricity of applied load in relation to the web plane. From the results of this paper it can be seen that the fracture at the girder with the eccentric load is applied in some different way. There is a question if it will be possible to solve this problem with some corrective factor in the satisfied way, or it is necessary to develop the special procedure of correctness for these cases. According to Eurocode, for the cases of thicker webs, where considerable creeping is presented in the web, before the final fracture, it is predicted the special procedure for the calculation of the limit force. It should mention that in these cases, from the practical cases, usage is of better interest, because of large subsidence of flange under the load. Our regulations predict the checking of elements on the effect of concentrated load.(175), in one part of standard which prescribe the checking of tin buckling. It is about very fast and simple procedure, which is based on the new cognitions from this area. However, revision of these regulations is also necessary, regarding the mentioned.

This research is not finished, because for the need for more qualitative mathematical model for the calculation of the fracture force, and also for the new problems which appear after the performed experimental research.

Main directions of effects in the future will be to modify empirical expressions in the mathematical model according to the form and essentially in the statement of their meanings. Also, the procedure for calculation of the fracture force must be much simpler.

3.3. BUCKLING OF SHEET GIRDERS UNDER THE INFLUENCE OF LOCAL LOADS

Bearer authority: Faculty of Civil Engineering in Podgorica 2003
Ordering authority" Faculty of Civil Engineering, University of Belgrade
Head of project: Nenad MARKOVIĆ, MSc in civ.eng.
Project Coordinator: Prof. Duško LUČIĆ, PhD in civ.eng.
Researchers: Nenad MARKOVIĆ, MSc in civ.eng.

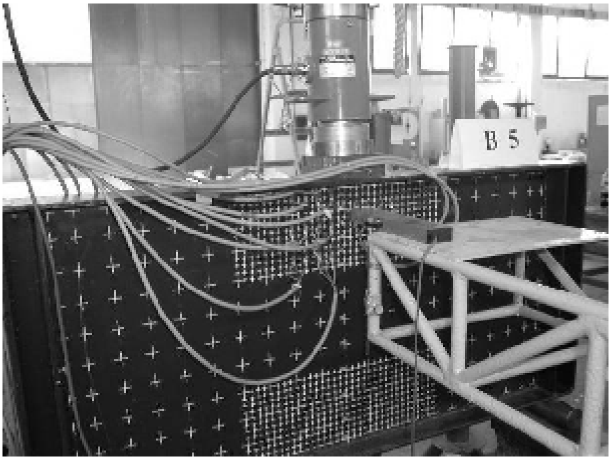

Figure 6. Fragment from experimental researches by N. Marković

Within the cooperation with colleagues from the Faculty of Civil Engineering in Belgrade on the study ‘patch loading’ problems, experimental part of research has been done in the laboratory of the Faculty of Civil Engineering in Podgorica.

In this paper it was studied the occurrence of buckling in the zone of entry loads on sheet girders (welded steel I beams) under the influence of localized loads in the plane of the ribs on a belt, when there is no transverse (vertical) stiffening of the rib at the intersection, but there is a longitudinal (horizontal) stiffening of the rib near the loaded belt.

The main aim of this work as also experimental research is the monitoring of the influence of distributed loads on the ultimate bearing capacity, when there are longitudinal stiffeners in the zone of entry load, but the comparison with the case when there is no longitudinal stiffeners is also done.

Two series of I girders were tested: major series A - the range of 500 mm with 14 tests and series B - the range of 1000 mm with 6 tests. There are girders without longitudinal stiffeners in both series and with longitudinal stiffeners on the distance from the loaded foot equal to the fifth of the rib height.

3.4. THIN-WALLED STEEL GIRDERS UNDER THE INFLUENCE OF CLOSELY DISTRIBUTED LOAD

Bearer authority: Faculty of Civil Engineering in Podgorica 2005/2007
Ordering authority: REPUBLIC OF MONTENEGRO MINISTRY OF EDUCATION AND SCIENCE
Head of project: Prof. Duško LUČIĆ, PhD in civ.eng.
Researchers: Prof. Duško Lučić, PhD in civ.eng.,
Biljana Šćepanović, MSc in civ.eng.,
Olga Mijušković, MSc in civ.eng.,
Srđa Aleksić, MSc in civ.eng.,
Milivoje Rogač, BSc in civ.eng.

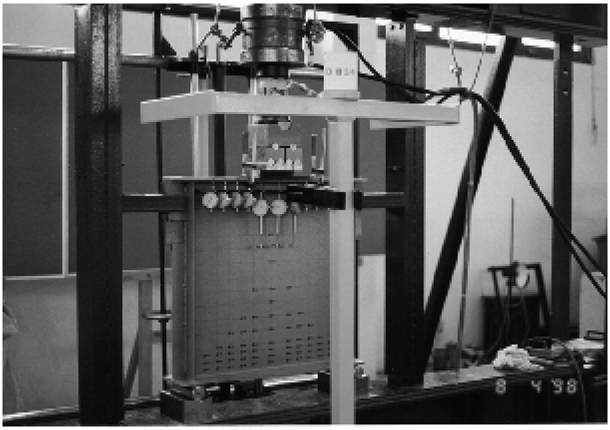

Figure 7. Fragment from experimental researches by D. Lučić

In this scientific research project it is collected more sub-projects related to research in the area of 'patch loading' or behavior problems and loss of capacity of thin-walled I profiles under the influence of closely distributed load. Included researches are: Excentro 2001, Numerical analysis (in 2002), Mathematical model for calculating the breaking load (2005), Calculation of the breaking load of eccentrically loaded girders (2005).

'Excentro 2001' is a continuation of research from 1998. Unlike the first study, a new experiment treats only the behavior of eccentrically loaded girders. Throughout four series 4 x 6 = 24 tests were done to the break of girders. In the first, second and fourth series deflections of the foot and distortion of ribs were measured, as well as the load level. In the third series, in addition to the forces, deflection of the foot and rib were measured with the help of measuring tapes.

Numerical analysis (2002). After the experimental research it was approached to the modeling problem, i.e. tested samples, by the finite element method. The idea was to gain, by comparing of experimental and theoretical results, more comprehensive, detailed picture of the stress-deflective state of the girders. The problem was analyzed in the linear region, which is only a first step for possible future theoretical analysis of problems in the nonlinear domain.

A mathematical model for calculating the breaking load (2005). On the basis of new experimental findings it was proposed the improved method for calculating the breaking load of thin-walled steel girders loaded by closely distributed load in the ribs plane. The mathematical procedure, in this case, relied on the upper limit theorem of plasticity theory (cinematic theorem) and the presumed mechanism of the break. Among the thirty, today in the world known procedures for the analysis of the breaking load, this mathematical model shows the best compliance with the results of experimental researches.

The calculation of the breaking load of eccentrically loaded girders (2005). In the case of an eccentric form of break, it the reduction of the breaking load with increasing eccentricity is obvious. One approach to the problem of determining the breaking load is quantification of this reduction by the reduction factor ***R***, which is the ratio of breaking force of eccentrically and centrally loaded girders.

3.5. EXPERIMENTAL-THEORETICAL ANALYSIS OF ECCENTRICALLY LOADED THIN-WALLED STEEL GIRDERS "EXCENTRO 2007"

Bearer authority: Faculty of Civil Engineering in Podgorica 2007/2010
Ordering authority: Faculty of Civil Engineering in Podgorica
Head of project: Prof. Duško LUČIĆ, PhD in civ.eng.
Researchers: Biljana Šćepanović, MSc in civ.eng., Srđa Aleksić, MSc in civ.eng., Milivoje Rogač, MSc in civ.eng., Mirjana Lakićević, MSc in civ.eng.

Figure 8. Fragment from experimental researches by D. Lučić

In preparation for the following experimental and theoretical research, the optimization and the modeling of the experiment has been done, using artificial neural networks. Depending on the input parameters (girders size and eccentricity of the load), it was predicted marginally load as only output data. Artificial neural networks were created of different structure, trained on data from experimental studies in 1998 and 2001.

’Excentro 2007’ was organized in 17 series (EB V, EB VI, EB VII ... EB XXI), each with six girders. Total amount of 17 × 6 = 102 carriers were tested.

In accordance with plans and expectations, various forms of break on eccentrically local loaded girders were registered: the breaking form typical for eccentric loading, but also breaking form typical for centric loading, as also mixed breaking form, which combines the previous two forms.

The results of nonlinear analysis of the underlying problem by finite elements method (University of Granada, 2006-07), and the results of experimental studies 'Excentro 2007' (University of Montenegro, 2007) do not match with the previous offered expression for calculation of breaking load and points to the necessity of reconsideration and modification of expressions for reduction factor ***R***. Of the four developed calculation procedure separates the one that gives the best results, i.e. it has the greatest compliance with the results of the experiment. Unlike all previous expressions with a linear connection of factor reduction and eccentricity, it is now considered a square factor ***R*** dependence on the relative eccentricity e/b_f .

3.6. EXPERIMENTAL-THEORETICAL ANALYSIS OF CIRCULAR LOADED THIN-WALLED STEEL GIRDERS – "CENTRO 2009"

Bearer authority: Faculty of Civil Engineering in Podgorica, 2009/2010
Ordering authority: Faculty of Civil Engineering in Podgorica
Head of project: Prof. Duško LUČIĆ, PhD in civ.eng.
Researchers: Srđa Aleksić MSc in civ.eng., Biljana Šćepanović, MSc in civ.eng., Milivoje Rogač, MSc in civ.eng., Mirjana Lakićević, MSc in civ.eng.

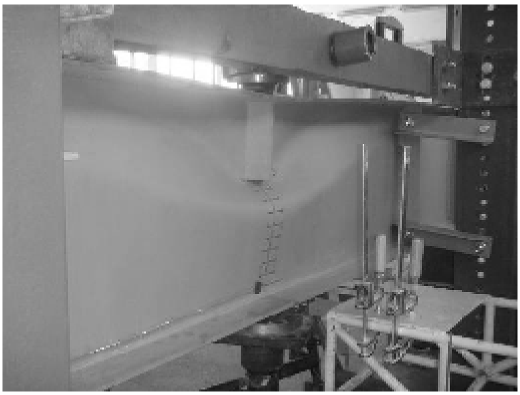

Figure 9. Fragment from experimental researches by D. Lučić

The purpose of this experimental study is to extend so far acquired knowledge about the influence of normal stresses arising due to global moments of bending to the level of ultimate load.

Experimental research 'Centro 2009' was conducted on twenty girders. Twenty-four tests were carried out through four series of five girders, namely six tests. The girders are divided into four series, classified according to the nominal thickness of the ribs.

The conclusion is that the global normal stresses caused by the moments of bending, which occur as a consequence of loading the girders, do not affect the intensity of the breaking load as long as they cause the stress in the girder up to about 80% of the stress on the direction limit of the rib.

In an attempt to solve the breaking load of 'patch loading' problem the linear bending theory was abandoned some forty years ago. Experiments have shown that the breaking force depended approximately on the square of the thickness of the ribs, while the critical bending force depended on the thickness of the ribs graded to the cube. However, the experience that was slowly acquired from a great number of experimental data that we collected in our laboratory, have led to rethinking the application of a simple Euler bending force, slightly modified and calibrated by the results from the experiments. The method for calculating the breaking load has been developed, which took over the lead from the preceding one, so that among the thirty proposed procedures in the world this procedure most accurately predicts fracture force and has the smallest statistical discrepancies.

4. CONCRETE STRUCTURES

4.1. Introduction

Beginning from the foundation of the Faculty of Civil Engineering the largest researches have been performed from the concrete structures.
These researches included the following aspects of the concrete structures:

– Time behavior of certain types of concrete structures during the long-term effects;
– Seismic resistance of the structures;
– Behavior of reinforced concrete girders in the area of limit states during the reduction of transversal forces;
– Development of the analysis methods for the deigning of high RC beams;
– Some aspects of the application of the concrete of high strength;
– Models and methods for the study of thermal stresses and prediction of the concrete fissures.

The part of the most important researches will be presented in this paper.

4.2. Analysis of the bearing capacity of the bridge systems composite by the concrete of different ages

These researches were performed at the Faculty of Civil Engineering in Sarajevo 1998-89. in the scope of doctoral dissertation by mr Radenko Pejović, grad.civ.eng., the author of this paper.

In this paper the general case of composite of two concrete is analyzed, the one of them is prefabricated and prestressed and which is strengthened by the new concrete in certain ages. Both concretes include certain percentage of the steel for reinforcement.

The estimate of such composite sections, consisting of two concretes of different ages and different deformation features, of high-valued steel for prestressing and non-tensile reinforced steel, is very compound regarding the very complex high-elastic deformation of concrete behavior which depends on time.

Beside the complexness of the section which is analyzed, the calculated inclusion of the strike and contraction effects in the bearing structure is difficult because there is no constant but variable stresses during the time.

For the exploitation stresses which appear in the concrete and for the other conditions where a large number of structures are, it is possible to adopt, with the good correctness, linear dependence of the deformations of creep than the constant stress and the principle of superposition for the creep deformations. These suggestions lead to the integral connection between dilatations and stresses.

The solution of the complex composite sections by the integral connection between dilatations and stresses is very difficult and practically possible only by the numerical integration step by step.

In the engineering practice for the calculation of the complex composite girders the simple algebra connections are used between stresses and dilatations, which are used according to the approximately theories. One of the usual used simple connections in the international engineering practice is the algebra connection between the stresses and dilatations which are formulated by Trost 1967. and Bažant 1972.

Beside the analysis of the time behavior of the composite section during the long-term effects, there is a serious issue about the state of stress and deformations of this section during the short-effects.

As a special problem there is an issue of the transversal load distribution. Because for the practical reasons, except on the support, there are no transversal girders, the plate has a function of the transversal distribution. Regarding the fact that the plate lies with its total area on the main girders, the way of the calculation is very complex. The good choice of the plate thickness, that is the relation of the girder stiffness and the plate has a lot influence on the rationality of the designed structure.

Experimental researches were done on the girders of the prestressed concrete which are designed like the girders for the span structures of the assembly-monolithic bridges of the arbitrary width and span of 6 to 20 m, tim “Put-Traser-MP 6-20”.

The system is analyzed in the Designing institution for roads and bridges “Traser” from Sarajevo, and the girders are produced by the Engineering working organization “Put” from Sarajevo. Prestressed girders is adhesious.

The testing was performed on the girders of the type T-40, of the length 6 m, in the Institution for the materials and structures of the Faculty of Civil Engineering from Sarajevo (fig.10).

Beside the testing on the girders, the finished structures - bridges were tested, in the scope of the obligatory tests of the new-built bridges.

The program of the testing is made with the goal to get the insight to the behavior of the structured systems which are got by the composite of the concrete of the different ages during the different phases of exploitations and degrees of load up to the fracture.

For the testing of the behavior during the long-term effects (the dead load and prestressed stresses) the one prestressed girder was performed with the belonged part of the plate and it was examined like the linear element. Regarding the uniform disposition of the constant effects, this treatment can be adopted as correct. This treatment neglects transversal contraction of the plate which effects to the total system behavior are negligible.

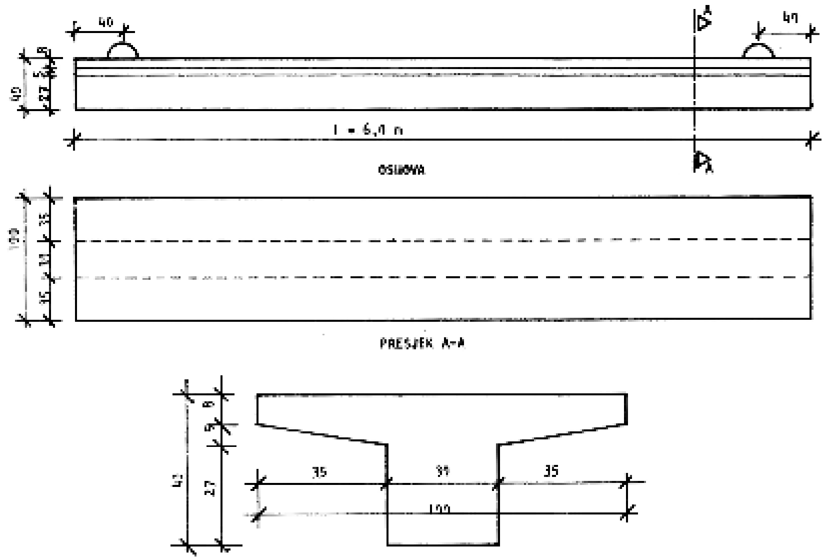

Figure 10. Girder type T-40

Four girders are done for the investigation of the type T-40, length 6 m. Two of four girders are done according to the design, according to which the bracing of the girder and plate is done by dropping the jaw from the girder. In order to examine the contribution of the outfall reinforcement to the bracing, the rested two girders are done without reinforcement for bracing, that is the only connection was the direct adhesiveness of old and new concrete.

The time following of the behavior of the composite girder was done up to the oldness of the concrete girder of 227 days. After that the statical testing of the girder was done in two phases. In the first phase, the load is distributed in steps of ¼ exploited load, and then after the removal, in steps, up to the girder fissure (fig. 11).

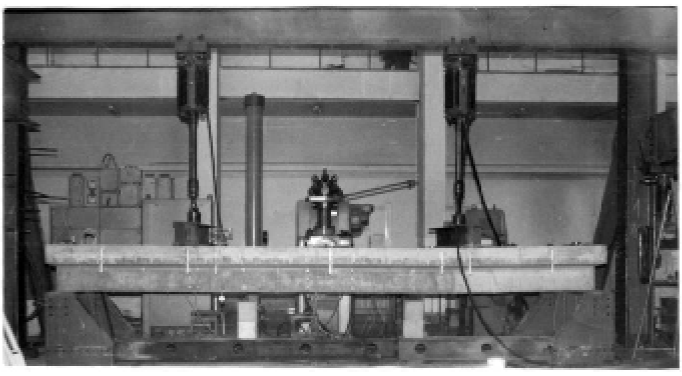

Figure 11. Static testing of girder

The structure behavior during the short-term effects (live load) was tested on all new structures in the scope of the obligatory tests. Characteristic positions of load were analyzed for getting the real insight into the structure behavior and the transversal distribution of load.

In order to establish the real deformation characteristics of the basic materials (concrete, steel for reinforcement and high-valued steel for prestressing) which are necessary for the production of experimental and theoretical tests, the lab tests of these materials were performed.

The performed tests were real, because the new comprehension in the behavior of the composite girder was realized during the permanent and short effects.

According to the time tests of the model behavior, as well as to the realized numerical and calculated analyses of stresses and dilatations, the real picture was got about the changes of stress and dilatations during the time. Experimental results did not give the obvious differences in the girder behavior with the reinforcement and without reinforcement for bracing. In certain sense it can get the small advantage to the girders with the reinforcement for bracing. Large stresses should be pointed out here, in the steel for reinforcement on the cable levels which, get over the limit of stretching and they are the consequence of relatively large normal stresses on that place and redistribution of stresses between the concrete and reinforcement.

Experimental results and calculated analyses of the time behavior confirm that it happens the activation of composite section during the time without the external effects that is it comes to the stresses redistribution.

Numerical solution for the time dependant stresses and dilatations shows the good coordination with the experimental results and it can be adopted like the correct solution.

According to the experimental results of the girder testing during the statical effects, it comes to the significant cognitions about the behavior of the composite section during the short-term effects. According to the detailed analysis of these results it comes to the conclusion that in both cases of bracing it neither works the hypothesis of the dilatation compatibility on the joint of two concrete of different ages and different mechanical features, and at the same time nor the Bernul's hypothesis of the even deformed section. It clearly sees discontinuity in dilatations in the joint spot, from the diagram of dilatations changes according to the section height (fig. 12.). Beside that, the measured deformation values are higher than the calculated ones with the suggestion of the total composite, which also confirms the conclusion. If it requires the more correct calculated analysis, it is necessary to include the mentioned effects.

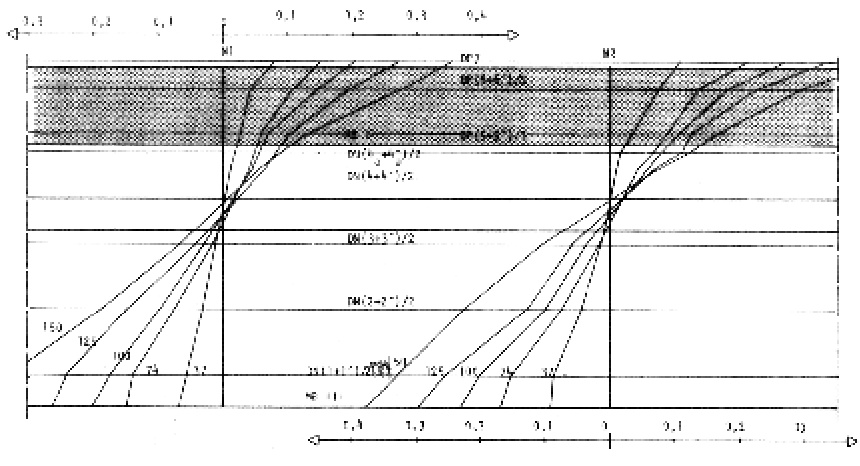

Figure 12. Diagrams of dilatation

Because of that fact, there is a new phenomenon composite coefficient φ according to which it defines the degree of composite. This coefficient goes in the limits from 0 to 1. This coefficient can not reach the value zero, but it approaches to this value for the case of free adhesiveness of one girder to the other, and the value 1 reaches for the case of total composite. Practical problems appear between these two extreme cases.

The coefficient φ is defined by the relation of the stiffness of the real girder and the idealized girder (the case of total bracing), that is, by the axial moments of inertia. It is obvious the regularity of the dilatations changes, from the dilatation diagram according to the section height. According to the approximation of the dilatation diagram and the bracing coefficient it is suggested the way of calculation of the composite section where the terms are defined for the deformed values. The suggested procedure is simple and it gives the satisfied results, which confirms the parallel analysis of the calculated and measured values.

The transversal distribution is analyzed according to the experimental researches and numerical analysis. Numerical analysis is realized by the usage of finished programs “STRESS” and “SAP 80”. According to the mentioned analysis it comes to the data about the system stiffness in the transversal direction. Both analyses gave the good approximation of the system behavior, but the analysis with the program “SAP 80” gave better results. During the calculation of this system for the stiffness in the transversal section it should include the flange of the prefabricated girder jointly with the plate, and in longitudinal direction it should include the prefabricated girders with the adequate plate part.

The further researches of the composite issues should be directed firstly to the detailed researches of the joint behavior for the different stresses histories, especially to the dynamic effects, as well as to the total behavior of the system as a whole to the fracture phase.

4.3. BEHAVIOUR OF REINFORCED-CONCRETE GIRDERS IN THE LIMIT STATE AREA DURING THE BENDING BY TRANSVERSAL FORCES

Bearer authority: Faculty of Civil Engineering Titograd 1987.
Ordering authority: SIZ FOR SCIENTIFIC ACTIVITIES SRMNE
Head of project: Prof. Arsenije Vujović, PhD in civ. eng.
Research team: prof. Milorad IVKOVIĆ, PhD, PhD h.c. in civ. eng.,
prof. Mirko AĆIĆ, PhD in civ.eng. and
prof. Arsenije VUJOVIĆ, PhD in civ.eng.

"By the research theme "Behaviour of reinforced concrete girders in the limit state area during the bending by transversal forces", the current problem of calculated design and designing of RC beam girders is successfully analized, theoretically and experimentally, for different levels of limit states. Contribution of the analyzed theme is equally significant, experimentally as well as theoretically. Results of the experimental researches present special and very significant contribution to the study of beam girders behavior under load for all stress levels. The received results, supplement a lot experimental results which are received in this area so far, in the world, especially in our country. By the analysis of the theme, on the concrete experiments, by investigation the behavior of quite numerous girders of the real dimensions, with optimal number of measuring plots, modern testing technique is applied and interpretation of received results is performed, as scientific identification of research task, where the important parameters are noticed and selected, which are significant for analysis of characteristic limit states. Generalization of the results of experimental researches, which are done in the area of analyzed theme, as well as by the other researchers, shows that there are conditions for solving the problem of limit state in the area "shearing span", and especially in the inclined cross section. In research project, conditions of plasticity-failure for concrete are analyzed in plane stress state. Beside the fact that frameworks for calculated design and designing of beam girders are suggested under the cross forces, validity of current suggestions for calculated design of these girders in the paper are graded by using experimental researches.

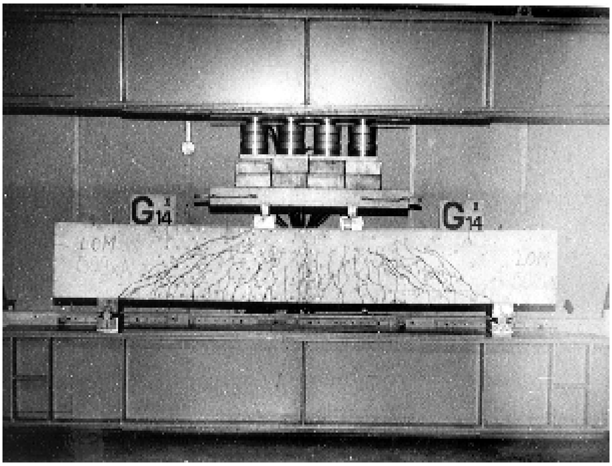

Figure 13. Fragment from experimental researches by A. Vujović

4.4. DEVELOPMENT OF ANALYSIS METHODS AND DESIGNING OF THE HIGH RC GIRDERS

Bearer authority: Faculty of Civil Engineering in Podgorica 1992/93
Ordering authority: MINISTRY OF EDUCATION AND SCIENCE OF MONTENEGRO
Head of project: Prof. Mladen ULIĆEVIĆ, PhD in civ.eng.
Research team: Prof. Mladen ULIĆEVIĆ, PhD in civ.eng.,
Radomir ZEJAK, MSc in civ.eng. and
Đuro LOPIČIĆ, civ. eng.

"This research project successfully deals with the current problem of behaviour of reinforced concrete girders in all stress-deformations states, for theory and practice. Contribution is equally important to experimental and theoretical plan of researches. However, it should point out rich data fund which researchers got, by carrying out their large and original experimental researches in reinforced concrete girders in natural size. Results which are gained in research of behaviour of high beam girders fill the large deficiency which exited not only in our country but also in the world, especially when we talk about continuos high girders. It should point out that this type of girders is tested least in the world, so during the calculation, roughly approximations were performed, giving it the characteristics of linear or wall girder, strained in its plane. By generalization of the results of experimental researches, received by their authors or by the others, as well as with the application of modern theoretical identification of the problem-forming the analytical model for splashed reinforced concrete element in the zone of the emphasized shearing stress, the author gave, except theoretical even the practical contribution, recommending very simple procedures for calculation and construction of high girdres."

Figure 14. Fragment from experimental researches by M. Ulićević

4.5. DEFORMATIONAL RESPONSE OF STRUCTURES UNDER SUSTAINED LOADING

Bearer authority: Faculty of Civil Engineering in Podgorica 2000.
Ordering authority: Faculty of Civil Engineering in Podgorica 2000.
Head of project: Mr. Pero VUJOVIĆ PhD,MSc,DIC,C.Eng.

Project 1: DEFORMATIONAL RESPONSE OF IN-PLANE LOADED PLAIN AND RC PANELS DUE TO CONCRETE AGEING 2000.
Autor of project: Dr. Pero VUJOVIĆ dipl.inž.građ.

This scientific research project deals with basic parameters influencing on behaviour of RC panels under in plane long-term loading. With task to achieve more accurate parameters, author done experimental research on 36 concrete and reinforced concrete panels. Behaviour of half of the specimens was investigated under various cases of in-plane long term loading and on remaining half behaviour of unloaded twin panels. With such concept of experimental research it was possible to identify all long-term stress- dependent deformations on loaded panels, as well as stress-independent deformations on unloaded panels. Quality and way of deformation measurements (independent measurements of reinforcement and concrete) gave the author ability for identification of phenomena appearing in bond of reinforcement and concrete due to aging. Author tried to define parameters for relationship of stress levels and long-term deformations as well as relationship between concrete and reinforced concrete panels due to long term loading. This scientific research also presents existing theoretical foundations and overview of relevant researches with author's comments on it. In favor of defining adequate criteria for crack appearance and existence of some important phenomena author carried on with appropriate conclusions and design proposals.

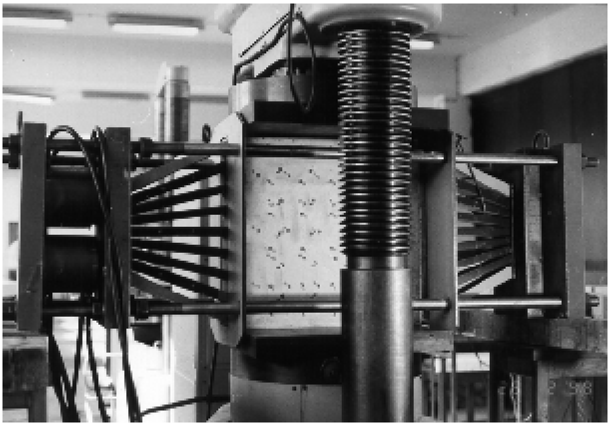

Figure 15. Fragment from experimental researches by P. Vujović

Project 2: DEFORMATIONAL RESPONSE OF THE BIAXIALY BENDED SLENDER RC ELEMENTS DUE TO CONCRETE AGEING 2001.
Autor of project: Dr. Radomir ZEJAK dipl.inž.građ.

This scientific research carried out the analyses of slender reinforced concrete columns under the biaxial bending with the ratio of buckling length to the cross section width in the range between 16 and 32 or characterized slenderness ratio between 60 and 120. Particular attention was given to the investigation of behavior of biaxial bending of slender columns due to creep of concrete during the constant load during the time. In that sense, on the effect of long-term load the 6 slender columns λ=60 were investigated (in period of app. one year) and 2 slender columns λ=120 (in period of 3, that is 4 month). All of these columns, after conducting the treatment of long-term load effect, were investigated up to the breaking on the same way as adequate unloaded "twins" e.g. columns of same geometrical and physical-mechanical characteristics. Accepting previous experience of the researchers, theoretical and experimental character, related to this field, as well as the results of own experiment, the author suggested simplifying the procedure for determination of complete answer to the biaxial bending of slender reinforced concrete columns. On the bases of statistic treatment of the data of own experimental research as well as some relevant results of past researches, the author gave a part of original proposals for estimation and dimensioning of biaxial bending of slender RC columns.

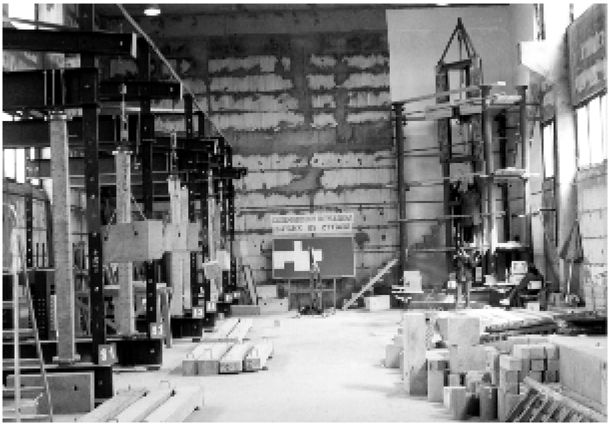

Figure 16. Fragment from experimental researches by R. Zejak

4.6. SOME ASPECTS OF THE CONCRETE APPLICATION OF HIGH STRENGTH

Bearer authority: Faculty of Civil Engineering in Podgorica 2005/2007
Ordering authority:Ministry for education and science of Montenegro
Head of Project: Prof. Radenko PEJOVIĆ, PhD, in civ.eng.
Research team: Prof. Radenko Pejović, PhD in civ.eng.
Radmila-Sinđić Grebović, MSc in civ.eng.

This research project deals with the important issues from the area of concrete and concrete structures from the aspect of knowing and application of concrete as the modern material. Getting familiar with the improved technologies in the concrete production as the building material and the effect of its application is necessary prerequisite for coordination and application of modern technical regulations from that area. Modern European standards for concrete (EN 206-1) includes concrete of much highly strength than the one which is usual in our engineering practice. Within this research project it was done with the concrete of the strength C90/105, the highest class which the European regulations predict for now, during the calculated design of reinforced-concrete structures. Beside that, definition of particular calculated parameters which are in accordance with the suggestions of European regulations, should be adopted by the National Annexes, for adaptation to the local conditions, can find the base in this and similar researches. Experimental research which is realized within this research project created the base for additional analyses from this area. Big base of numerical data, formed as the result of this research, make possible the continuation of the analyses and conduction of new conclusions. One of the important results of the research is the suggestion for the calculated design of the shearing bearing capacity of beams, adapted to the concrete application of high strength. The suggestion is formed on the famous theoretical bases with unavoidable empirical analysis, based on the results of the published experiments. Within this experimental research, the doctoral dissertation is performed by Radmila Sinđić-Grebović, who the candidate successfully defended 08/10/2009.

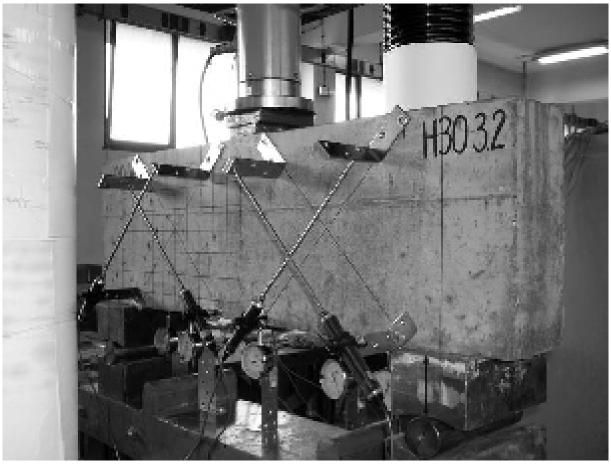

Figure 17. Fragment from experimental researches by R. Sinđić Grebović

4.7. MODELS AND METHODS FOR THE CALCULATION OF THERMAL STRESSES AND PREDICTION OF CRACKS IN CONCRETE

Bearer authority: Faculty of Civil Engineering – Podgorica 2005/2007
Ordering authority" Ministry of Education and Science of Republic of Montenegro
Head of project: Prof. Mladen Ulićević, PhD in civ.eng.
Researcher: Željka Radovanović, PhD in civ.eng.

Subject of scientific research is a response of real structure - a bridge to the influence of temperature changes in the environment. In this research project series of measurements were carried out on a real model of the bridge, and thereby provided a large number of relevant data, which helped in making conclusions about the effects of temperature of the environment on the bridges in a Mediterranean climate.

In conducted research the experimental and theoretical research methods were applied, and the focus of this project was the change of the temperature in cross section depending on air temperature in the shade. Theoretical methods were applied in the formulation of appropriate connections in concrete temperature and measured air temperature in the shade.

The research consisted of three parts: the first part of the research included the analysis of theoretical and experimental researches in this area, the second part was experimental and the third part was the analysis of the results of the research and results of other researchers who have studied the effect of temperature on the bridges.

After the instrumentation of the facility, several series of measurements were carried out in 2006 and 2007. Based on the measured data on temperatures in the concrete of characteristic sections and gained data on air temperatures in the shade, the equations were set for defining of uniform components of the temperature in the concrete of box bridges. After data processing and perception of the interdependence, the statistical method of data processing was applied, using the diagram of dissipation and the method of regresion analysis. Regression analysis was performed with the usage of software package CurevExpert in which 'monitoring' data (experimental data) were processed by

Lagrange's interpolation, regression and nonlinear regression in order to 'match' experimental data with certain mathematical equation-model, given in parametric form.

A method for calculation of maximum middle temperature in the concrete of boxbridge for any area of Montenegro is proposed, using function of dependence on average daily temperatures from daily temperatures, of air in the shade, on the basis of data from Hydrological and Meteorological Service of Montenegro, for the corresponding area.

Researches carried out under this research project are the initial step for creation of our own database on the impact of temperature on the bridges.

5. CONCLUSION

In these thirty years of existence of the Faculty of Civil Engineering in Podgorica, beside the very hard material conditions of working, over 20 scientific projects have been realized. Very large experimental and theoretical researches have been performed within these projects.

The results of the performed researches are mainly used for the preparation of doctoral and master theses. The received results are presented to the wider scientific and professional public by publishing papers in international and domestic scientific magazines as well as their presentation at the international and domestic conferences.

According to the mentioned researches this faculty has become recognized in the region, and further.

The performed researches from the area of timber structures were the pioneer intake, because the wood prestressing was performed for the first time in ex-Yugoslavia. The tests confirmed the rationality of prestressing of lignified wood for large spans and loads.

Very large lab researches are performed from the area of steel structures at the Faculty of Civil Engineering in Podgorica. It is seriously reached to the examination of this problem which is confirmed by the fact that over 200 tests have been performed. Except the analysis of the proper researches, all significant researches are analyzed which are done from this area.

Except the experimental research, the special attention is paid to modeling of problems by the method of finite elements. Linear and non-linear analyses were performed. In the scope of this research, the cooperation is realized with the University in Granada (Spain) where non-linear analyses of this problem were performed.

It should point out the fact that for the first time, in the scope of this research, for modeling of these girders behavior, artificial neuron nets are applied. Regarding the fact it is about the artificial procedure of modeling, it can significantly help in the analysis of these and similar problems.

Although it is about very complex problem, which is the subject of attention of many researchers, the performed researches at the Faculty of Civil Engineering gave a significant contribution in solving of these problems. Four doctoral dissertations are prepared, and dozens of scientific papers are published in the referent modern magazines.

These researches are not finished and they are continued at this Faculty because new problems are open through the performed researches which will be investigated in future.

As it is mentioned in the introduction the largest researches have been performed in this area.

The calculated inclusion of the time dependant effects of concrete creep and contraction and relaxation of steel for prestressing is very complex and not enough investigated. Regarding the very complex high-elastic deformation behavior of concrete the performed researches contributed a lot to the solving of some problems.

In the research project 4.5. the problem of the time behavior is successfully solved for non-primary and reinforced plates that is effects of time deformations on the limit state of those plates. Biaxial bent slender columns are analyzed in the same project, during the concrete creeping under the constant load in a time.

Certain results are realized in the research of the limit states under the bending of transversal forces, as well as in the analysis of the high RC beams.

The problems of the concrete application of high strengthness are examined in the research project 4.6. Through these researches, as one of important results is the suggestion for the calculation of the shearing bearing capacity of beams, made of these concretes.

In the research project 4.7. the effects of the temperature changes on the real structures are examined. Researches are done on the finished bridge. According to the performed researches, the method is suggested for the calculation of the maximal mean temperature in the concrete of the box bridges, for any area, using the data of Hydrometeorology institution.

Using the results of the researches from the area of concrete structures, eight doctoral dissertations are done and a large number of master theses. A large number of scientific papers are presented in the referent modern and domestic magazines, as well as in the international and domestic conferences. At our Faculty, the doctoral dissertation was done by foreign citizen from China Nie Jianguo under the title “Behavior of reinforced beams under the effects of short-term and long-term transversal forces”.

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Svetlana Petkovska Oncevska[1], Koce Todorov[2]
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APPLICATION OF SHAPE MEMORY ALLOYS IN SYSTEMS FOR PASSIVE STRUCTURE CONTROL

Summary: *The term shape memory alloys (SMA) refers to the group of metallic materials that demonstrate the ability to return to the same predefined shape or size when exposed to an appropriate thermal procedure. The unique characteristics of SMA, i.e., the high damping capacity, the high level of stress and strains recovery, the re-centring capabilities and the high corrosion resistance make this material attractive for scientific research. This paper shows the general characteristics of the shape memory alloys and the main physical-mechanical properties of the most frequently used commercial alloys.*
Definition of a material model for numerical simulation of the superelastic response of shape memory alloys is very important for their mathematical modelling. For that purpose, a new original polygonal hysteretic model with variable tangent stiffness has been developed for description of the superelasticity effect in the case of cyclic axial loading.
At the end, the procedure for designing and modelling of a diagonal damper made by combining steel and shape memory alloys is given. The applicability of shape memory alloys in a system for structural control has been analyzed by comparative dynamic analysis of two reinforced concrete frame structures with and without SMA braces, for a given acceleration history.

Keywords: *shape memory alloys, martensite phase transformation, shape memory effect, superelasticity, hysteretic model, SMA dampers, dynamic response.*

ПРИМЕНА НА ЛЕГУРИ ШТО СИ ГО ПОМНАТ ОБЛИКОТ ВО СИСТЕМИ ЗА ПАСИВНА КОНТРОЛА НА КОНСТРУКЦИИ

Резиме: *Терминот легури што си го помнат обликот обединува група на метали кои имаат способност да се вратат во претходно дефинирана форма, облик или геометрија доколку се изложат на одреден термомеханички третман. Поради своите уникатни карактеристики: можност за дисипација на сеизмичка енергија, можност за развивање на големи еластични деформации, самоцентрирачки способности и одлична отпорност на корозија, легурите што си го помнат обликот се предмет на голем број научни истражувања.*
Дефинирањето на материјален модел за нумеричка симулација на суперeластичното однесување на легурите што си го помнат обликот е од големо значење за нивното математичко моделирање. За таа цел, развиен е нов оригинален хистерезисен модел за опишување на ефектот на суперeластичност при циклично аксијално товарење. Според својата природа овој модел спаѓа во групата на полигонални хистерезисни модели со променлива тангентна крутост.
На крајот, прикажана е процедура за проектирање и моделирање на дијагонален пригушувач изработен со комбинација на челик и легури кои си го помнат обликот. Потенцијалот за негова примена е разгледуван преку споредбена динамичка анализа на армирано бетонски рамки со и без вградени пригушувачи.

Клучни зборови: *легури кои што си го помнат обликот, суперeластичност, хистерезисен модел, SMA пригушувачи.*

[1] Prof. Dr. University Ss. Cyril and Methodius, Faculty of Civil Engineering, Skopje, R. Macedonia, once@gf.ukim.edu.mk
[2] Assistant, M.Sc. University Ss. Cyril and Methodius, Faculty of Civil Engineering, Skopje, R. Macedonia, todorov@gf.ukim.edu.mk

1. INTRODUCTION

Shape memory alloys belong to the group of intelligent materials with a high potential of application to intelligent structures and systems. They represent a relatively new group of materials, whose application in civil engineering, as one of the more conservative engineering fields, has gradually been imposed for the last ten years, following the latest knowledge gained from the investigations in this field [12, 7].

The shape memory effect and the effect of superelasticity make shape memory alloys different from the remaining materials that are applied in engineering practice. The manifested unusual macroscopic effects result from the change of the crystal structure of the material caused by martensite and reversible transformation.

2. DEFINITION OF SHAPE MEMORY ALLOYS

The term shape memory alloys - SMA refers to a group of metallic materials, which have the ability to return to their predefined form, shape or geometry if exposed to an appropriate thermo-mechanical treatment. At micro level, the material changes its crystal lattice. At macro level, this is reflected by two characteristic effects: the shape memory effect and the effect of superelasticity. Depending on the state of the crystal structure, shape memory alloys can recover residual plastic deformations if subjected to a heating process (one-way shape memory effect) after unloading, or can return to the initial shape in the process of loading and unloading despite the previously experienced relatively large (sometimes up to 18%) nonlinear deformations.

2.1. Crystallographic structure and phase transformation

Depending on the state of the crystal structure, shape memory alloys can be found in two different stable phases: austenite and martensite. Typically, the austenite phase is stable at high temperatures and low stresses while the martensite phase is stable at low temperatures and under high stresses. Moreover, the martensite phase can be found in two different variants: twinned or detwinned martensite. On the other hand, the austenite phase only takes on a single form and tends to be harder and stronger. A microscopic and schematic presentation of the austenite and martensite phase with two typical variants is given in Fig. 1.

Phase transformation of these alloys occurs at solid state of the material, i.e., at a temperature, which is quite lower than the melting point of the material. For this reason, phase transformation is usually called solid-to-solid transformation.

At relatively high temperatures, a shape memory alloy is in its austenite phase, characterized by a cubic crystal structure, whereat each atom remembers its neighbor. With cooling, the crystal lattice is transformed into twinned martensite with monoclinic crystal structure. The change of the crystal structure from austenite to martensite phase is called martensite transformation. Since the desired shape is defined when the material is in the austenite phase, this phase is often referred to as the natural stage. The transformation from austenite to martensite phase occurs through a displacive process, which does not imply, however, macroscopic changes in the shape of the specimen [10].

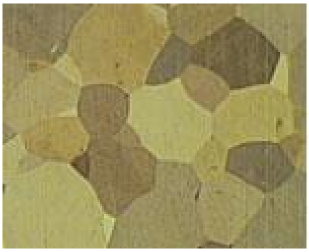
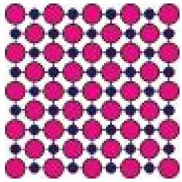
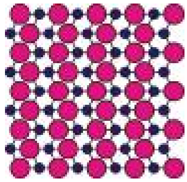
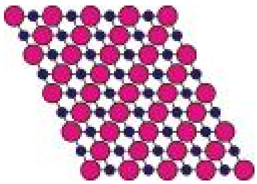
Phase	Microscopic view	Schematic presentation	Crystal structure
Austenite			Cubic
Twinned martensite			Monoclinic
Detwinned martensite			

Figure 1. Characteristics of austenite and martensite phase

Martensite transformation can be induced by change of temperature (cooling) or by application of an external load (change of internal stresses). If there is a temperature increase after the completion of the martensite transformation, the martensite phase becomes unstable. This results in return of the crystal lattice to it natural phase, i.e., original configuration. This process is called reversible transformation, Fig.2.

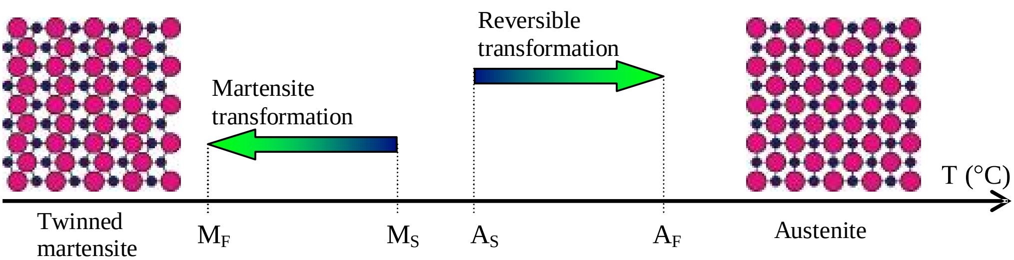


Figure 2. Schematic presentation of martensite and reversible transformation

In a stress-free state, a SMA is characterized by four transformation temperatures: the temperatures at the start of the martensite transformation MS, (the material structure starts to transform from austenite to martensite), the temperatures at the completion of the martensite transformation MF (the whole structure is in the matensite phase), the temperature at the start of the reversible transformation AS (the material structure starts to transform from martensite to austenite) and the temperatures at the completion of the reversible transformation AF (the whole structure is in the austenite phase).

The temperatures of transformation, that can range quite widely, mainly depend on the composition and the processing of the alloy, but also on other factors such as the atomic order, the internal stresses, the defects in the crystal lattice, etc.

2.2. Thermo-mechanical behaviour

As a result of the thermo-elastic martensite transformation, the stress-strain relationship of shape memory alloys is nonlinear, hysteretic and with a great ability for reversible nonlinear deformations. At macroscopic level, the transformation of the crystal structure as a microscopic phenomenon is reflected through two effects that are characteristic only for shape memory materials. These are: effect of superelasticity and the shape memory effect [10].

2.2.1. Effect of superelasticity

The superelastic effect takes place if the crystal structure of the material is in the austenite phase. The martensite transformation can be initiated either by change of the internal stresses at constant temperature (isothermal process) or by change of temperature at constant level of internal stresses (isobaric process), Fig. 3.

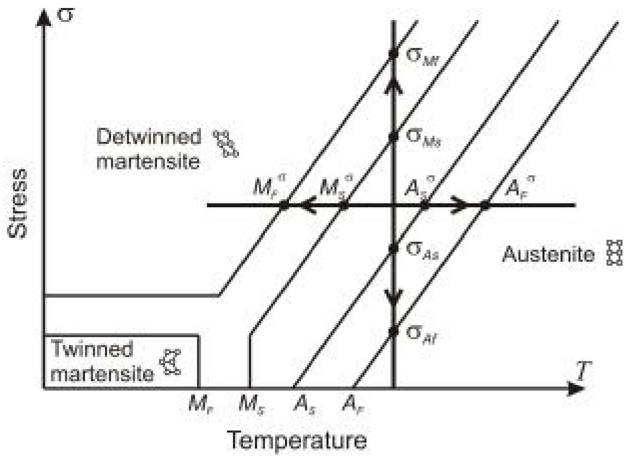


Figure 3. Schematic presentation of phase change at isobaric and isothermal martensite transformation

In an isothermal process, the martensite transformation starts at the moment when the intensity of internal stresses reaches the value of σ_{Ms}, and is completely finished at stress σ_{Mf}. At such transformation, the structure of the material gets into detwinned martensite variation. If, in this state, the material is unloaded, at stress σ_{As}, there starts the transformation from martensite to austenite phase that ends at stress σ_{Af}. At complete unloading of the material, due to the change of the inner structure, all the experienced deformations are lost and the material returns to the initial non-deformed state. The stress-strain relationship during superelastic behaviour of an axially stressed element in the austenite phase is shown in Fig. 4a.

In the process of loading and unloading, due to the difference between stresses σ_{Mf} and σ_{As}, as well as between stresses σ_{Ms} and σ_{Af} , the level of stress at the transition from the martensite to the austenite phase ($\sigma_{As} \rightarrow \sigma_{Af}$) is lower than the stress level at the transition from the austenite to the martensite phase ($\sigma_{Ms} \rightarrow \sigma_{Mf}$), whereat there occurs a difference in the deformation energy for loading and unloading. This leads to a hysteretic effect during which a certain quantity of input energy is dissipated without occurence of residual plastic deformations. The temperature intensity at loading and unloading has an influence upon the intensity of the transformation stresses, however the form of the hysteretic loop does not change, in principle.

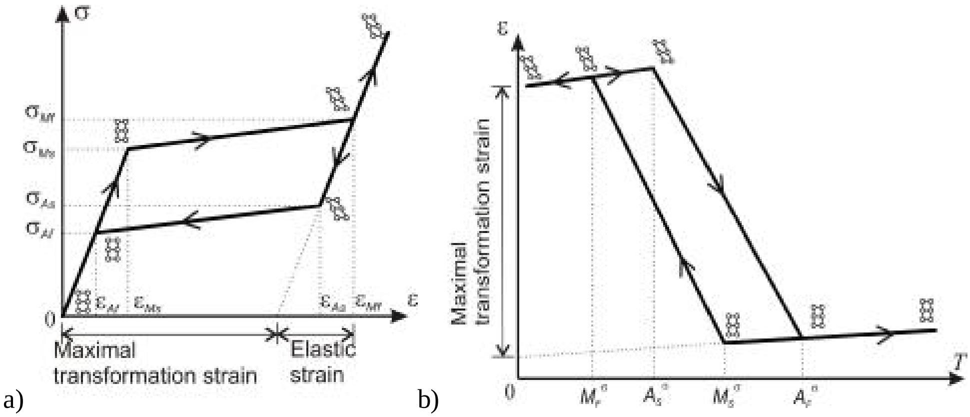


Figure 4. Characteristic diagrams: a) stress-strain at isothermal process and, b) strain-temperature at isobaric process of superelastic behavior

The effect of superelasticity can also be caused by change of the inner temperature of the material at constant level of internal stresses. If the material is in the austenite phase and is exposed to constant stress, which is lower than the stress at the beginning of the martensite transformation, then it will still be in the austenite phase. If in this state, there starts a decrease in temperature, a thermally initiated martensite transformation will take place in conditions when the inner temperature reaches the value of M_S^{σ}. Also in this case, the martensite transformation is characterized by development of large macroscopic deformation strains, whose development ends when the temperature reaches the value of M_F^{σ}. Further cooling at temperatures lower than the temperature at the end of the martensite transformation leads only to occurrence of thermo-elastic shrinkage strains that are proportional to the coefficient of linear spreading/shrinkage of the material in the martensite phase. The transformation strains are several tens of times higher than the thermo-elastic deformations for the same temperature difference. At increase of temperature up to the achievement of the temperature at the beginning of the reversible transformations A_S^{σ}, thermo-elastic spreading strains take place. After the exceeding of such temperature, there starts the process of reversible transformation, which is reflected through the development of transformation shrinkage strains that end at temperature of A_F^{σ}, Fig. 4.b.

2.2.2. Shape memory effect

The shape memory effect takes place if the material is in the martensite phase under working temperature. If, in this state, it is stressed to a certain level, similar to the remaining ductile materials, the material suffers plastic deformations that remain even after complete unloading. For the shape memory alloys, it is characteristic that, if in such deformed non-stressed state, the material is exposed to thermal treatment by increase of temperature, its inner structure changes from martensite to austenite phase. Such transition is characterized by complete reversibility of the residual plastic deformations, which is maintained also after re-cooling, i.e.„ transition from austenite to martensite phase, Fig. 5. The process during which the material memorizes its natural form in the austenite phase is referred to as one-way shape memory effect.

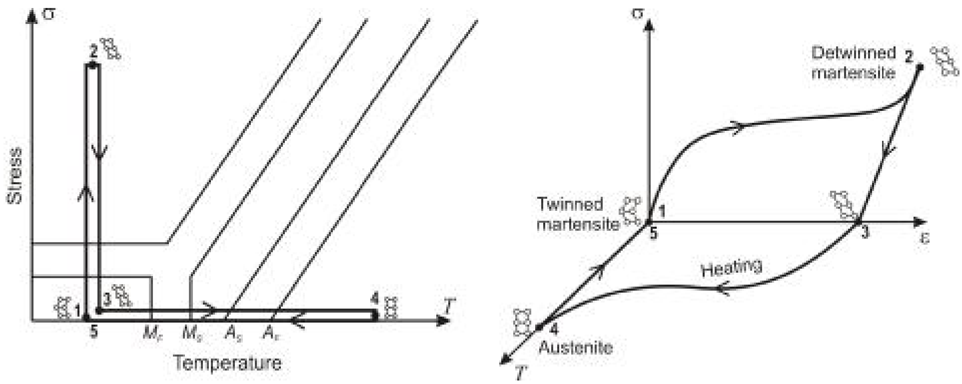


Figure 5. Phase change and characteristic stress-strain diagram at the occurrence of shape memory effect

The reason for the occurrence of the shape memory effect is in the thermo-elastic martensite transformation of the crystal structure. Namely, in the case of decrease of temperature, the crystal structure is transformed from austenite (natural phase) to „twinned“ martensite phase, whereat the material does not suffer any significant change at macroscopic level. From microscopic viewpoint, during the martensite transformation, a fine relief structure is formed on the free surface of the material.. If in such state, the material is deformed (tension, compression, bending, torsion), the crystal structure passes over to a „detwinned“ martensite variation whereat the material undergoes visible deformations that remain even after termination of the external load effect.

2.3. Commercial shape memory alloys

According to their commercial use, shape memory alloys are divided into the following three groups: alloys that are commercially used and produced in assembly- line production (Ni-Ti; Ni-Ti-Cu; Cu-Zn-Al), alloys that are on the threshold of commercial production (Cu-Al-Ni; Fe-Mn-Si) and alloys that, according to the performed scientific research, show a certain potential for use but are characterized by either great brittleness or quite difficult production (Ni-Al; Ni-Ti-Zn) [12].

2.3.1. Nickel-Titanium alloys

The basis for the nickel-titanium alloys is the binary equiatomic intermetal synthesis of nickel and titanium. Compared with other memory shape alloys, these alloys exhibit a great number of superior characteristics that contribute to their most frequent commercial use. Nickel-titanium alloys have the ability of memorizing a big percentage (even up to 8%) of reversible strains. They are stable under cyclic treatment with thermal and mechanical loads, highly resistant to corrosion and with great ductility, Table 1.

The transformation characteristics of the nickel-titanium alloys extensively depend on the percentage of presence of the components of the alloy, possible presence of third metals in the composition as well as the very process of their production. For example, with the increase of the presence of nickel for 1%, the transformation temperatures are decreased even up to 100°C, while, at the same time, the yield limit is increased in the austenite phase. Change of characteristics can also be enabled by adding third elements as follows: iron and chromium for decreasing the temperatures of transformation and copper for decreasing the hysteresis and the stresses during deformation in the martensite phase. Oxygen and carbon are unwanted elements in the nickel-titanium alloys and they can contribute to the change of temperatures of transformation as well as degradation of mechanical characteristics.

Although nickel-titanium alloys exhibit excellent strength-deformability characteristics and high resistance to corrosion, their mass application is limited due to the high cost of their production. Copper alloys compared with the nickel-titanium ones show lower capability regarding reversible strains (4 to 6%) and are less resistant to corrosion, but due to the lower cost of production, the conventional metallurgic process of production and wide range of transformation temperatures, they are increasingly replacing the Ni-Ti alloys in certain applications.

2.3.2. Copper-based shape memory alloys

The most frequently applied copper-based alloys are the alloys of copper with zinc and aluminium (Cu-Zn-Al), while the alloys with aluminium and nickel (Cu-Al-Ni) are still in the phase of development, but exhibit a great potential for practical application.

Copper-zinc-aluminium alloys represent a composition of three metals, whose presence in percentage is different and ranges within the following limits: 65-80% copper, 10-30% zinc and 5-10% aluminium. The temperature at the beginning of the martensite transformation in these alloys extensively depends on the percentage of presence of the individual components and ranges within the limits of -200°c to 120°c. The main negative characteristic of these alloys is the stabilization of the martensite phase which takes place after their long term exposure to room temperature.

The composition of the copper-aluminium-nickel alloys ranges within the limits of 11-14.5% aluminium to 3 to 5% nickel. The percentage of presence of aluminium has a great effect upon the temperatures of transformation and if it is below 12%, there is a considerable improvement of the mechanical characteristics of the alloy. The main physical-mechanical characteristics of the Cu-Zn-Al and Cu-Al-Ni alloys are presented in Table 1.

Characteristic	Unit	Ni-Ti	Cu-Zn-Al	Cu-Al-Ni
Melting point	°c	1240-1310	950-1020	1000-1050
Density (volume mass)	Kg/m^3	6400-6500	7500-8000	7100-7200
Coefficient of thermal expansion in austenite phase	10^{-6} °c^{-1}	10	/	/
Coefficient of thermal expansion in martensite phase		6.6	16-18	16-18
Biological compatibility		Excellent	Bad	Bad
Modulus of elasticity in austenite phase	GPa	47-98	70-100	80-100
Modulus of elasticity in martensite phase		28-41	70	80
Yield limit in austenite phase	MPa	100-800	150-350	150-300
Yield limit in martensite phase		50-300	80-300	150-300
Tensile strength in austenite phase	MPa	800-1500	400-900	500-1200
Tensile strength in martensite phase		700-2000	700-800	1000-1200
Strain at failure in austenite phase	%	15-20		
Strain at failure in martensite phase		20-60	10-15	8-10
Dynamic strength n=106 cycles	MPa	350	270	350
Temperatures of transformation	°c	-200 to 110	-200 to 120	-200 to 200
Temperature hysteresis	°c	20-30	5-25	20-40
Reversible strains at one-way shape memory effect	%	6-8	4-6	5-6
N<102		6-8	4	4
N<105		2		
N<107		0.5		
Specific damping capacity	%sdc	15-20	30-85	10-20
Energy dissipation during a superelasticity cycle	J/g	6.5	1.8	
Maximum reversible stresses	MPa	600-900	400-700	300-600
Maximum reversible strains	%	8	3.5	2

Table 1. Main characteristics of Ni-Ti and Cu-based shape memory alloys, [80]

3. APPLICATION OF SMA IN STRUCTURE CONTROL

The unique characteristics of alloys with shape memory in austenite and martensite phase have contributed to a large number of scientific investigations and development of a series of seismic energy dissipation devices. The efficiency of the developed devices for application in different types of structural systems has been investigated numerically and experimentally by many authors [3, 4, 7, 9, and 13].

Through a comparative numerical analysis of a frame model with braces, Baratta и Corbi have investigated the effect of the type of brace upon the dynamic response of a structure. The analyzed braces have been made by use of material characterized by purely elastic-plastic behaviour or by application of the superelastic effect. Through a large number of nonlinear dynamic analyses, Bruno и Valente have explored the possibility of replacement of the known devices for passive control of structures by devices based on application of shape memory alloys. Andrawes and DesRoches [1, 3] have investigated the possibility of application of restrainers made of shape memory alloys in multi span beam bridges. He uses restrainers to connect the lower belt of the beam girder with the supporting beam of the pier whereat he controls the relative displacements of the longitudinal girders caused by the effect of horizontal forces. With this, he enables development of large elastic deformations and dissipation of input energy. In a series of experimental studies, the same author explores the possibility of using alloys with shape memory in the martensite phase for connection of steel columns and beams, Fig. 6. Similar investigations have been done by Ocel et al. [9].

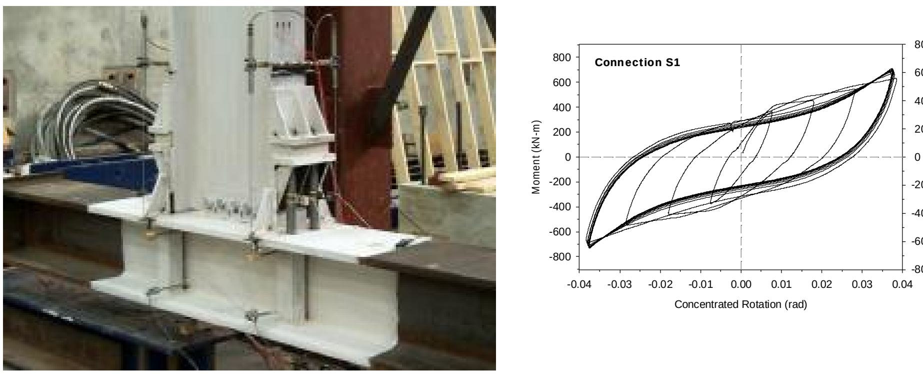


Figure 6. Integral beam-column connection investigated by DesRoches and Ocel, [13]

Extensive numerical and experimental investigations of the possibilities of application of shape memory alloys have resulted in several successful examples of improvement of seismic resistance of existing structures. The first known example is the repair of the 18.5m high bell tower of the San Giorgio church in Trignano, Italy (2001) [6] damaged by the earthquake with intensity of 4.8 degrees according to Richter in 1996. The structure has been repaired by four prestressed cables placed at the bell-tower corners, running from the top to the foundation of the structure. Hanged on these cables are devices composed of 60 wires with a diameter of 1 mm and length of 300 mm made of nickel-titanium alloys whose purpose is to provide constant pressure in the bell tower walls, Fig. 7.

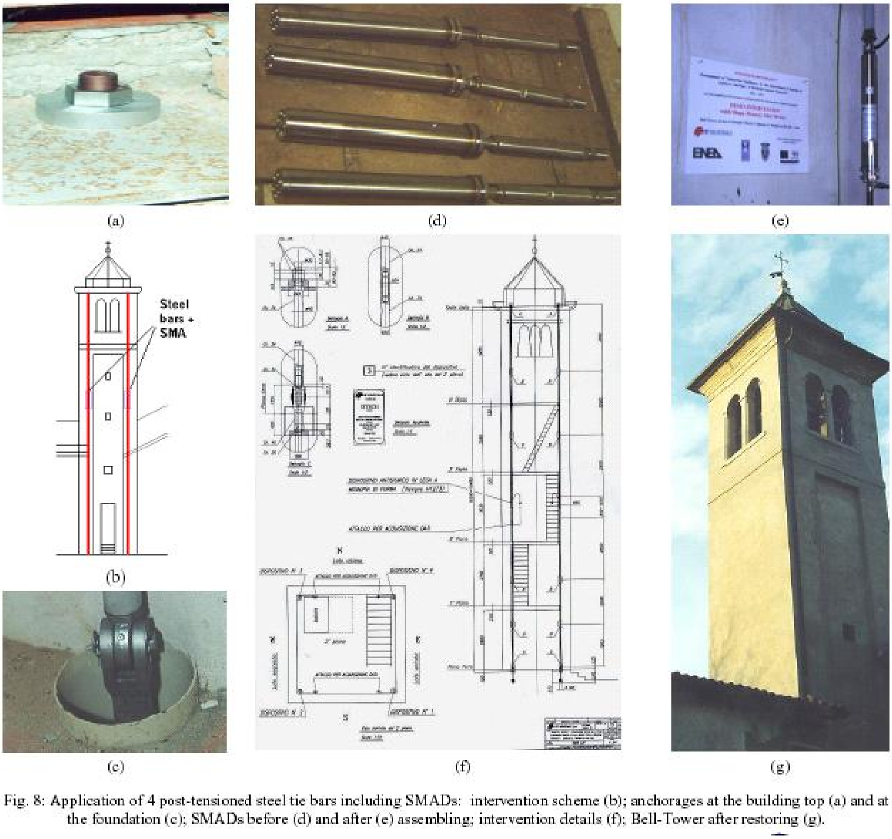


Figure 7. Improvement of the seismic resistance of the bell tower of San Giorgio church in Trignano, [6]

Another example of application of shape memory alloys in improvement of seismic resistance is the repair of the St. Francesco basilica in Assissi, Italy (2001) [2]. In this case, for the purpose of reducing the transfer of seismic forces from the roof to the tympanum, a soft connection has been realized by application of isolators made of shape memory alloys, Fig. 8.

Figure 8. Isolator with SMA seismically favourable connection of the tympanum with the roof of the St. Francesco basilica in Assissi, Italy (2001), [2]

4. POLYGONAL HYSTERETIC MODEL FOR SIMULATION OF SUPERELASTICITY EFFECT

As mentioned above, shape memory alloys are characterized by unique behaviour under cyclic loads whereat, depending on the conditions of the material structure, two states are possible: shape memory effect and superelasticity effect. Although in literature there are a large number of material models that successfully simulate the behaviour of shape memory alloys under the effect of cyclic loads[5, 11], some of them can be quite complex and difficult for implementation in the existing computer codes. For that purpose, a new hysteretic model has been developed to describe the superelasticity effect under cyclic axial loads. According to its nature, this model belongs to the group of polygonal hysteretic models with variable tangent stiffness [13].

The defined model enables a satisfying simulation of the process of cyclic loading and unloading of tensile elements. In the model, it is assumed that the negative branch, which defines the behaviour under compression, is symmetrical with the positive one, whereat the same material constants are used for its description. If the corresponding experimental investigation proves that the characteristic values under compression are different from the values under tension, it is possible to define an asymmetric hysteretic loop by application of the same constitutive laws. Certainly, for that purpose, it will be necessary to define seven new material constants that will hold for the part referring to compression.

4.1. Definition of material model

The description of the stress-strain relationship in the proposed material model is based on material constants obtained by experimental tests, Fig. 9.

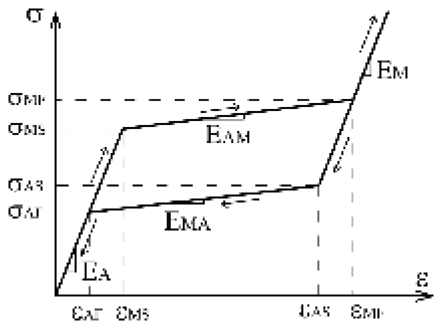


Figure 9. Stress-strain relationship at superelastic behaviour

Input parameters for definition of the material model are the two characteristic modulus of elasticity in the austenite and martensite phase E_A and E_M, the four transformation stresses: the stresses at the beginning and at the end of the martensite transformation σ_{MS} and σ_{MF}, the stresses at the beginning and at the end of the austenite transformation σ_{AS} σ_{AF}, and the strain at the end of the martensite transformation ε_{MF}.

The remaining parameters that are necessary for definition of the stress-strain relationship, the modulus of transformation, the strains at the beginning of the martensite and reversible transformation and the strain at the end of the austenite transformation, can be computed by means of the previously defined constants:

$$E_{AM} = \frac{s_{MF} - s_{MS}}{e_{MF} - e_{MS}} \; ; \qquad E_{MA} = \frac{s_{AS} - s_{AF}}{e_{AS} - e_{AF}} \tag{1}$$

$$e_{MS} = \frac{s_{MS}}{E_A} \; ; \qquad e_{AS} = e_{MF} - \frac{s_{MF} - s_{AS}}{E_M} \; ; \qquad e_{AF} = \frac{s_{AF}}{E_A} \tag{2}$$

The stress-strain relationship in the proposed model is defined by the variation of the tangential modulus of elasticity for the procedure of cyclic loading at control of deformations. This means that the history of variation of strains is known and it is necessary to obtain the history of variation of stresses. The increase of strains is equal to the difference of strains in two successive steps.

$$\Delta e = e_i - e_{i-1} \tag{3}$$

If this value is positive, a loading process is recorded. In the opposite case, there is an unloading process.

The increase of stresses is obtained as a product of the tangential modulus of elasticity and the strain increase, while the stress value is obtained by adding the computed increase to the previous stress value.

$$\Delta s = E_T \cdot \Delta e \tag{4}$$

$$s_i = s_{i-1} + \Delta s \tag{5}$$

With this, the problem of describing the stress-strain relationship is reduced to defining the current tangential modulus of elasticity. For its easier definition, the stress-strain relationship is divided into three branches, Fig. 10, whereat the tangent stiffness of each of these is defined by corresponding relationships.

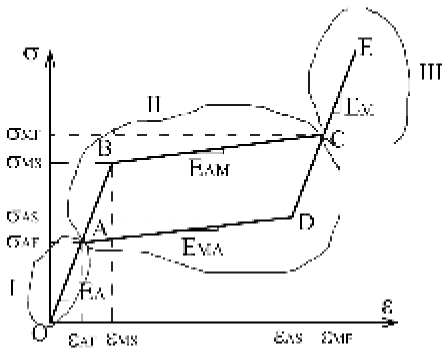


Figure 10. Characteristic branches of the stress-strain relationship

Part I (O÷A), defines the stress-strain state when the material is in complete austenite phase, $e < e_{AF}$. For this part, during loading and unloading, the tangential modulus is equal to the austenite modulus of elasticity, $E_T = E_A$.

Part II (A÷B÷C÷D) represents the main hysteretic loop, whereat the value of the strains is in the range between the strain at the end of the austenite phase and the strain at the end of the martensite phase, i.e., $e_{AF} < e < e_{MF}$. During complete loading and unloading, the tangential modulus is equal to the modulus characteristic for each phase: part (A÷B), $E_T = E_A$; part (B÷C), $E_T = E_{AM}$; part (C÷D), $E_T = E_M$; part (D÷A), $E_T = E_{MA}$.

If the value of the strains during loading and unloading exceeds the value of the strain at the end of the martensite transformation $e > e_{MF}$, then the relationship is in part III, whereat the tangential modulus is equal to the modulus in the martensite phase, $E_T = E_M$.

The described relationships hold for the process of loading and unloading whereat complete martensite, i.e., complete reversible transformation is performed. For such process, the values of stresses and strains range within the boundaries of the main hysteretic loop (O÷B÷C÷E÷D÷A÷O).

If in the process of loading, one starts with unloading prior to the reaching of the strain value at the end of the martensite transformation, $e_{MS} < e < e_{MF}$, a new, the so called secondary hysteretic loop is formed, Fig. 11.

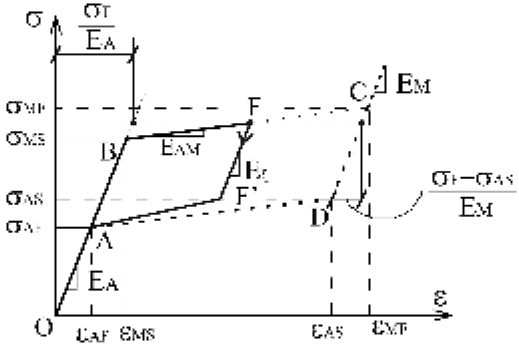


Figure 11. Secondary hysteretic loop at unloading

From the beginning of the process of unloading (point F) until the stresses reach the value at the beginning of the austenite transformation $s < s_{AS}$ (point F'), the tangential modulus can be computed as percentage of participation of the austenite and martensite modulus, depending on the reached value of strains at the moment of starting with the unloading.

$$E_T = E_x = x \cdot E_M + (1-x) \cdot E_A \tag{6}$$

where: $$x = \frac{e_F - \dfrac{s_F}{E_A}}{e_{AS} + \dfrac{s_F - s_{AS}}{E_M} - \dfrac{s_F}{E_A}}\ ;\quad 0<\xi<1 \tag{7}$$

At the moment when stress $s = s_{AS}$, there starts the process of reversible transformation whereat relationship $s \div e$ continues to follow the F'÷A branch:

$$E_T = \frac{s_{F'} - s_{AF}}{e_{F'} - e_{AF}} \tag{8}$$

Similar relationships hold also if, during the process of unloading, when the material is in the part of reversible transformation (D÷A), for strain $e_{AF} < e < e_{AS}$, there starts the process of loading, Fig. 12.

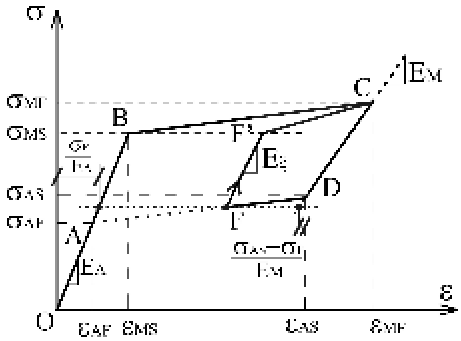


Figure 12. Secondary hysteretic loop at loading

The tangential modulus, from the beginning of loading (point F) until the stresses reach the value at the beginning of the martensite transformation $s < s_{MS}$ (point F'), will be obtained as a combination of the modulus in the austenite phase and the modulus in the martensite phae (Eq. 6). Also in this case, the participation of the martensite part can be computed according to the relationship given in Eq.7.

In the process of loading, when the stresses reach the value of the stresses at the beginning of the martensite transformation $s = s_{MS}$ (point F'), there starts the martensite transformation, whereat the tangential stiffness is equal to:

$$E_T = \frac{s_{MF} - s_{F'}}{e_{MF} - e_{F'}} \tag{9}$$

This relationship will hold until the strains reach the value corresponding to the end of the martensite transformation $e = e_{MF}$, whereat the material starts to behave in complete martensite phase with a tangential modulus $E_T = E_M$.

4.2. Numerical simulation

The defined hysteretic model has been used for simulation of several characteristic cases of cyclic loading under the effect of axial compression and tension. The results obtained from the numerical simulation have been compared with the results obtained from performed experimental investigations as well as with some other material models.

The first considered example represents simulation of the process of partial cyclic loading and complete unloading, Fig. 13 and Fig. 14, whereat the values of stresses are alternately changed from compression into tension and vice versa. The second example represents a process of complete loading and partial unloading, Fig. 15 and Fig. 16 so that, in the first 7 cycles, the strains are positive (tension), while in the next 7 cycles, they are negative (compression). The obtained results have been compared with the numerical results obtained by Fugazza [5]. The third considered example represents a process of partial loading and partial unloading under compression and tension. The results from the analysis are presented by three characteristic diagrams, input history of strains, computed history of stresses and stress-strain relationship.

Numerical analysis for the first two examples has been carried out for the same modulus of elasticity in the austenite and martensite phase. The values of the material characteristics have been adopted according to the data given in reference [5] and amount to: $E_A=E_M=40000$ MPa, $\varepsilon_{MF}=7.2\%$, $\sigma_{MS}=500$MPa, $\sigma_{MF}=600$MPa, $\sigma_{AS}=300$MPa, $\sigma_{AF}=200$MPa.

To present the functioning of the hysteretic model also for material of different deformation characteristics in the austenite and the martensite phase, in the third example, different modulus of elasticity have been adopted for the two characteristic phases: $E_A=60000$ MPa, $E_M=20000$ MPa, $\varepsilon_{MF}=6\%$, $\sigma_{MS}=500$MPa, $\sigma_{MF}=600$MPa, $\sigma_{AS}=300$MPa $\sigma_{AF}=200$MPa, Fig.17 and Fig.18.

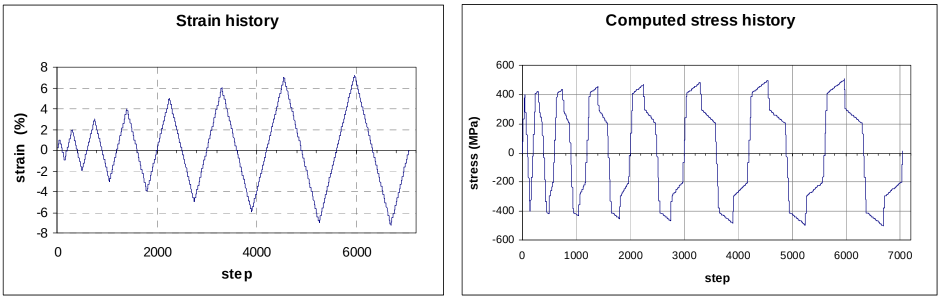


Figure 13. Input history of strains and computed history of stresses in the case of partial loading and complete unloading, Example 1

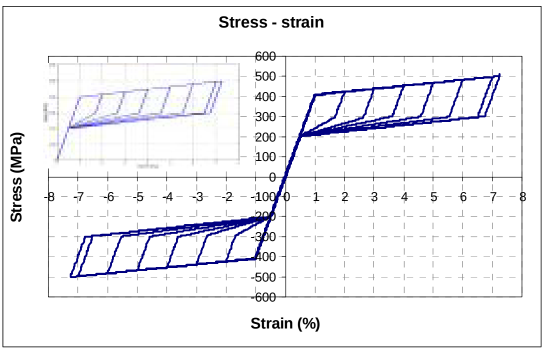


Figure 14. Computed stress-strain relationship and comparison with the results obtained by Fugazza [5], *Example 1*

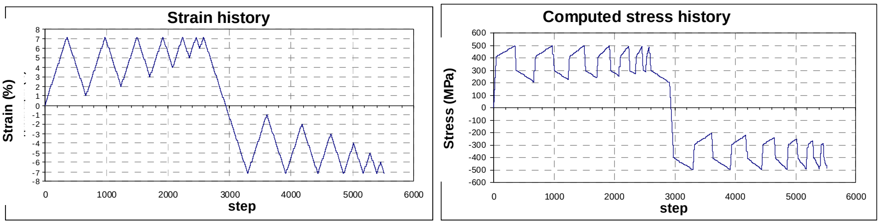


Figure 15. Input history of strains and computed history of stress in case of complete loading and partial unloading, Example 2

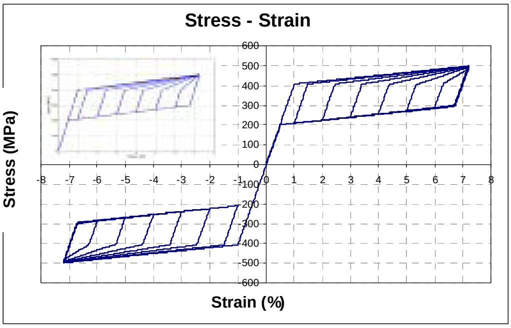


Figure 16. Computed stress-strain relationship, Example 2, and comparison with the results obtained by Fugazza [5]

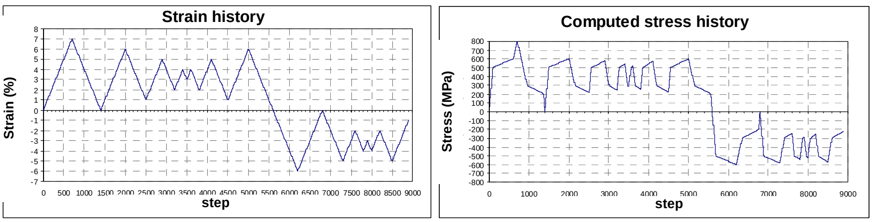


Figure 17. Input history of strains and computed history of stress in case of partial loading and partial unloading, Example 3

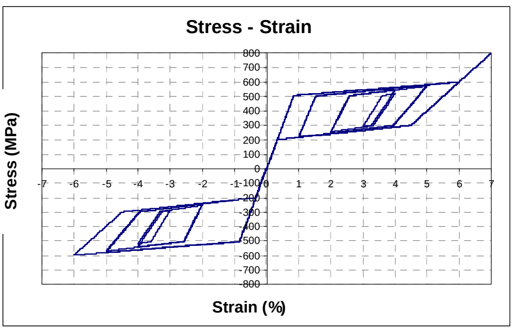


Figure 18. Computed stress-strain relationship, Example 3

The defined material model has been used for numerical simulation of the results obtained from the experimental testing of nickel-titanium wires with a diameter of 1.49mm. The material has been produced by Nitinol Devices & Components [8], while the results from the experimental and numerical testing are taken from Fugazza [5]. From the analysis of the experimental curve obtained at complete loading and unloading, the following characteristics have been adopted for the numerical simulation: E_A=50000 MPa, E_M=16000 MPa, ε_{MF}=8.2%, σ_{MS}=520MPa, σ_{MF}=600MPa, σ_{AS}=260MPa, σ_{AF}=160MPa, Fig.19 and Fig.20.

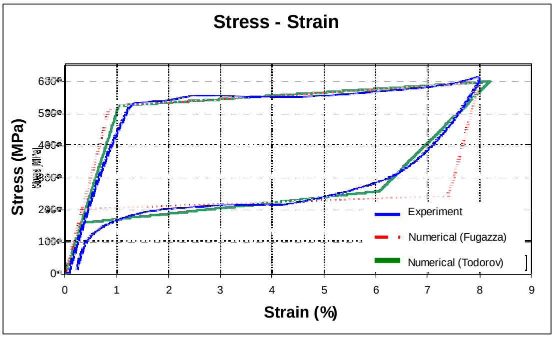


Figure 19. Comparison between the experimentally and numerically obtained curve at complete loading and unloading

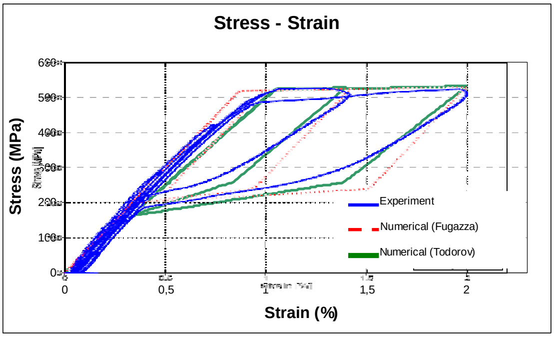


Figure 20. Comparison between the experimentally and numerically obtained curve at cyclic loading and unloading

5. DESIGN AND MODELLING OF DIAGONAL SMA DAMPER

The development of a diagonal steel damper to contain an element made of shape memory alloys of a certain length can significantly reduce the costs and, at same time, can be effective in reducing the seismic response. Such a diagonal structural element composed by serial connection of a steel part with an appropriate length and cross section and part made of shape memory alloys (SMA) in the austenite phase, with its length and cross section, will increase the lateral structural stiffness, and in the case of a strong earthquake, it will dissipate a part of the input seismic energy through hysteretic damping [14]. Because it is known that the amount of hysteretic damping depends on the deformations that occur in structural elements, the question that arises is what are the optimal dimensions of the two constituent parts of such a hybrid damper and how will it behave during a cyclic action.

5.1. Definition of geometric characteristics of SMA damper

The geometric characteristics of diagonal damper components can be defined by few preliminary analyses based on the conditions for equilibrium of forces and compatibility of deformations, Fig. 21. For the required design parameters (length of the diagonal, maximal force and maximal deformation of the diagonal at a certain level of earthquake action) and for known material properties of both elements, Fig. 22, the geometric characteristics of the main damper parts can be calculated [14].

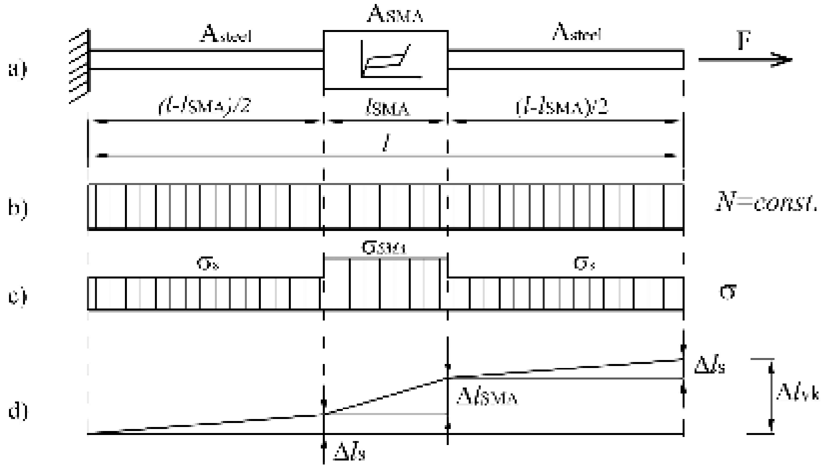


Figure 21. State of stress and deformation at axially loaded hybrid damper made of serial connected elements

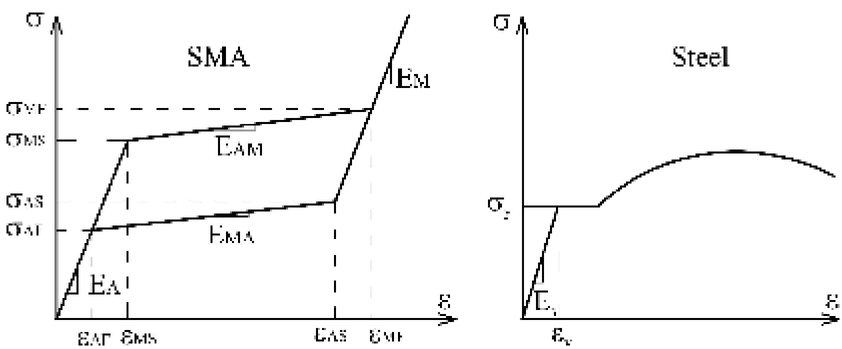


Figure 22. Stress-strain diagrams for the main parts of the damper

In this example, the geometrical properties of the diagonal damper are calculated for an element with a total length of 5m and known material properties for the part made from shape memory alloys (modulus of elasticity in the austenite and martensite phase,

$E_A=E_M=60GPa$, strains at the end of the martensite transformation, $\varepsilon_{MF}=6\%$, stress at the start of the martensite transformation, $\sigma_{MS}=500MPa$, stress at the end of the martensite transformation, $\sigma_{MF}=600MPa$, stress at the start of reversible transformation, $\sigma_{AS}=300MPa$ and stress at the end of the reversible transformation, $\sigma_{AF}=200MPa$), and for the part made of mild steel with modulus of elasticity, E_{steel} =210GPa and yield stress equal to $\sigma_y=240MPa$.

The maximal elongation of the diagonal at the end of the martensite transformation has been adopted to be Δlmaxmf=20mm, which is smaller than the elongation of the diagonal at interstory drift of 1%. The maximal force in the damper at the end of the martensite transformation has been restricted to Fmax=120kN. For these input parameters, the following dimensions of the main parts of the damper have been calculated: shape memory part with a length of 32cm and cross section area of 2cm2, and steel part with a length of 468cm and cross section area of 30cm2. The initial axial stiffness of the element made by serial connection of these two main parts is equal to 30000kN/m. The axial force – deformation relationship for the adopted material and the geometric properties of the main parts of the damper are presented in Fig. 23.

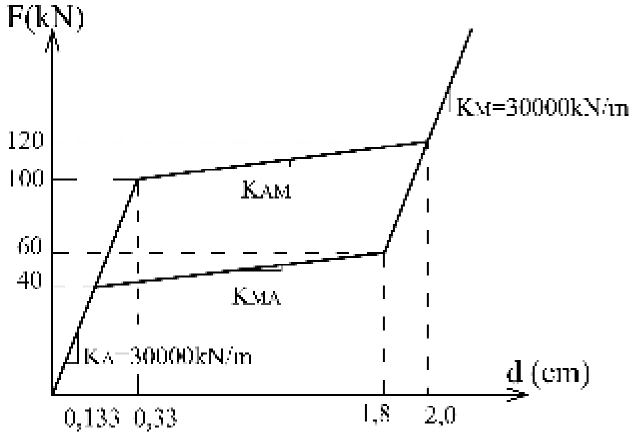


Figure 23. Force – deformation diagram for the designed damper

Numerical simulation of the resulting force-deformation diagram for the designed damper can be performed using specially developed hysteretic phenomenological models for simulation of the superelastic behavior of shape memory alloys [11], or by parallel and/or serial connection of two or more elements with a well known hysteretic role of behavior.

5.2. Application of existing link elements for modelling superelasticity behaviour

The development of the finite element method, aided by the rapid development of information technology in the last thirty years has contributed to the development of many sophisticated computer programs for analyzing complex engineering problems. Nonlinear dynamic analysis, as one of the most complicated methods for analysis of structural systems in structural engineering, is incorporated in a number of three-dimensional computer programs, but material nonlinearity under the effect of dynamic loads is still mostly limited to the level of line elements. Hysteretic behavior simulation in many of these programs is modeled through specially developed nonlinear line elements, whose behavior is defined on the basis of known physical laws. Hysteretic complex models that are not embedded in a computer code can be simulated by parallel and/or serial connection of two or more elements with known law of behavior. These elements are usually defined by two nodes with six degrees of freedom per node, where the number of the required material parameters depends on the observed degree of freedom and type of element, i.e. material model by which it is defined.

The hysteretic behavior of the SMA damper element in this study has been modelled by parallel connection of two link elements, multi-linear elastic link element and Wen plastic link element. Thus, a hybrid element with complex hysteresis behavior has been formed, where the multi-linear elastic element has the role of raising the hysteretic loop and executing re-centering after unloading, while the plastic Wen element has the role of establishing the hysteretic loop and allowing dissipation of energy, Fig. 24. At a certain load level, both elements of this system exhibit the same deformation and generate internal forces proportional to their stiffness. From the equilibrium condition, the total force at any moment is equal to the sum of forces in both elements. It should be noted that this modeling approach provides an approximate simulation of the superelastic hysteresis obtained during cyclic loading and unloading in the direction of the longitudinal axis of the elements.

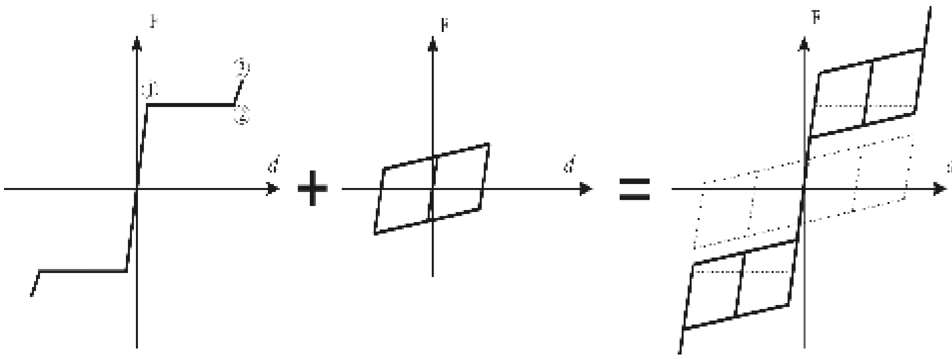


Figure 24. Schematic presentation of the element behaviour obtained by parallel connection of the linear elastic and Wen plastic model

The necessary parameters for defining the nonlinear characteristics of the two link elements can be obtained from the characteristic force – deformation diagram for the SMA damper, established on the basis of geometrical and material characteristics of both its constituent parts. The force that defines the horizontal plateau in the elastic element (point 1, Fig. 24) is equal to the mean of the sum of forces that cause the beginning of the martensite and the end of the austenite phase, $F_1 = (F_{MS}+F_{AF})/2$. Hardening during the martensite transformation can be simulated by the ratio between the postelastic and the elastic stiffness of the Wen model. The yield force in the Wen plastic element is equal to one half of the force which is defined by the hysteretic loop, $F_y = (F_{MS} - F_{AF})/2$. The end of the martensite transformation is determined with the second point of the multilinear elastic element, by the deformation at the end of the martensite transformation, after which the element has stiffness equal to the stiffness in the martensite phase.

5.3. Comparative dynamic analysis of frame structure with and without SMA diagonals

In order to investigate the influence of the SMA diagonal dampers on the dynamic response of a structural system, a comparative dynamic analysis of a reinforced concrete frame with and without SMA diagonals has been performed for a given time history of acceleration.

5.3.1. Description of analyzed structures

The analyzed structures represent five story three bay reinforced concrete frames with a central span of 4m, and an end span of 5m. The frames are designed with a total height of 15.0m. The distance between the frames in transversal direction is 5m. Frame R1, Fig. 25a, represents a plain reinforced concrete frame made from concrete C25/30 and reinforcement with yield stress 400 MPa.

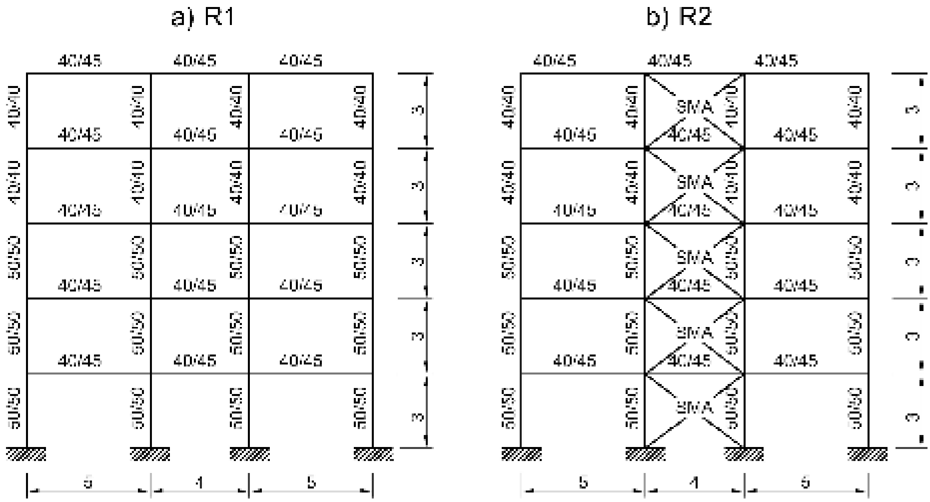


Figure 25. Geometric characteristics of the analyzed frames

The geometrical characteristics of the structural elements have been calculated in accordance with the requirement in the current seismic code in our country. In order to reduce deformation and increase dissipation of input seismic energy, the designed frame has been upgraded by incorporation of "X" diagonal SMA dampers in the middle span of the frame. Thus, frame structure R2 has been formed, Fig. 25b. For the purpose of the analysis, it has been adopted that all diagonals work with the same characteristics in compression and tension, i.e. the problem of buckling is neglected. For the purpose of comparison of the results, another frame R3 has been included in the dynamic analysis. Its stiffness characteristics have been the same as those of frame R2, but it has been analyzed by linear dynamic analysis.

The total mass of one floor has been determined for a uniformly distributed load of 14 kN/m2, i.e. 70 kN/m' line loads at each storey of a frame. So, a defined mass of 100t (70x14/9.81= 100t) has been distributed over four nodes at one level of the analyzed frame, Fig. 26.

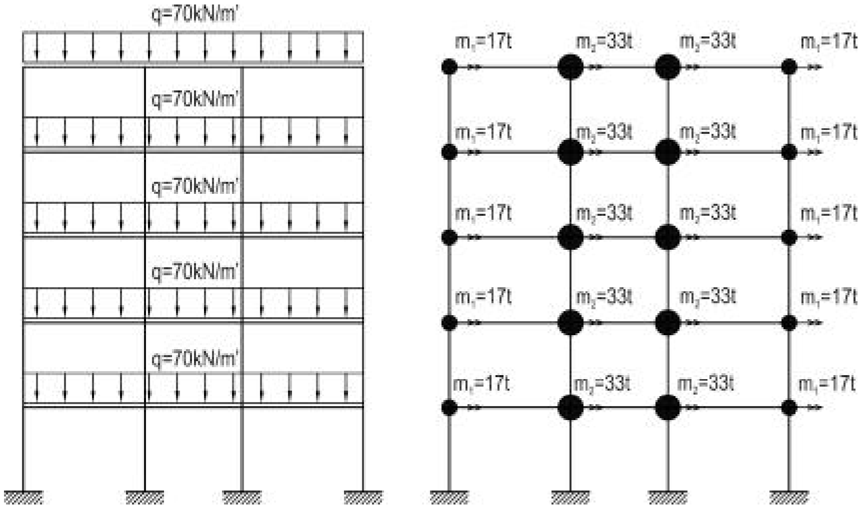


Figure 26. Distribution of loads and mass over the analyzed frames

First, a comparison of the dynamic behavior has been performed for harmonic acceleration time history at the base of the structure with amplitude of 1m/sec^2 and period equal to the first period of vibration of the considered structures. Then, both structures have been exposed to the acceleration time history of a real earthquake, El Centro time history with PGA = 0.313g. All the analyses have been performed for three different values of equivalent modal damping, 0, 2% and 5%. From the performed analyses, a huge amount of data (time histories of displacements, accelerations, base reaction, force-displacement diagrams in non-linear elements, energy diagrams, etc.) showing the behavior of the analyzed structures in different scenarios of ground motion, have been obtained.

5.3.2. Analysis of obtained results

The presence of diagonal dampers in frame R2 increases the horizontal stiffness in the case of moderate earthquakes and decreases the horizontal displacements. The results of the modal analysis indicate reduction of the first periods of vibration in frame R2 for about 19% compared with frame R1 (T1 (R1) = 0.827 sec compared with T1 (R2) = 0.671 sec.). A similar trend of reduction of periods of vibration has been observed regarding the higher modes (T2 (R1/R2) = 0.279sec. /0.225sec.; T3 (R1/R2) = 0.151sec./0.127sec. etc) .

The time histories of displacement obtained under the El Centro earthquake without modal damping, have shown that the stiffer frame R3 exhibits a larger total displacement (27cm.) compared with the more flexible frame R1 (23cm), Fig. 27. These seemingly contradictory results are due to the frequency content of the input earthquake which actuates the stiffer structure R3 more than the more flexible R1. In this analysis, the frame with SMA dampers, R2, exhibits much smaller maximal displacements on the fifth floor amounting to about 12 cm, i.e. about 2 times smaller than those of the corresponding frame R1. This difference in the obtained displacement is mainly due to the variable stiffness of frame R2, which “escapes” from the dominant frequency range of the earthquake during the seismic action as well as due to the hysteretic damping which occurs in SMA dampers. From the analysis with 5% modal damping, Fig. 27, frame R2 exhibits 40% smaller displacement compared with frame R1.

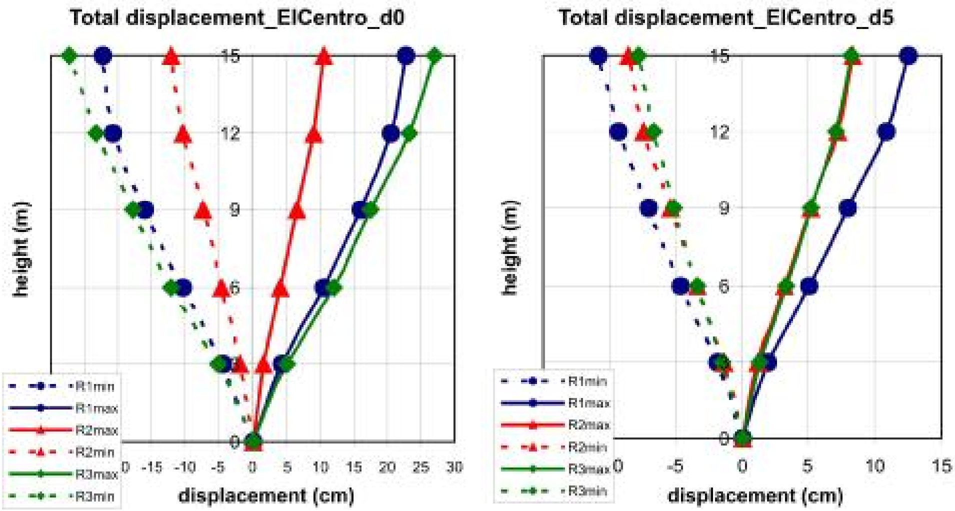


Figure 27. Maximal storey displacement for 0 and 5% damping

The maximal value of interstorey drifts obtained from the El Centro time history analysis without modal damping, Fig. 28, is 6.98cm at frame R3, and it is 2.8 times larger compared with the interstory drift of frame R2.

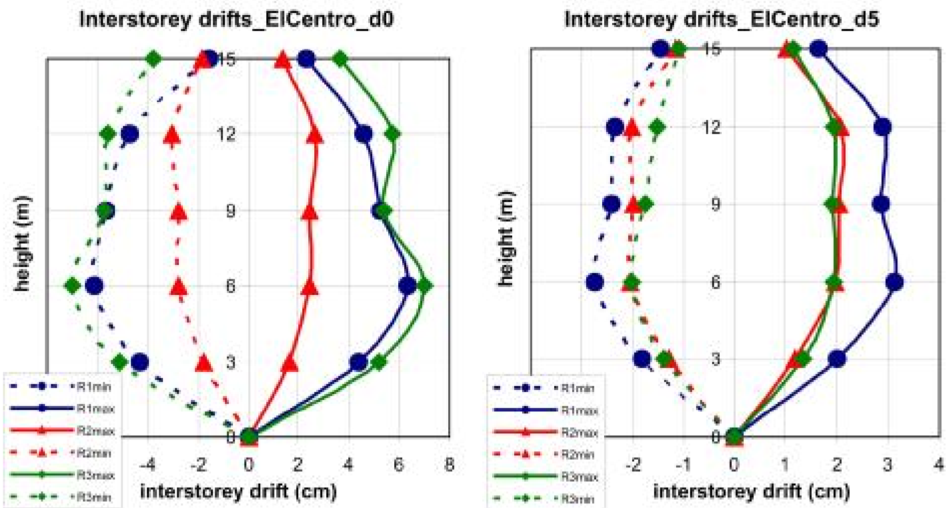


Figure 28. Maximal interstorey drifts for 0 and 5% damping

In the analysis with 5% modal damping, frame R1 exhibits maximal interstory drift equal to 3.12cm which is about 1.52 times larger than that of the corresponding frames R2 and R3. The relative interstory drifts of frame R2 are closely related with the deformations in diagonal dampers placed in the middle span of that frame. The optimal design of the geometric characteristics of the dampers' constituent elements should follow the change of interstory drifts along the height of the building. Since the same values have been adopted for the damper dimensions at all storeys of the structure, the force-deformation curves show a different state of deformation of dampers at individual storeys.

Maximal deformation of dampers occurs at the place with the maximal interstory drift, i.e., the second storey of the structure in the case of periodic time history without modal damping, Fig. 29. A complete martensite transformation of that damper is finished, corresponding with maximal displacement of 2.35 cm and force of 225kN. Complete martensite transformation is also finished at the dampers of the third and the fourth storey of the structure in the case of periodical input, and at the dampers of the second, the third and the fourth storey in the case of the El Centro time history without modal damping. In the case of analysis with 5% modal damping, a complete martensite transformation is not finished in any damper. The maximal interstory drifts of the second, the third and the fourth storey are fairly uniform at about 1.5 cm, compared with the deformations of the first storey that are around 1cm, or deformations of the fifth storey equal to 0.75 cm. The force – displacement curve for the dampers at the second storey in the case of time history analysis for periodical input without modal damping, and for the El Centro earthquake with 5% damping are presented in Fig. 29.

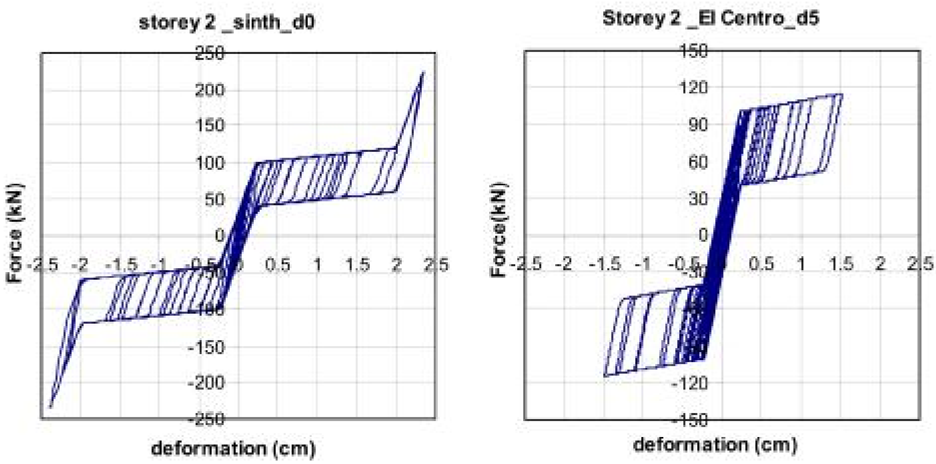


Figure 29. Force-deformation diagrams for the dampers at the second storey of frame R2

On the energy diagram for frame R2, Fig. 30, participation of hysteretic damping in the total dissipated input energy can be observed. In the case of the El Centro time history analysis without modal damping, the total input energy is dissipated through hysteretic damping. In the analysis with 2% modal damping, the hysteretic damped energy is around 10% larger compared with the modal damped energy. A similar ratio can be observed from the energy diagram for 5% modal damping, where the modal dissipated energy (224 kNm) is about 2 times larger compared with the hysteretic dissipated energy (113 kNm). According to these results, the effective damping of the designed hysteretic dampers is equivalent to 2.5% modal damping for the analyzed structure.

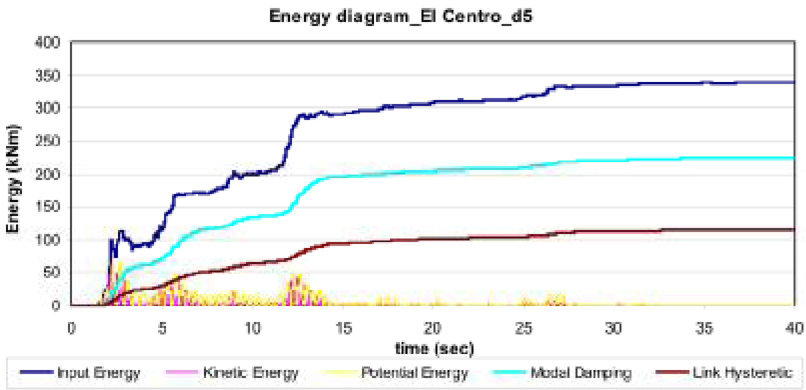


Figure. 30. Energy diagrams for the El Centro time history analysis with 5% modal damping

6. CONCLUSION

From the review of available literature and the performed analytical and numerical investigations, a number of conclusions can be drawn. These can generally be divided into conclusions regarding the characteristics of the shape memory alloys, conclusions regarding the proposed material model and conclusions in respect to the potential for use in systems for passive control of structures.

Shape memory alloys belong to the group of intelligent materials of a high potential to be used in intelligent structures and systems. The shape memory effect and the superelasticity effect make shape memory alloys different from other materials that are applied in engineering practice. The manifested unusual macroscopic effects result from the change of the crystal structure of the material caused by martensite and reversible transformation. The physical-mechanical characteristics of shape memory alloys range widely and depend on the control of the process of their obtaining, the percentage of presence of the components in the alloy, the presence of third elements and alike.

The proposed original hysteretic model for description of the effect of superelasticity under cyclic axial load belongs to the group of polygonal hysteretic models with variable tangent stiffness. The model has a number of advantages as are the simple algorithm for computation, the easy definition of the necessary material parameters by means of a uniaxial compressive or tension test, the different elastic characteristics in the austenite and martensite phase, the possibility of simulation of different loading conditions at cyclic loading and the easy implementation in some of the open software codes.

The potential for application of shape memory alloys in devices for passive control of structures has been considered through quasistatic and dynamic analysis of diagonal elements composed of part made of steel and part made of shape memory alloys.

From the analyzed results, it can be concluded that the built-in diagonal damper has a dual role in the structure, namely, it increases the structural stiffness for lateral loads and increases the total structural damping in the case of dynamic loads. From the diagrams of energy balance, it can be concluded that the effect of hysteretic damped energy is equal to the energy damped by 2.5% of the equivalent modal damping for the analyzed structure. By proper design of the geometric characteristics of its constituent parts, the diagonal damper can provide linear structural behavior with stiffness in the austenite phase under the action of slight earthquakes, nonlinear hysteretic behavior with variable stiffness under moderate earthquakes and hardening with stiffness in martensite phase and control of interstory drifts in the case of strong earthquakes. With its non-linear superelastic behavior, SMA-dampers change the dynamic characteristics of a structure, allowing shifts from the dominant frequency range of the earthquake to which it is exposed.

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J. Radnić[1], A. Harapin[2], D. Matešan[3], N. Grgić[4],
M. Smilović[5], M. Sunara[6], G. Šunjić[7], A. Džolan[8]
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NUMERICAL SIMULATION OF FLUID-STRUCTURE COUPLED PROBLEMS

Summary: *This paper briefly describes the numerical models for the simulation of fluid-structure coupled problems. The applied models are primarily intended to simulate the fluid-structure dynamic interaction in seismic conditions. Models can simulate the most important non-linear effects of plane and spatial structures that are in direct contact with fluid. Some of models' possibilities are illustrated in numerical analyses of the seismic behavior for several practical examples.*

Keywords: *Numerical model, coupled problems, fluid-structure interaction, seismic behavior*

NUMERIČKA SIMULACIJA VEZANIH PROBLEMA FLUID-KONSTRUKCIJA

Rezime: *U radu su ukratko prikazani numerički modeli za simulaciju tzv. vezanog problema interakcije fluida i konstrukcije. Primenjeni modeli su prvenstveno namenjeni za simulaciju interakcije fluida i konstrukcije u seizmičkim uvetima. Modelima se može da simulišu najznačajniji nelinearni efekti ravanskih i prostornih konstrukcija koje su u direktnom doticaju s tečnošću. Neke mogućnosti modela prikazane su kroz numeričku analizu nekoliko praktičnih građevina pod seizmičkim delovanjem.*

Ključne reči: *numerički model, vezani problem, interakcija fluid-konstrukcija, seizmičko delovanje*

[1] Prof. Jure Radnić, full professor, University of Split, Faculty of Civil Engineering, Architecture and Geodesy, Matice hrvatske 15, Split, Croatia, jure.radnic@gradst.hr
[2] Prof. Alen Harapin, full professor, University of Split, Faculty of Civil Engineering, Architecture and Geodesy, Matice hrvatske 15, Split, Croatia, harapin@gradst.hr
[3] Prof. Domagoj Matešan, associate professor, University of Split, Faculty of Civil Engineering, Architecture and Geodesy, Matice hrvatske 15, Split, Croatia, domagoj.matesan@gradst.hr
[4] Nikola Grgić, assistent, University of Split, Faculty of Civil Engineering, Architecture and Geodesy, Matice hrvatske 15, Split, Croatia, nikola.grgic@gradst.hr
[5] Marija Smilović, assistent, University of Split, Faculty of Civil Engineering, Architecture and Geodesy, Matice hrvatske 15, Split, Croatia, marija.smilovic@gradst.hr
[6] Marina Sunara, assistent, University of Split, Faculty of Civil Engineering, Architecture and Geodesy, Matice hrvatske 15, Split, Croatia, marina.sunara@gradst.hr
[7] M. sc. Goran Šunjić, lecturer, *University of Mostar, Faculty of Civil Engineering, Matice hrvatske bb, Mostar, Bosnia and Herzegovina, goran.sunjic@tel.net.ba*
[8] Ante Džolan, assistent, *University of Mostar, Faculty of Civil Engineering, Matice hrvatske bb, Mostar, Bosnia and Herzegovina, ante.dzolan@gfmo.ba*

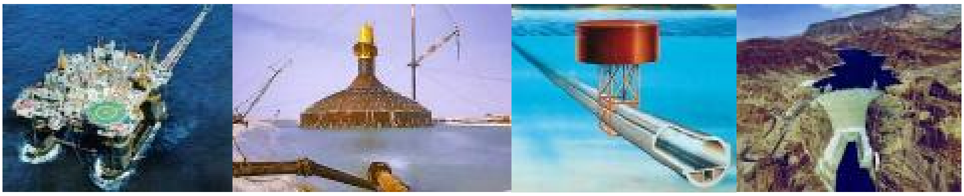
1. INTRODUCTION

Department of Concrete Structures and Bridges at the University of Split, Faculty of Civil Engineering, Architecture and Geodesy, has already more than 25 years in research and developing of models and software for analysis of structures which are in direct contact with fluid. In the last 10 years the researchers from Faculty of Civil Engineering University of Mostar joined with them in improvement the old and development of new techniques and models for this problem.

During past years the developed software was tested on various examples from literature. Also, these researches were basis for many articles which were published in journals and conference proceedings, as well as books' chapters. Also, developed software was used as useful tool for design and calculation of many standard engineering structures of practical proposes.

Structures which are in direct contact with fluid, for example: dams, water tanks (reservoirs), off shore structures, pipelines and water towers etc, can often be encountered in engineering practice. Numerical models for real simulations of these structures have to include the simulation of the fluid-structure interaction to ascertain the real behavior of this complex system. This problem is particularly emphasized under dynamic/seismic conditions and it is commonly referred to as a coupled (multi-field) problem [3, 30].

Some of those structures can be seen on Fig. 1.



Off-shore structure *Underwater tank* *Underwater tunnel* *Dam*

Fig. 1. *Examples of structures in direct contact with fluid*

A coupled multi field problem involves two or more interacting fields, for example: gravity dam with accumulation, water tower full of water etc. Such a problem is time dependent and the state of one field is continuously linked to the state of other fields and neither field can be solved independently from the other. Here, the coupling normally occurs through the differential equation representing different physical phenomena. The coupled fields may be overlapping as in case of seepage and thermo-mechanical problems. On the other hand, the coupled fields may be non-overlapping as in fluid-structure interaction problems as discussed in this work. Here, the coupling occurs due to the imposed boundary condition at the interface. The fields may be coupled with all the other participating fields or with only few of them. The coupling in some problems like seepage may disappear when steady state is reached [3].

Two main approaches exist for the simulation of fluid–structure interaction problems:

– Monolithic approach: the equations governing the fluids' pressures and the displacement of the structure are solved simultaneously, with a single solver

– Partitioned approach: the equations governing the fluids' pressures and the displacement of the structure are solved separately, with two distinct solvers

The monolithic approach requires a code developed for particular combination of physical problems whereas the partitioned approach preserves software modularity because an existing fluid solver and structural solver are coupled. Moreover, the partitioned approach facilitates solution of the fluid equations and the structural equations with different, possibly more efficient techniques which have been developed specifically for either fluid equations or structural equations.

Thus the partitioned approach has various advantages: (i) the resulting model is very modular, (ii) it's easy to make any modifications, (iii) every modification in one field improves the whole model, (iv) the programmer/improver can have knowledge in (only) a single field.

This article briefly describes the developed models and software for numerical modeling of the dynamic interaction of water-structure systems. The described models are suitable for problems with limited fluid motions, such as the response of offshore structures and dams to waves or earthquake.

2. NUMERICAL MODELS

2.1. Introduction

All solutions shown here are based, as it mentioned earlier, on the partitioned scheme where individual fields are solved independently by considering the interaction information transfer between them at every stage of the solution process. This approach allows the usage of ordinary approaches and appropriate mathematical/physical models for separate fields (structure and fluid) that include minor modifications for the influence of interactions.

Developed models and software are based on finite elements method for the spatial discretization and finite differences method for the time discretization of the system [1-3]. For structure and soil the displacement formulation is used, and for fluid the displacement potential formulation is used [3, 30].

In articles [1-5] the basic algorithms for separate fields as well as for fluid-structure interaction problems are given. Furthermore, articles [6-7] present the development of non-linear numerical models for dynamic interactions of the fluid-soil-structure system for plane and spatial problems.

Some non-linear models for structures are described in articles [8-13], and some models for solve eigen value problem are presented in articles [14, 15].

Articles [16-21] present some recent works in this field.

2.2. Numerical model for fluid

2.2.1. Introduction

A fluid is a substance (either a liquid or a gas) that continuously deforms under the action of applied surface stresses. Fluid flow may be classified as either inviscid or viscous. Inviscid flows are frictionless flows characterized by zero viscosity. No real flows are inviscid, but there are numerous fluids and flow situations in which viscous effects can be neglected. Often viscosity effects are confined to thin regions or boundary layers near flow boundaries, and the rest of the flow can be considered frictionless. Inviscid flows may be further classified as either compressible or incompressible, depending on whether density variations are large or relatively unimportant. In this investigation the fluid is considered inviscid and the Eulerian formulation is used [3].

2.2.2. Displacement Potential Formulation

The behavior of fluid, in the most general form, can be expressed by Navier-Stokes equations:

$$\rho\frac{\partial v_i}{\partial t} = \rho R_i - \nabla p + \mu\nabla^2 v_i \tag{1}$$

In this equation ρ represent the density of the fluid, v_i the fluid velocity, p the pressure, R_i the gravitational forces and μ is the dynamic viscosity.

The displacement potential is defined as:

$$\nabla\psi = -\rho\ u_i \tag{2}$$

The u_i are the displacements, which can be expressed:

$$\varepsilon_v = \frac{\partial u_i}{\partial x_i} = \nabla u = -\frac{p}{E} \tag{3}$$

where E represent the bulk modulus.

If the changes in fluid density (ρ) can be neglected, then using (2), Navier-Stokes equations (1) can be reduced, and with neglecting of viscosity and gravitational forces, the equation (1) becomes:

$$\nabla\ddot{\psi} = \nabla p \tag{4}$$

This formulation is called displacement potential formulation, and is in very common use, because it can easily described non-linear fluid behavior, which will be shown later.

Spatial integration of (4) yields:

$$\ddot{\psi} = p \tag{5}$$

and by eliminating p, u and ρ_f from (3), (4) and (5) we obtain:

$$\nabla\ddot{\psi} = \frac{1}{c^2}\psi \tag{6}$$

where c is sound velocity in fluid. The equation (6) represents well known wave equation for inviscid fluid.

2.2.3. Boundary conditions

(i) The prescribed pressure on the free surface may be expressed as:

$$p = \ddot{\psi} = g\frac{\partial \psi}{\partial n} \tag{7}$$

(ii) On moving boundaries, where u_n is the normal displacement of the surface:

$$\partial\psi/\partial n = -\rho\ u_n \tag{8}$$

On a rigid boundary u_n=0, therefore, $\partial\psi/\partial n = 0$. In the case of a base excitation, u_n is composed of the translation at the base and the relative displacement of the structure with respect to the base.

(iii) At radiating boundaries (Sommerfeld's condition):

$$\partial\psi/\partial n = -\dot{\psi}/c \tag{9}$$

2.2.4. Finite element discretization

Using the standard finite element Galerkin process:

$$\begin{aligned} \mathbf{\Psi} &= \mathbf{N}_\psi \mathbf{\Psi} \\ \mathbf{u} &= \mathbf{N}_u \overline{\mathbf{u}} \end{aligned} \tag{10}$$

where $\mathbf{N}_\psi$ and $\mathbf{N}_u$ are the shape functions for displacement potential Y and structural boundary displacement **u**, then it can be shown that:

$$\mathbf{M}_f \ddot{\mathbf{\Psi}} + \mathbf{C}_f \dot{\mathbf{\Psi}} + \mathbf{K}_f \mathbf{\Psi} = \mathbf{f}_f - \rho\ \mathbf{Q}_t(\mathbf{u} + \mathbf{d}) \tag{11}$$

Above equation represent the differential equation of dynamic equilibrium of system, in matrix formulation. In equation (11), $\mathbf{M}_f$ is the mass matrix of the fluid; $\mathbf{C}_f$ is the radiation damping matrix; $\mathbf{K}_f$ is the stiffness matrix of the fluid; $\mathbf{f}_f$ is the vector of applied nodal forces; $\mathbf{Q}_t$ is the fluid-structure interaction matrix; Y is the vector of unknown displacement potential; ρ_f is the density of the fluid; **u** is vector of displacements of the moving boundary relative to the base and **d** is the vector of base excitations.

Above matrices can be expressed:

$$\begin{aligned}
(\mathbf{K}_f)_{ij} &= \int_{V_f}\left[\left(\frac{\partial \mathbf{N}_{\psi i}}{\partial x}\frac{\partial \mathbf{N}_{\psi j}}{\partial x}\right)+\left(\frac{\partial \mathbf{N}_{\psi i}}{\partial y}\frac{\partial \mathbf{N}_{\psi j}}{\partial y}\right)+\left(\frac{\partial \mathbf{N}_{\psi i}}{\partial z}\frac{\partial \mathbf{N}_{\psi j}}{\partial z}\right)\right] dV \\
(\mathbf{C}_f)_{ij} &= (1/c)\int_{\Omega_r}\mathbf{N}_{\psi i}^T\ \mathbf{N}_{\psi j}\ d\Omega \\
(\mathbf{M}_f)_{ij} &= (1/g)\int_{\Omega_f}\mathbf{N}_{\psi i}^T\ \mathbf{N}_{\psi j}\ d\Omega + (1/c^2)\int_{V_f}\mathbf{N}_{\psi i}^T\ \mathbf{N}_{\psi j}\ dV \\
(\mathbf{Q}_t)_{ij} &= \int_{\Omega_i}\mathbf{N}_{ui}^T\ \vec{\mathbf{n}}\,\mathbf{N}_{\psi j}\ d\Omega
\end{aligned} \tag{12}$$

where V_f is the fluid domain; Ω_r is the radiating fluid boundary; Ω_f is the free surface boundary; Ω_r is the interaction boundary. The solution procedure for (8) is described in Section 2.5.

2.2.5. Nonlinear fluid model

In the fluid-structure interaction, nonlinearities are often confined to the structural behavior where the fluid is considered linear. Linear approach - pressure formulation, which presumes unlimited negative pressures in fluid, is natural and very suitable for this approach. However, the displacement potential formulation is more general, and can be used in linear or non-linear models.

The fluid can take some tension which depends upon the concentration and size of micro bubbles present in the fluid. However, if the absolute pressure in a subregion of fluid drops to a value close to vapor pressure of the fluid, bubbles are formed and this physical phenomenon is known as cavitation. Physically, cavitation occurs when the total absolute pressure is less then the vapor pressure of the fluid. Cavitation can cause significant damaging effects on solid surfaces.

Cavitation occur when the total absolute pressure is less then the vapor pressure of the fluid i.e.:

$$p_{abs} = p + p_h + p_a \leq p_v \tag{13}$$

where p_{abs} is the total absolute pressure, p is hydrodynamic pressure, p_h is hydrostatic pressure, p_a is atmospheric pressure and p_v is vapor pressure. This implies that cavitation occurs when the hydrodynamic pressure drops below (p_v-p_s). The vapor pressure of water, for all practical purposes, can be taken from 0.02 to 0.03 MPa.

The changes which the fluid may undergo under hydrodynamic excitation are a direct function of the mass dilatation s, defined as:

$$s = \nabla^T(\rho_f \mathbf{u}_f) = \mathrm{Div}(\rho_f \mathbf{u}_f) \tag{14}$$

where $\mathbf{u}_f$ is displacement of the fluid relative to the initial static state. As long as p_{abs} is greater then the vapor pressure p_v, a linear relation between s and p is assumed:

$$p = -\alpha\, s\,; \qquad \alpha = c^2 \tag{15}$$

where c is the acoustic velocity of the fluid.

If equation 13 is true, cavitation occurs and the stage of linear fluid is no longer valid. A simple fluid model can be represented by the bilinear pressure-mass dilatation relation shown in Fig. 2. Cavitation, therefore, commences when the following condition is reached:

$$s \geq (p_h + p_a - p_v)/c^2 \tag{16}$$

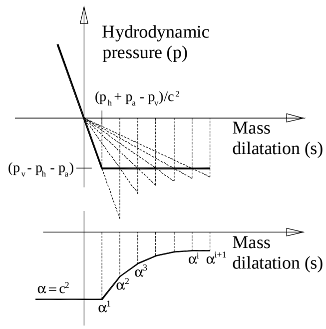


***Fig. 2.** Relation between mass dilatation and hydrodynamic pressure*

If cavitation occurs, the iteration procedure, shown in Fig. 2, has to be performed to obtain the value of the coefficient α.

2.3. Numerical model for structure

2.3.1. Introduction

For dynamic equilibrium of a solid body in motion the Principle of virtual work can be used to write the equations independent of the material behavior:

$$\int_{\Omega}(\delta\underline{\varepsilon})^{T}\underline{\sigma}d\Omega-\int_{\Omega}(\delta\underline{u})^{T}(b-\rho_s\ddot{\underline{u}}-\mu'\dot{\underline{u}})d\Omega-\int_{\Gamma_t}(\delta\underline{u})^{T}td\Gamma=0 \tag{17}$$

where $\delta\underline{u}$ is the vector of virtual displacements, $\delta\underline{\varepsilon}$ is the vector of associated virtual strains, b is the vector of applied body forces, t is the vector of surface tractions, $\underline{\sigma}$ is the vector of stresses, ρ_s is the mass density, μ' is the damping parameter and a dot refers to differentiation with respect to time. The domain of interest Ω has two boundaries: Γ_t on which boundary tractions t are specified and Γ_u on which displacements are specified.

In dynamic analysis, the finite element method can be applied in both analyses: for space and time. However, it is the general practice to use finite elements in space and finite differences in time [1, 2, 3]. This approach is adopted in this work. Here, the displacement formulation is used because of its simplicity, generality and good numerical properties. For a finite element representation, the displacements and strains and also their virtual counterparts are given by the following relationships:

$$\begin{array}{ll}\underline{u}=\mathbf{N}\mathbf{u} & \partial\underline{u}=\mathbf{N}\partial\mathbf{u}\\ \underline{\varepsilon}=\mathbf{B}\mathbf{u} & \partial\underline{\varepsilon}=\mathbf{B}\partial\mathbf{u}\\ \underline{\sigma}=\mathbf{D}\underline{\varepsilon}=\mathbf{D}\mathbf{B}\mathbf{u} & \partial\underline{\sigma}=\mathbf{D}\partial\underline{\varepsilon}=\mathbf{D}\mathbf{B}\partial\mathbf{u}\end{array} \tag{18}$$

where $\mathbf{u}$ is the vector of nodal displacements, $\partial\mathbf{u}$ is the vector of virtual nodal variables, $\mathbf{N}$ is the matrix of global shape functions, $\mathbf{B}$ is the global strain-displacement matrix and $\mathbf{D}$ is the global constitutive matrix.

If (18) are substituted into (17), and if we note that the resulting equation is true for any set of virtual displacements, then the following equation can be obtained:

$$\mathbf{M}_s\ddot{u}_i+\mathbf{C}_s\dot{u}_i+\mathbf{R}_s(u_i)=\mathbf{f}_s \tag{19}$$

where:

$$
\begin{aligned}
(\mathbf{M}_s)_{kj} &= \int_{\Omega_s} \mathbf{N}_{sk}^T \rho_s \mathbf{N}_{sj} \, d\Omega \\
(\mathbf{C}_s)_{kj} &= \int_{\Omega_s} \mathbf{N}_{sk}^T \mu' \mathbf{N}_{sj} \, d\Omega \\
\mathbf{R}_s(u_i) &= \int_{\Omega_s} \mathbf{B}^T(u_i)\sigma_i \, d\Omega \\
(\mathbf{f}_s)_i &= \int_{\Omega_s} \mathbf{N}_{sk}^T \mathbf{b}_i \, d\Omega + \int_{\Gamma_t} \mathbf{N}_{sk}^T \mathbf{t}_i \, d\Gamma
\end{aligned}
\tag{20}
$$

where **N** is the matrix of global shape functions.

For real structures, relationship strain-deformation is generally non-linear:

$$
\varepsilon = \mathbf{B}\,u \quad ; \quad \mathbf{B} = \mathbf{B}(u) \tag{21}
$$

which represent so called geometrical nonlinearity. In fact, because of geometry transformation, array **B** is not linear but depends on system displacement. Relationship ε-u is known as model of geometry.

Relationship between stress (σ) and strain (ε) is also generally nonlinear, and represent material nonlinearity. It is usually called material model or constitutive relationship.

In all calculation, origin point is linear elastic behavior. For example, for a plane strain state and isotropic elastic material, matrix **D** is given as:

$$
\mathbf{D} = \frac{E}{(1+\nu)(1-2\nu)} \begin{bmatrix} (1-\nu) & \nu & 0 \\ \nu & (1-\nu) & 0 \\ 0 & 0 & \dfrac{(1-2\nu)}{2} \end{bmatrix} \tag{22}
$$

where E and ν is well known Young’s elasticity modulus and Poisson’s ration, respectively.

2.3.2. Structure nonlinearity

Two main types of nonlinearity which occur in structures are the geometrical and the material nonlinearity, as mentioned earlier.

Geometric nonlinearity

Structures may undergo either (i) small deformations which are negligible compared to the dimensions of the body or (ii) large or finite deformations in which the theory of small deformations is no longer valid. The second type of deformation is termed geometrically nonlinear problems and special procedure is required for their solutions. In many real civil engineering structures geometric nonlinearity can be neglected.

Material nonlinearity

Deformations may be divided in recoverable or unrecoverable. A recoverable deformation implies that when the load is removed, the solid body retains its original position, whereas an unrecoverable deformation implies that when the load is removed, the solid body exhibit permanent deformations. All real materials have nonlinear behavior, but in many cases material can be consider linear.

Many types of material model were developed to represent the variety of behavior such as linear elastic, nonlinear elastic, elasto-plastic, visco-elastic, visco-plastic, creep, cracking or fracture etc. Different hardening laws such as isotropic and kinematic hardening also developed in the plastic and visco-plastic models. Apart from the linear elastic models, all of these representations are nonlinear in some sense.

2.3.3. Numerical model for reinforced concrete structures

A special material model was developed for the simulation of reinforced concrete structures [11-15, 22, 24]. It includes the most important nonlinear effects of reinforced concrete behavior: yielding in compression and opening and propagation of cracks in tension, with tensile and shear stiffness of cracked concrete, as well as nonlinear behavior of reinforced steel. In every integration point of every element, simulation of cracks opening and closing is possible.

Special materials models were developed for plane (2D) problems, spatial (3D) problems and for shell structures. These models will be only briefly discussed here, and for further reading can be found in quoted references.

Yielding in compression

There is still no accepted constitutive model which could describe the complexity of concrete behavior under different stress states. Various models have been proposed for describing the stress-strain relation under multiaxial static stresses. Each of these has certain advantages and disadvantages, considering the analyzed problem. The simplified models, based on small number of the basic concrete parameters, are better for engineering practice because complex models based on greater number of parameters, cannot be used.

A rather simple concrete model is preferred (Fig. 3), intended for ordinary engineering practice, founded on basic concrete parameters (uniaxial compressive and tensile strength, the modulus of elasticity and Poisson's ratio) which should be known for other purposes anyway.

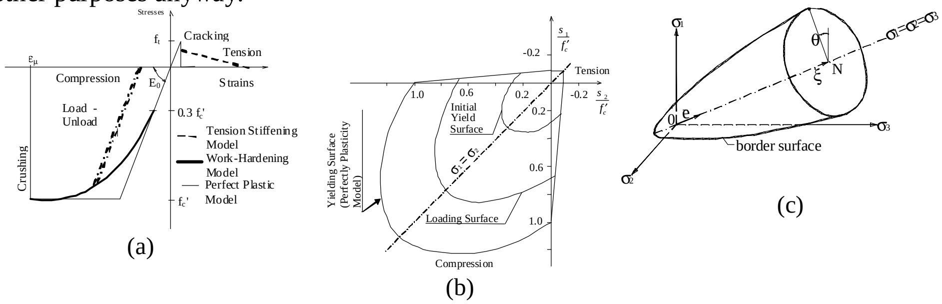


Fig. 3. *Non-linear behavior of concrete in compression (yielding) – 1D (a), 2D (b) and 3D (c) problems*

Modeling of cracks

The graphic interpretation of the concrete model under tension, for 2D problems, is presented in Fig. 4. Linear-elastic behavior has been assumed until tensile strength of concrete f_t' is reached. After that, the first crack in concrete is assumed to appear perpendicular to main tensile stress.

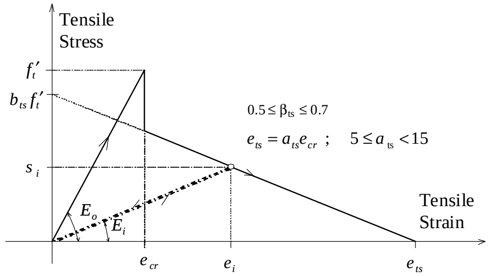
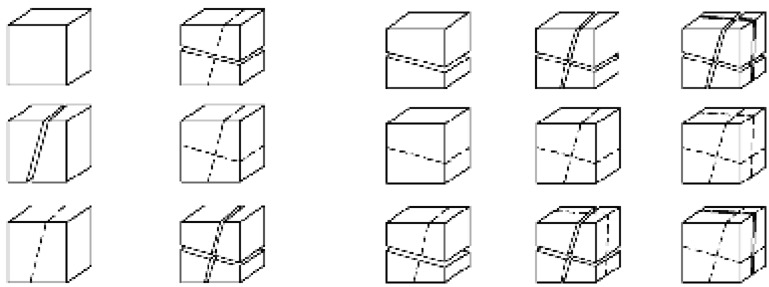


Fig. 4. *Graphical representation of concrete behavior in tension*

It is also assumed that, even after cracking, the concrete remains as a continuum. A model of the so-called smeared cracks has been used. It has been adopted that after the occurrence of the first crack, its position and direction do not change after subsequent changes of loading. Hence, the so-called model of fixed orthogonal cracks has been used.

After the occurrence of cracks, concrete becomes anisotropic and the cracks directions define the main axis of anisotropy. Both, partial and complete crack closings at unloading have been modeled, as well as new opening of the previously developed cracks under repeated loading. The potential states of cracks, for 2D and 3D problems, are presented in Fig. 5.



(i) 2D (plane) problems *(ii) 3D (spatial) problems*

Fig. 5. *Schematic presentation of possible cracks pattern*

The stiffness contribution of uncracked concrete between cracks was simulated by gradually decreasing the component of tensile stress perpendicular to the crack plane.

Modeling of reinforcement

The reinforcement model is graphically presented in Fig. 6, and the adopted stress-strain relationship for steel is presented in Fig. 7.

The reinforcement bars are modeled as separate beam elements incorporated in base elements (Fig. 6a and 6b). The stresses can occur only in the bars direction. It was assumed that the concrete and reinforcement displacements were entirely compatible. Detail description of this model can be found in [11, 24].

A bi-linear stress-strain relation was used to describe the steel behavior, both in compression and in tension. For unloading conditions, a linear behavior with the initial modulus of elasticity was assumed. The bars collapse occurs when the strain in their direction exceeds the limit value ε_{au} .

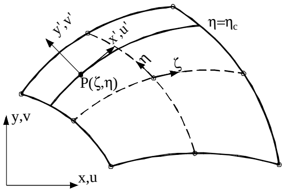


Fig. 6a. *Modeling of Reinforcement for 2D problems*

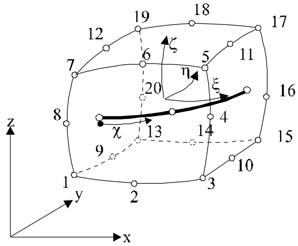


Fig. 6b. *Modeling of Reinforcement for 3D problems*

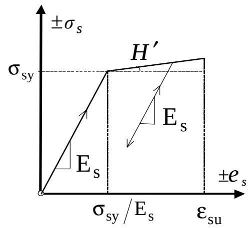


Fig. 7. *Stress-Strain Relation for Steel*

Model for composite joint

Interface elements (Fig. 8) are used for simulation of continuous connection between two composite members [14, 15, 22]. They physically represents connection surface of base composite elements with determinate little width w (Fig. 9). These elements allow the simulation of sliding, splitting, detachment and impress on contact surface, according on acquired models of the interface element material behavior.

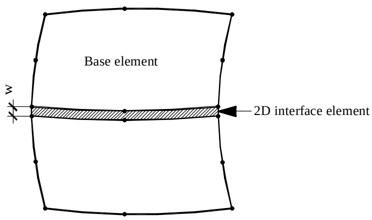


Fig. 8. *Interface element*

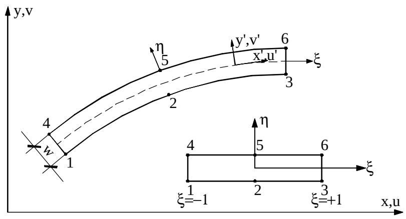


Fig. 9. *Interface element in global (x,y) and local (x,h) coordinate system*

2.4. Equation for Coupled Fields Motions and Spatial Discretization

2.4.4. Coupled Fields Motions

Behavior of the fluid-structure system (structure includes the structure itself as well as the surrounding soil) in dynamic load conditions, can be expressed with two second order differential equations [1, 3, 5-7]. If we use the displacement formulation for the structure and the displacement potential formulation for the fluid, dynamic equilibrium equations can be expressed in the following form:

$$\begin{aligned}&\mathbf{M}_s\ddot{\mathbf{u}}+\mathbf{C}_s\dot{\mathbf{u}}+\mathbf{K}_s\mathbf{u}=\mathbf{f}_s-\mathbf{M}_s\ddot{\mathbf{u}}_g+\mathbf{f}_{cs}\\&\mathbf{M}_f\ddot{\boldsymbol{\Psi}}+\mathbf{C}_f\dot{\boldsymbol{\Psi}}+\mathbf{K}_f\boldsymbol{\Psi}=\mathbf{f}_f+\mathbf{f}_{cf}\end{aligned} \tag{23}$$

where

$$\mathbf{f}_{cs} = \mathbf{Q}\,\boldsymbol{\Psi}$$
$$\mathbf{f}_{cf} = -\rho_f\ \mathbf{Q}^T\left(\mathbf{u}+\mathbf{d}\right) \tag{24}$$

In the above equations $\mathbf{M}_s$, $\mathbf{C}_s$ and $\mathbf{K}_s$ represent mass, damping and stiffness matrices for structure, and $\mathbf{M}_f$, $\mathbf{C}_f$ and $\mathbf{K}_f$ represent mass, damping and stiffness matrices for fluid. Vectors $\mathbf{u}, \dot{\mathbf{u}}, \ddot{\mathbf{u}}$ represent structure's displacements and displacement's derivations (velocities and accelerations) and $\psi, \dot{\psi}, \ddot{\psi}$ are the displacement potential and associated derivations. $\mathbf{Q}$ is the interaction matrix between structure and fluid.

Interaction between structure and base soil is modeled indirectly by contact elements in the connection surface. In fact, by applying the appropriate material model for contact elements, various effects in the contact surface can be simulated, such as: separating, embedment and sliding.

Fluid-structure interaction surface with fluid and structure elements is shown in Fig. 10. Interaction matrix $\mathbf{Q}$ includes only the surface integration and is defined as:

$$\left(\mathbf{Q}\right)_{ij} = \int_{\Gamma_i} \mathbf{N}_{ui}^T\, \vec{n}\, \mathbf{N}_{pj}\, d\Gamma_i \tag{25}$$

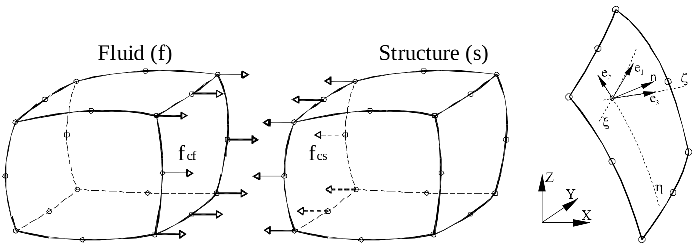


Fig. 10. *Fluid-structure interaction surface and unit norm*

2.4.5. Spatial Discretization

As mentioned before, for plane (2D) problems, 8-node and 9-node isoparametric elements are used for fluid and structure. For spatial (3D) problems, 20-node and 27-node ("brick") elements are used for fluid and structure. For thin curved structures, 8-node or 9-node degenerated shell elements can be used for structure and 20 or 27-node spatial element for fluid. Those shell elements are free of membrane and shear locking, according to [10].

For the simulation of connections between the foundation soil and the structure, 6-node contact elements can be used for plane and 16 or 18 nodes for spatial problems.

Elements for 3D (spatial) problems are presented on Fig. 11, and elements for shell structures on Fig. 12.

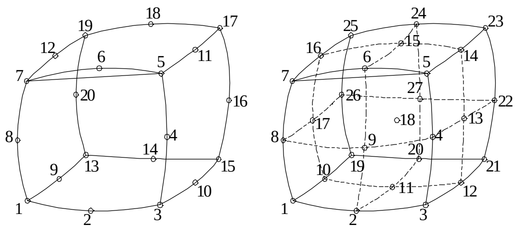


Fig. 11. Elements for spatial (3D) discretization

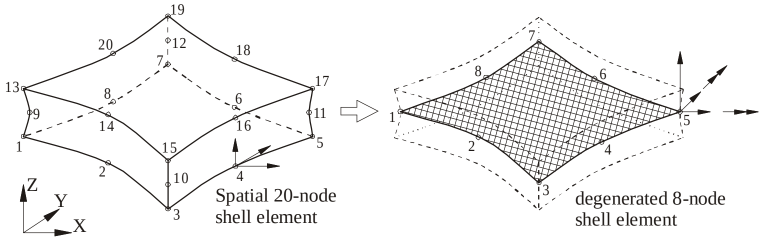


Fig. 12. Elements for shell structures

2.5. Solution Concept for the Dynamic Fluid-Structure Interaction Problem

Direct solution of the equation system (23) requires large computer capacity. So, the previously described partitioned scheme is ideal for this kind of problems. In that approach for every increment of the imposed load and every non-linear problem iteration step, each field is solved separately by including interaction forces on the contact surface between fluid and structure. Presentation of the solution scheme is given in Fig. 13.

In the presented approach, structure is solved first and fluid second. This approach allows the developed independent models to be used for each field (partition analysis), with additional calculations of the interaction forces only. Thus, in the fluid-structure interaction model, all non-linear effects of material and geometry, that are present in a particular field, can also be simulated in the coupled problem. In Fig. 13 for time integration, explicit-implicit algorithm developed by Hughes [1, 2] is used.

Predicted values $\mathbf{u}, \dot{\mathbf{u}}, \ddot{\mathbf{u}}$ and $\psi, \dot{\psi}, \ddot{\psi}$ at the beginning of every time step are corrected at the end of the same time step. For convergence control of the iterative procedure, the increase of the structure's displacements in comparison with current total displacements and the increase of the fluid's displacements potential in comparison with the current total displacements potential are simultaneously monitored. Various options of the Newton-Raphson method are used to solve the non-linear equations.

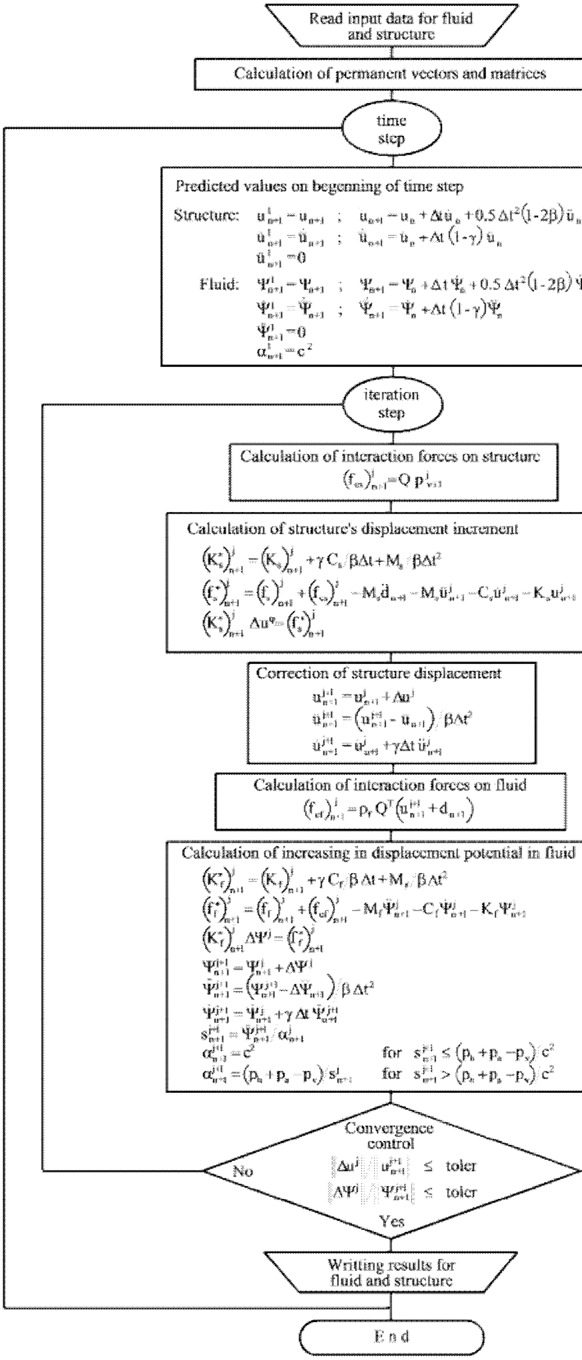


Fig. 13. *Flow chart for the solution of the fluid-structure coupled problem*

2.6. Additional Model Possibilities

Solution of eigen value problem is also based on the partition solution scheme, with the Wilson-Yuan-Dickens (so-called WYD) method [4, 8, 9, 23] as the solution procedure. In dynamic problems, as well as in the structures' response calculations, eigen values and eigen vectors are needed to know the vibration characteristics (determination of time step length).

Radiation damping can be simulated on artificially formed fluid boundaries, as well as radiation and multi-axis structure damping for structure [2, 5, 6].

Simulation of fluid pressure in open cracks of a structure is included by additional nodal forces in finite elements that have cracks that fluid can get into.

As external dynamic forces, various time-dependant dynamic loads can be applied. Also, seismic base excitations can be applied to the model.

Complex system structure-soil-fluid can be loaded with arbitrary seismic excitation in direction of three main axes, which can be given by series of measured (accelerograph) or generated input data in discrete time steps.

The computer programs (software) are equipped with adaptive and user-friendly postprocessor for the visualization of various numerical results.

3. EXAMPLES

What follows are four complex practical examples which illustrate some possibilities of the developed models and the applied software.

3.1. Example 1 – Koyna Dam

The Koyna Dam is the largest dams in Maharashtra, India. It is a rubble-concrete dam constructed on Koyna River. It is located in Koyna Nagar, Satara district, nestled in the Western Ghats on the state highway between Chiplun and Karad. The dam, built in 1963, is one of the largest dams in India (Fig. 14). It is an atypical gravitational dam, with a crest length of 853.44 meters. It consists of 56 dilatation blocks of 17.07 m in thickness. Spillway length is 91.44 m.

Fig. 15 presents the main geometric data of the Koyna dam. Detailed information of the dam geometry, construction materials, damages (cracks) and earthquake characteristics can be found in [25, 26].

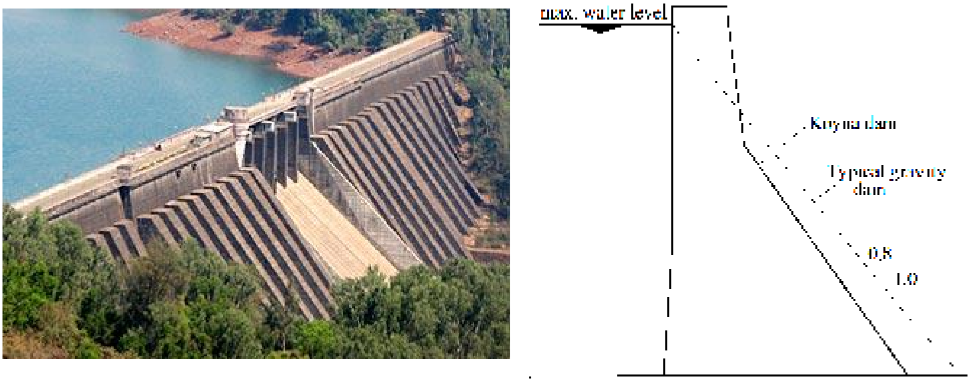


***Fig. 14.** Koyna dam, photograph [33] and comparison with typical gravity dam [26]*

During construction, two accelerographs were embedded in the dam, and in one of them, in 1967, an earthquake that caused several significant damages was registred. Dominant damages of the dam manifested as horizontal cracks on the up-stream and down-stream sides on many blocks, especially on lines where the total thicknes of the dam changes.

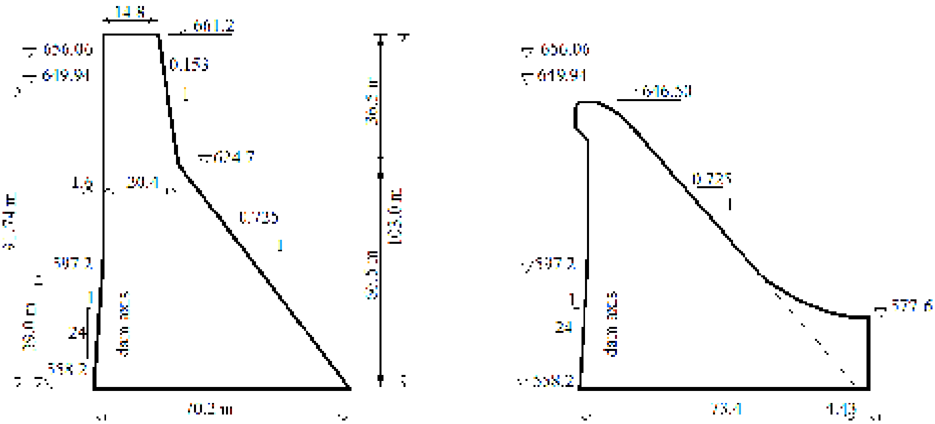


***Fig. 15.** Koyna dam – some geometrical data (all dimensions are in meters) [26]:*
(a) cross-section through dam body; (b) cross-section through spillway

Spatial discretization of the system is presented in Fig. 16, and the used material characteristics are presented in Tab. 1. The behavior of the water-dam-soil system was analyzed for the previously mentioned registered earthquake. The system was analyzed separately for the linear and for the non-linear (cavitation) fluid model, with the following structure models: a) non-linear model without including the fluid pressure in open structure cracks (no FPC), b) non-linear model which includes the fluid pressure in open structure cracks (FPC).

Some numerical results are presented in Figs. 17 and 18. Other results can be found in [25, 26]. Dam damages calculated through numerical models match the real crack pattern very well.

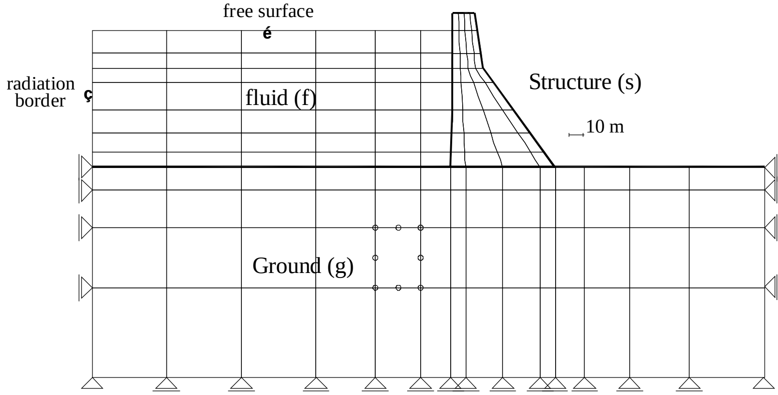


Fig. 16. *Spatial discretization of Koyna dam*

Fluid (water)	Structure (concrete dam)	Ground
$\rho_f = 1019.0\ \mathrm{kg/m^3}$ $c = 1439.0\ \mathrm{m/s}$ $p_a = 0.10\ \mathrm{MPa}$ $p_v = 0$	$E_s = 31640.0\ \mathrm{MPa}$ $\nu_s = 0.2$ $\rho_s = 2690.0\ \mathrm{kg/m^3}$ $(f'_c)_s = 24.6\ \mathrm{MPa}$ $(f'_t)_s = 2.46\ \mathrm{MPa}$ $(\varepsilon_{cu})_s = 0.003$ $(\max \varepsilon_t)_s = (\max \varepsilon_{sh})_s = 0.0012$	$E_g = 18000.0\ \mathrm{MPa}$ $\nu_g = 0.2$ $\rho_g = 1830.0\ \mathrm{kg/m^3}$ $(f'_c)_g = 20.0\ \mathrm{MPa}$; $(f'_t)_g = 2.0\ \mathrm{MPa}$ $(\varepsilon_{cu})_g = 0.003$; $(\max \varepsilon_t)_g = (\max \varepsilon_{sh})_g = 0.0017$

Table 1. *Material characteristics of the Koyna dam system*

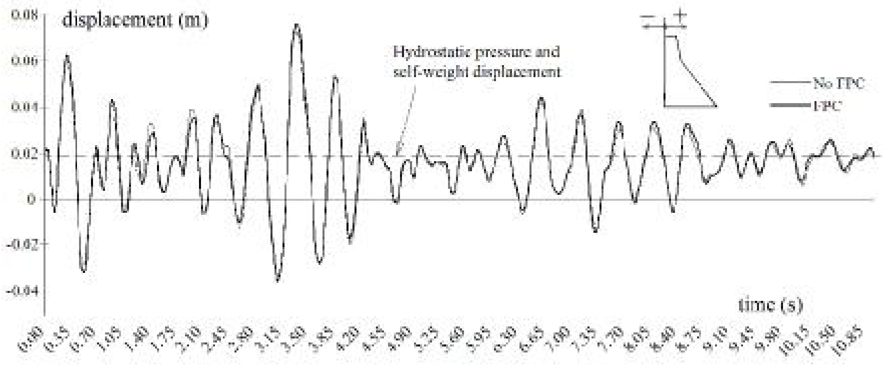


Fig. 17. *Horizontal displacement of the Koyna dam crest for non-linear fluid model*

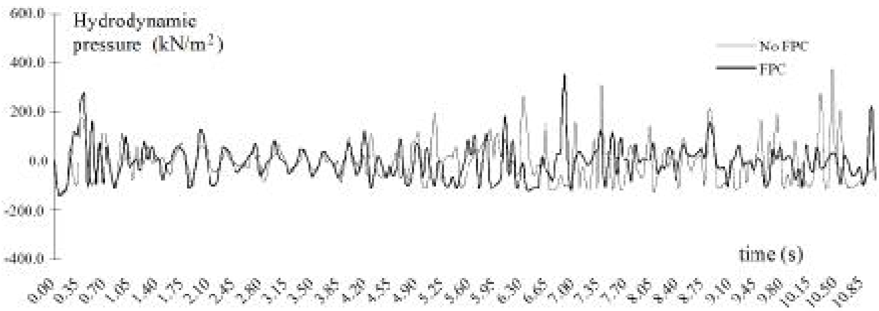


Fig. 18. *Hydrodynamic pressure at the bottom of the Koyna dam*

3.2. Example 2 – Grančarevo Dam

The Grančarevo Arch Dam in Bosnia and Herzegovina (Figs. 19, 20, 22) is a double-curvature concrete dam with a perimetral joint. The dam was constructed in 1968. The height of the dam is 123 meters and the crest length is 439 meters. Its bottom thickness is 27 meters and its top thickness 4.6 meters. The dam's foundation dig was 230.000 m^3 and the volume of poured concrete was 376.000 m^3. The head of the dam is 100 meters. The dam created the Bileća reservoir with a maximum water depth of 51 meters and an available storage capacity of 1100 million cubic meters. The Bileća reservoir is the largest storage lake in Balkan. Its dimensions are: total storage volume: 1280 hm^3 and surface of the reservoir on normal top water level: 2764 ha. Geometrical data tables (on Fig. 21) show basic geometrical characteristics for individual arches some of which are shown in Fig. 20 and 21. Other detailed information about dam can be found in [27-31].

Fig. 19. *Grančarevo Arch Dam*

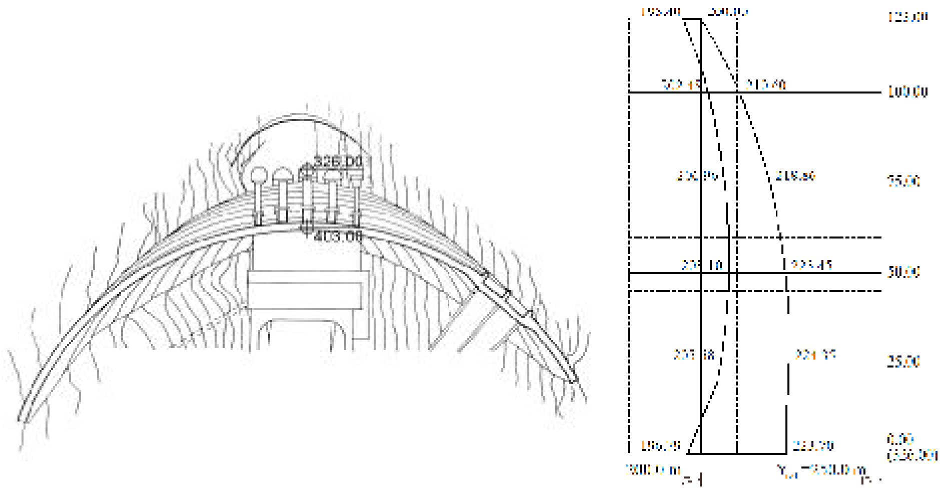


Fig. 20. *Plan of the dam's body with land topology (left) and cross section through central cantilever (right) [29]*

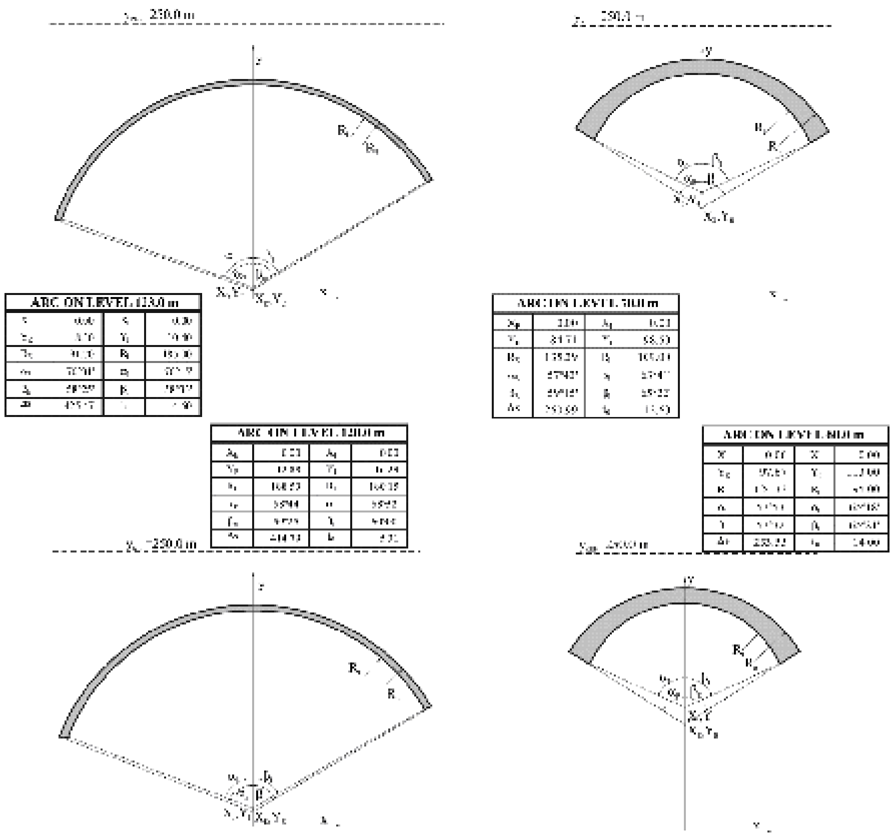


Fig. 21. *Geometry of some arch elements of the Grančarevo dam [29]*

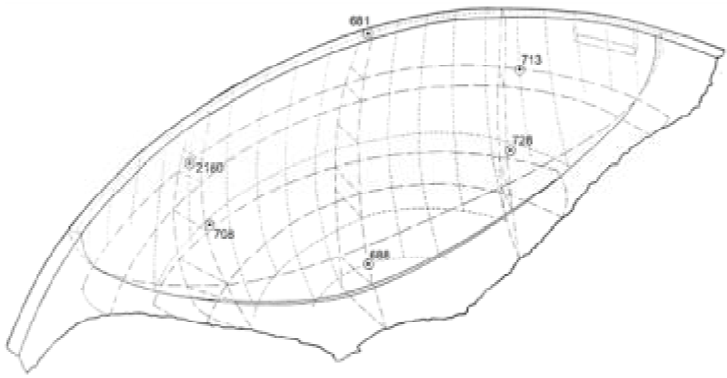


__Fig. 22.__ Positions of accelerographs in the Grančarevo dam body [29]

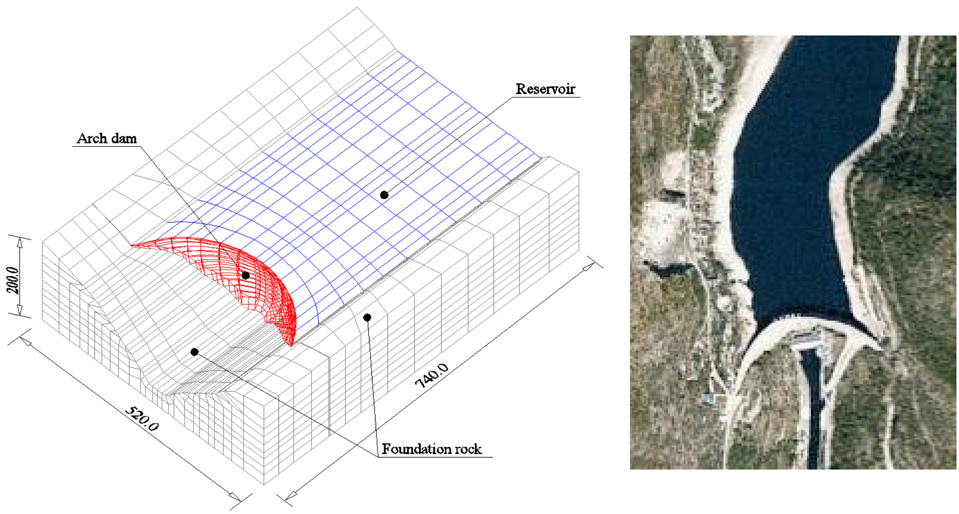


__Fig. 23.__ Finite element mesh of the Grančarevo dam–water–foundation rock interaction system – axonometric view and aerial view (Google map)

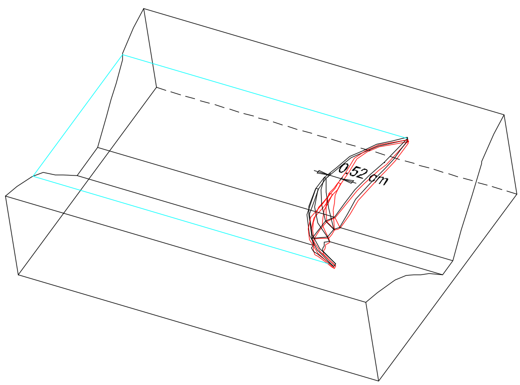


__Fig. 24.__ Displacement of the dam's self weight and hydrostatic pressure of water

The Institute of Earthquake Engineering and Engineering Seismology (IZIIS-Skopje, Macedonia) monitored the dam and performed several numerical simulations on different models, which were compared with results in situ [29]. All applied models included only the dam (structure), and water was treated as an additional mass on structure.

The complex model of the water-dam-foundation rock system is presented in Fig. 23. Material characteristics are given in Table 2. For the dam and foundation rock 27-node ("brick") elements are used, as well as for the accumulation (fluid). For the simulation of connections between the foundation rock and the structure 18 nodes contact elements are used.

The dam is first analyzed for the self weight and hydrostatic pressure of accumulated water. The water level in the accumulation (reservoir) is at a relative elevation of 120.0 m (3.0 m below the crest). Displacements' field for this load is relatively small. The displacement in the crest of the dam is 0.52 cm in the horizontal direction (perpendicular to the crest) and 0.62 cm in the vertical direction. These results are in very good agreement with the results of the dam monitoring (0.58 cm in the horizontal direction and 0.68 cm in the vertical direction). Fig. 7 shows the displacement of the dam's self weight and hydrostatic pressure of water.

The behavior of coupled system dam-water-foundation rock was analyzed for the registered earthquake from 1986, [29, 30] – Fig. 25.

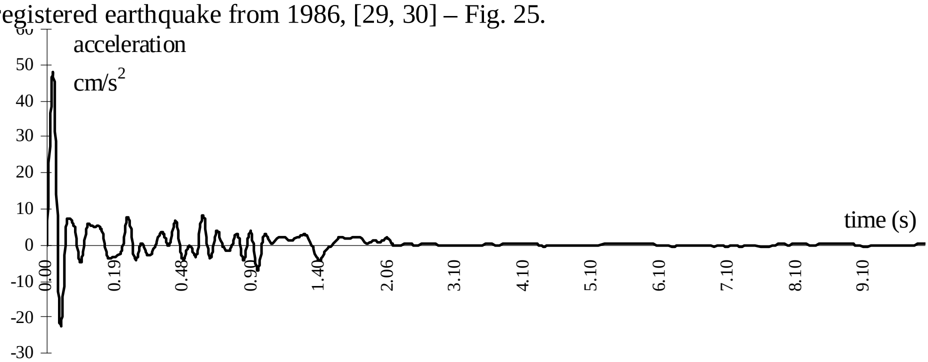


Fig. 25. *Registered earthquake in accelerograph 688 in Fig. 21*

Fluid (water)	Structure (concrete dam)	Foundation rock
$\rho_f = 981\,kg/m^3$ $c = 1440.0\,m/s$	$E_c = 33000.0\,MPa$ $\nu_c = 0.15$ $\rho_c = 2400.0\,kg/m^3$ $f_{ck} = 25\,MPa$; $f_{ct} = 2.5MPa$ $\varepsilon_t = 0.083$; $\varepsilon_{t,max} = \varepsilon_{s,max} = 1.7$	$E_r = 80\,GN/m^2$ $\nu_r = 0.2$ $\rho_r = 2620.0\,kg/m^3$ $f_{rk} = 12.0\,MPa$; $f_{rt} = 1.2\,MPa$

Table 2. *Material characteristics of Grančarevo dam system*

To determine the dynamic characteristic of the dam, analysis of the structure oscillation (modal analysis) was conducted [3, 7, 11]. The analysis was performed for the coupled system: dam, rock and water (full accumulation). The results of analysis, Fig. 26, are showing for the first four eigenvectors. It can be seen very good agreement with the results presented in the literature [31].

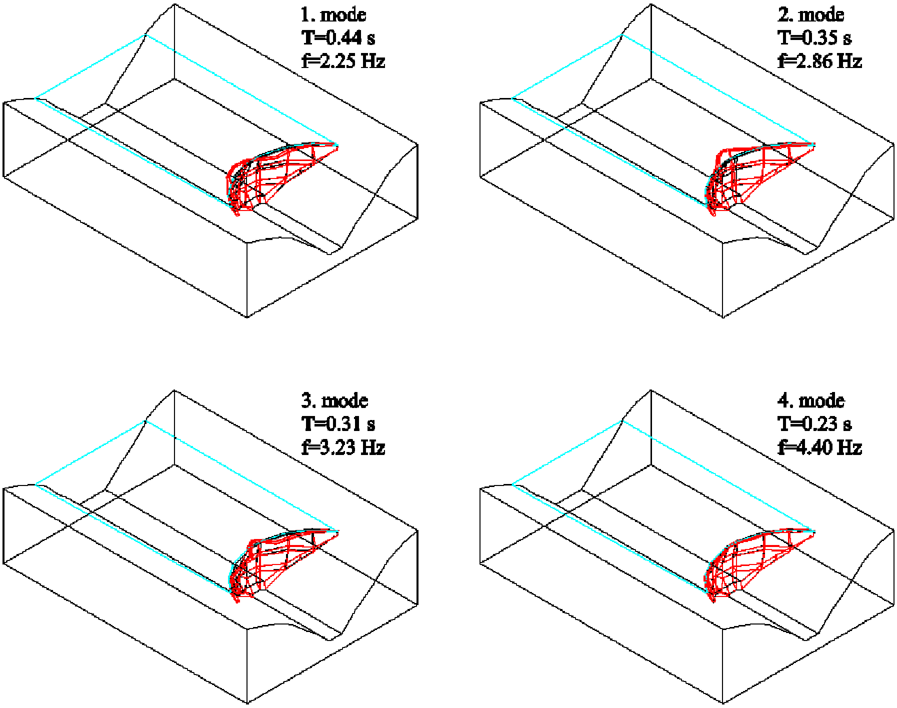


Fig. 26. *Oscillation periods, frequencies and eigenvectors for dam-accumulation system*

Based on the calculated data, time step for dynamic analysis was adopted as $\Delta t=0.002$ s, which represents approximately 1/200 of first period (T1), and also well approximate given accelerogram (Fig. 25). Time integration of equations of motion was performed by implicit method for water and construction.

Than, the registered accelerations on the bottom of the dam (accelerograph 688, Fig. 22) were taken as imposed accelerations of the foundation's rock (excitation) along the canyon (perpendicular to the dam axis). The maximal registered imposed acceleration was 47.8 cm/s^2. The maximal registered acceleration on the dam was 145.1 cm/s^2 (accelerograph 681, Fig. 22), and the maximal acceleration obtained through the numerical model was 149.3 cm/s^2 (Fig. 27). Applied excitations cause hydrodynamic pressures that are always less than the hydrostatic pressure, so cavitation did not occur.

Some calculation results are presented in Figs. 27, 28 and 29. Fig. 27 presents accelerations of the Grančarevo dam crest in time, Fig. 28 presents displacement of the Grančarevo dam crest in time and Fig. 29 presents hydrodynamic pressures on the bottom of the Grančarevo dam in time. Other results can be found in [29, 30, 31].

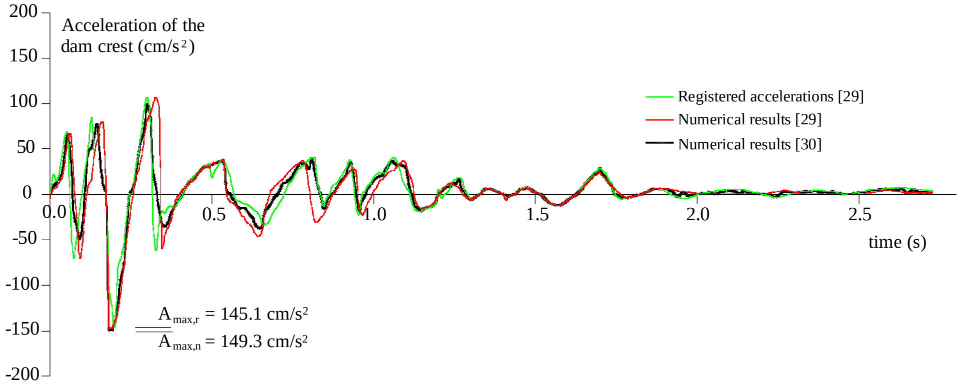


Fig. 27. *Accelerations of the Grančarevo dam crest*

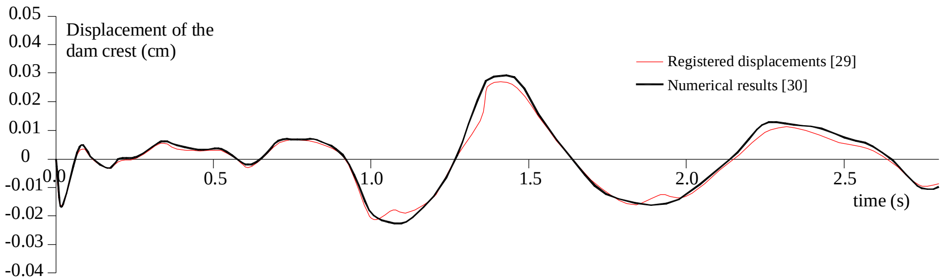


Fig. 28. *Displacement of the Grančarevo dam crest*

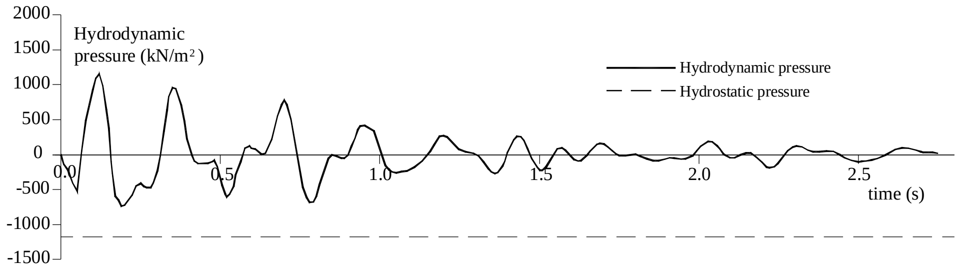


Fig. 29. *Hydrodynamic pressures on the bottom of the Grančarevo dam*

3.3. Example 3 – Underwater tank "Khazzan"

Khazzan (meaning: "To Store" in Arabic) was the name given to the tanks designed and built in late 1960s to store Dubai's Oil by Chicago Bridge and Iron Company. Dubai's Khazzans are unique in that they store Dubai's Oil under the Sea. Khazzan is a 500.000 barrel (80.000 m^3) oil storage tank (Fig. 30). The 15.000 ton structure is 80 m in diameter on bottom and 8 m diameter on top, and about 82.0 m in height. Sea depth is about 70 m, so tank crest is 12 m under sea level. It has no bottom and operates on the water displacement principle. It is filled by placing oil in the tank above water where the additional weight of the oil on the water creates an imbalance in pressure. This force pressures the water out of the tank through the openings in the wall at the bottom.

Initial construction was in a shallow, dewatered basin. When the tank was sufficiently complete so that it could float as a single unit, using compressed air, the basin was flooded, and the tank, a bottomless hemisphere, was moved laterally into a deeper basin and seated on its floor by releasing the internal air pressure. The structure was then fully completed. Floated once again by filling the tank with compressed air, it was towed to the site and positioned by mooring lines, and the air was gradually released. It was allowed to slowly sink further and settle on the seafloor [34].

The geometrical characteristic of the model were taken form [32, 34, 35]. Fig. 31 shows the vertical section of the oil tank with the adjacent part of the surrounding sea.

(i) – Construction on shallow dewatered basin on shore

(ii) – Towing to the site

Fig. 30. *Oil-storage tank "Khazzan" [34, 35]*

Fluid – sea water	Fluid – oil	Structure –steel (tank)
$\rho_f = 1000.0\ \mathrm{kg/m^3}$ $c = 1430.0\ \mathrm{m/s}$	$\rho_n = 900.0\ \mathrm{kg/m^3}$ $c = 1300.0\ \mathrm{m/s}$	$E = 210\ \mathrm{GN/m^2}$ $\nu_t = 0.3$ $\rho_a = 78.5\ \mathrm{kN/m^3}$

Table 3. *Material characteristics of the Khazzan store tank*

The sea-oil-tank system was modeled with the spatial 3D model, shown in Figs. 31 and 32. Spatial discretization of the liquid was done with 27-node 3D brick elements, and the structure with 9-node shell elements.

The harmonic ground acceleration with the period of 0.207 s (which is in accordance with the first period of the sea-oil-tank system), and amplitudes of 0.3 g for the horizontal and 0.2 g for the vertical acceleration component is accepted. The material characteristics are shown in Table 3. Implicit time integration (Δt=0.002 s) and diagonal mass matrix were used.

Some results are shown in Figs. 33-36, and a detailed description of the model and results can be found in [32].

Fig. 33 shows the hydrodynamic water pressure for the horizontal seismic action in the specified points on the tank surface and Fig. 34 shows the horizontal displacements of the specified points of the tank for the horizontal seismic action. Fig. 35 shows the maximal displacements of the Kazzan oil-storage tank for the horizontal and the vertical seismic action.

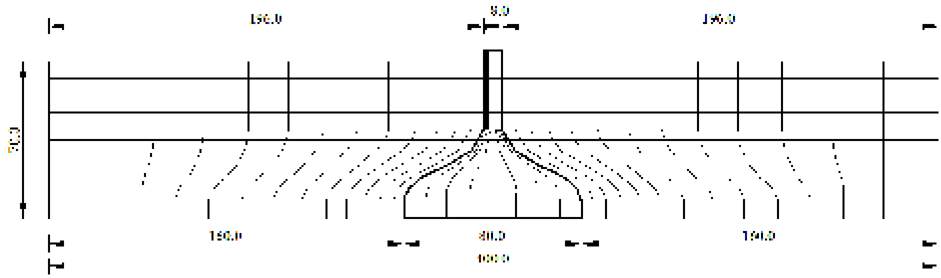
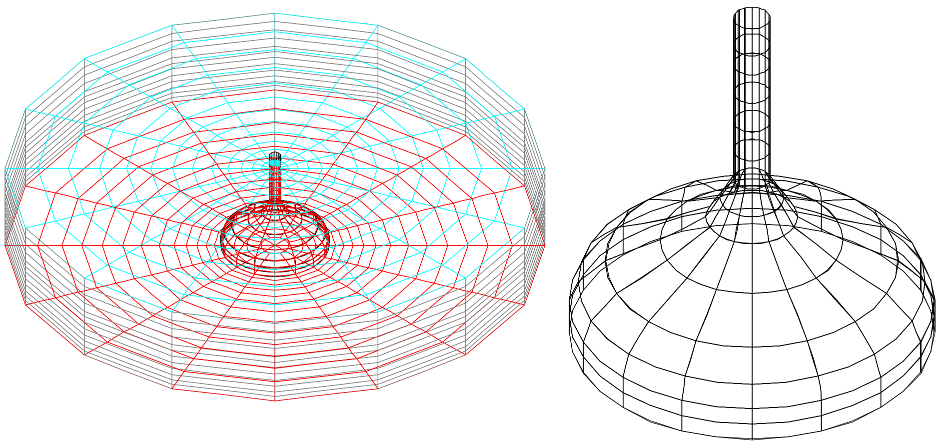


Fig. 31. *Spatial discretization of the Kazzan tank, the oil in the tank and the surrounding sea water – Longitudinal section of the finite element mesh (all dimensions in meters)*



(i) – 3D view of finite element mesh (3D “brick” elements for fluid and shell elements for tank)

(ii) – 3D view of oil-storage tank finite element mesh (shell elements)

Fig. 32. *Spatial discretization of the Kazzan tank, the oil in the tank and the surrounding see water – axonometric view*

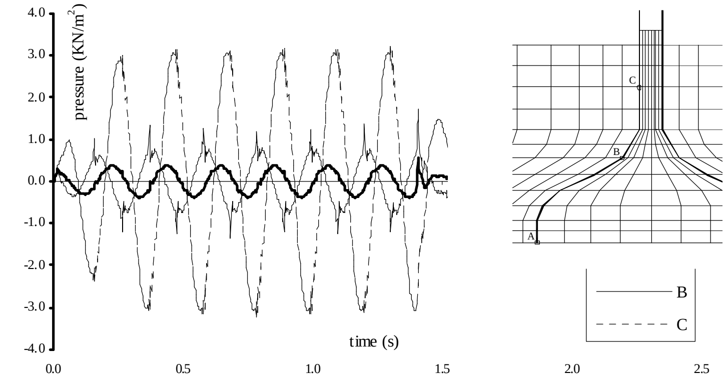


Fig. 33. *Hydrodynamic water pressure in the specified points on the surface of the Kazzan oil-storage tank for the horizontal seismic action*

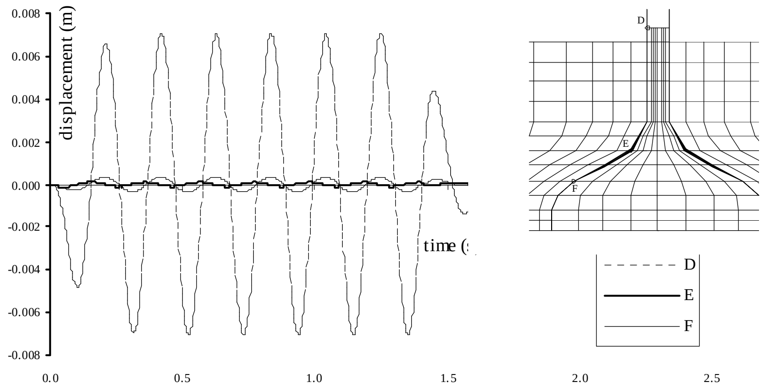
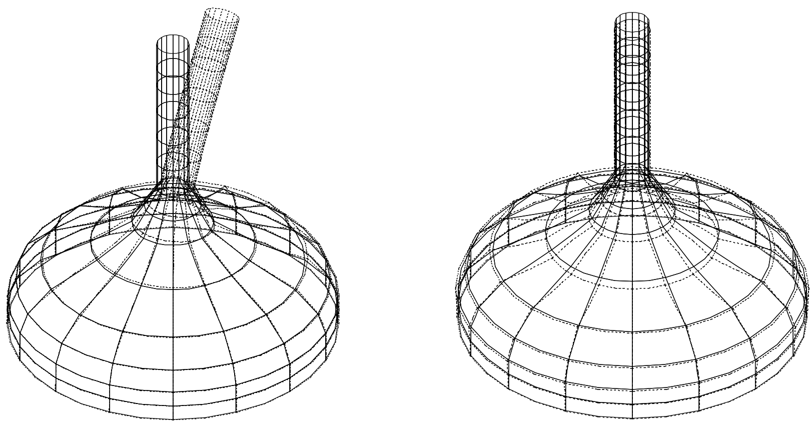
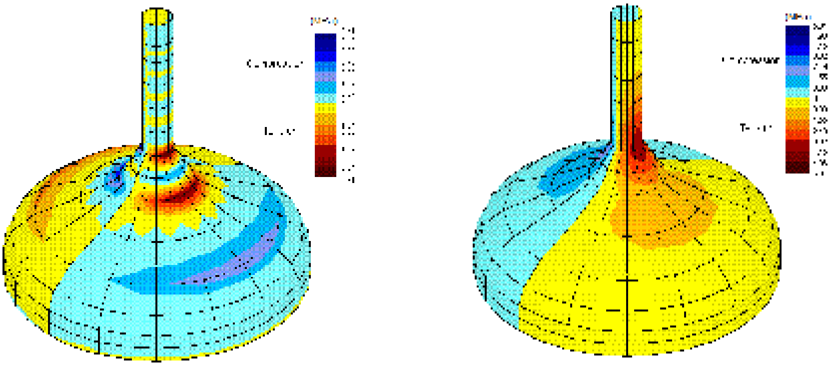


Fig. 34. *Horizontal displacements of the specified points of the Kazzan oil-storage tank for the horizontal seismic action*



(i) horizontal seismic action (ii) vertical seismic action

Fig. 35. *The Kazzan oil-storage tank maximal displacements*



(i) horizontal (σ_{xx}) stresses (ii) vertical (σ_{yy}) stresses

Fig. 36. *Maximum stresses of the Kazzan oil-storage tank in t = 0.73 s, for the horizontal seismic action*

3.4. Example 4 – Underwater tunnel "Høgsfjord"

The seismic behavior of the planned (but not yet realized) underwater tunnel "Høgsfjord" in Norway was analyzed. The tunnel is about 1400 m long and 20 m immersed in the sea. It's connected to the sea-bed with cables every 200 m (axial distance) (Fig. 37).

The tunnel has a circular cross-section with a 8.6 m inner diameter and 50-80 cm thick walls (Fig. 37). Intended construction material for the tunnel is prestressed concrete. Some other information about the planned structure can be found in [32, 36]. The material characteristics are shown in Table 4.

(i) the position of the tunnel (ii) cross-section of the tunnel (iii) visualization of the tunnel

Fig. 37. *Underwater tunnel "Høgsfjord", Photo: Statens Vegvesen Rogaland [36]*

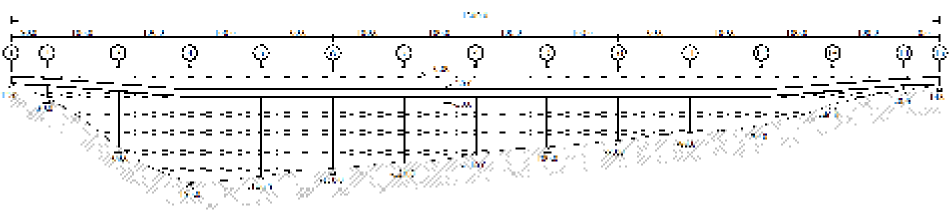


Fig. 38. *Longitudinal section of "Høgsfjord" tunnel [36]*

The seismic response to the vertical earthquake component was analyzed. A plane (2D) model was adopted with the discretization shown in Fig. 28. Some results are shown in Figs. 29-31, and a detailed description of the model and results can be found in [32]. Displacements and stresses in the tunnel from applied vertical excitations are relatively small, and the tunnel has significant seismic resistance. On these types of structures, wave and sea current actions have more influence.

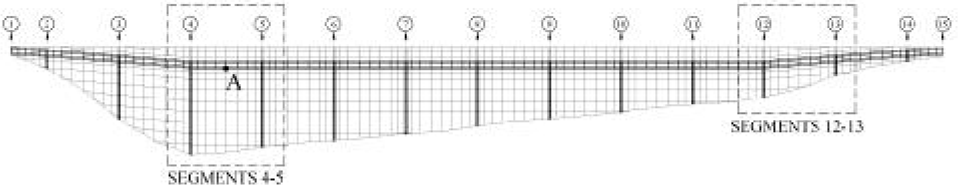
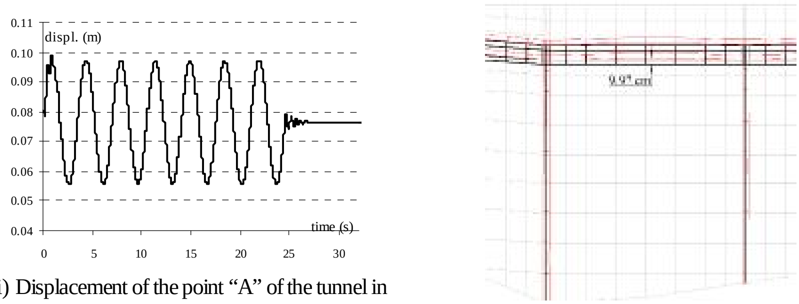


***Fig. 39.** Spatial discretization of "Høgsfjord" tunnel*

CONCRETE	Modulus of elasticity	Poisson's coefficient	Density	Compression strength		
	E_C (GN/m^2)	ν_c	ρ_c (KN/m^3)	F_{ck} (MN/m^2)		
	36.0	0.2	25.0	40.0		
STEEL	- REINFORCING STEEL -					
	Modulus of elasticity	Poisson's coefficient	Density	Hardening modulus	Yielding strength	Ultimate strength
	E_S (GN/m^2)	ν_s	ρ_s (KN/m^3)	H_s' (GN/m^2)	σ_{ys} (MN/m^2)	σ_{us} (MN/m^2)
	210.0	0.3	78.5	0.0	400.0	500.0
	- PRESSTRESING STEEL -					
	Modulus of elasticity	Poisson's coefficient	Density	Hardening modulus	Yielding strength	Ultimate strength
	E_{PS} (GN/m^2)	ν_{ps}	ρ_{ps} (KN/m^3)	H_{ps}' (GN/m^2)	σ_{yps} (MN/m^2)	σ_{ups} (MN/m^2)
	210.0	0.3	78.5	0.0	1570.0	1770.0
STEEL FOR TENDONS	Modulus of elasticity	Poisson's coefficient	Density	Hardening modulus	Yielding strength	Ultimate strength
	E_B (GN/m^2)	ν_b	ρ_a (KN/m^3)	H_s' (GN/m^2)	σ_y (MN/m^2)	σ_u (MN/m^2)
	210.0	0.3	78.5	0.0	1570.0	1770.0
SEA WATER	Velocity of sound			Density of sea water		
	c (m/s)			ρ_w (KN/m^3)		
	1500.0			10.5		

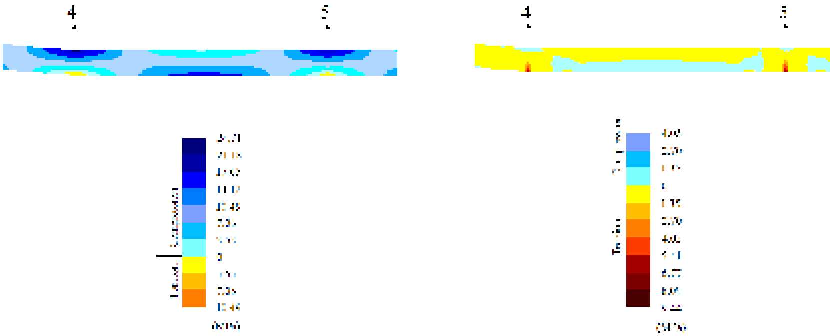
***Table 4.** Material characteristics of "Høgsfjord" tunnel*



(i) Displacement of the point “A” of the tunnel in time

(ii) The maximal tunnel deflections

Fig. 40. *Displacements of “Høgsfjord” tunnel, segment 4-5*



(i) Maximal horizontal stresses (σ_{xx})

(ii) Maximal vertical stresses (σ_{yy})

Fig. 41. *Stresses of “Høgsfjord” tunnel, segment 4-5*

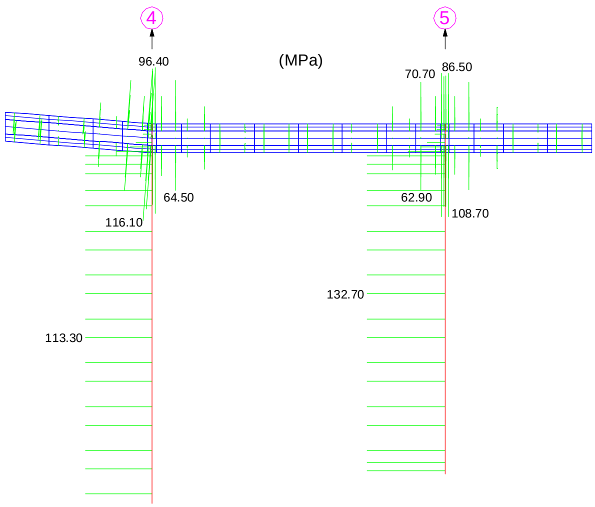


Fig. 42. *Stresses in reinforcement and cables, “Høgsfjord” tunnel, segment 4-5*

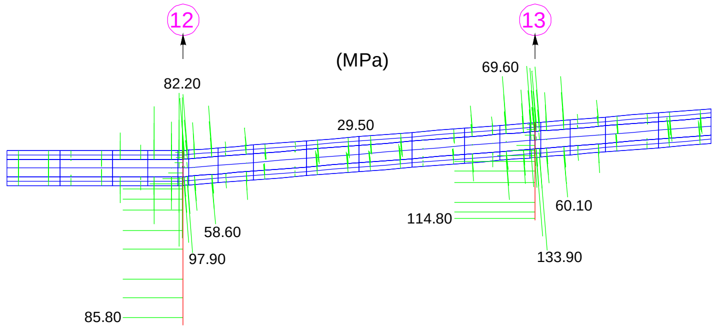


Fig. 43. *Stresses in reinforcement and cables, "Høgsfjord" tunnel, segment 12-13*

4. CONCLUSION

The presented models for the dynamic (seismic) analysis of various types of structures that are in contact with fluid can simulate some of the most important non-linear effects. The models are simple, reliable and can be used in a wide range of practical problems. Shown examples illustrate some of the possibilities of the models and the developed computer programs (software) for various types of engineering structures.

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Miha Tomaževič[1]

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MASONRY STRUCTURES AND EXPERIMENTAL RESEARCH FOR EUROCODES

Summary: *The paper summarizes the results of recent experimental studies carried out at Slovenian National Building and Civil Engineering Institute and aimed at providing information for the evaluation of values of design parameters introduced by Eurocodes. On the basis of the results of shaking table tests and taking into consideration damage limitation and displacement capacity of typical masonry buildings, the range of possible values of structural behavior factor, has been assessed. As regards the existing buildings, it has been shown that the simultaneous use of confidence and partial material safety factors in seismic resistance verification procedure is too conservative. Different types of units and a series of masonry walls have been tested to propose a measure for sufficient robustness of hollow clay masonry units. It has been shown that robustness of units mainly depends on the working stress/compressive strength ratio of masonry walls.*

Keywords: *Masonry, shaking table tests, structural behavior factor, existing buildings, confidence factor, masonry units, robustness, masonry walls, cyclic shear tests, shear resistance*

ZIDANE KONSTRUKCIJE I EKSPERIMENTALNA ISTRAŽIVANJA ZA EUROKODOVE

Rezime: *U radu su sažeti rezultati eksperimentalnih istraživanja koja su u poslednjih nekoliko godina rađena na Zavodu za gradbeništvo Slovenije sa ciljem dobivanja podataka za ocenu vrednosti nekih parametara za projektovanje zidanih konstrukcija, koje uvode Eurokodovi. Na osnovu rezultata ispitivanja na seizmičkoj platformi te uzimanjem u obzir ograničenje oštećenja i kapacitet pomeranja tipičnih zidanih zgrada urađena je ocena mogućih vrednosti faktora ponašanja konstrukcije. U vezi analize seizmičke otpornosti postojećih zgrada pokazalo se, da su rezultati dobijeni sa istovremenom upotrebom faktora pouzdanja i delimičnog faktora sigurnosti materiala suviše konzervativni. Da bi se dobilo merilo za zadovoljavajuću robustnost zidnih šupljih blokova, ispitani su različiti tipovi blokova i zidova. Ispitivanja su pokazala, da robustnost blokova u najvećoj meri zavisi od nivoa tlačnih napona u zidovima.*

Ključne reči: *Zidarija, ispitivanja na seizmičkoj platformi, faktor ponašanja konstrukcije, postojeće zgrade, faktor pouzdanja, blokovi za zidanje, robustnost, zidovi, ciklična ispitivanja na smicanje, otpornost na smicanje*

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[1] Prof. dr., Slovenian National Building and Civil Engineering Institute, Dimičeva 12, 1000 Ljubljana, Slovenia e-mail: miha.tomazevic@zag.si

1. INTRODUCTION

Eurocodes [1], [2], [3], European standards for structural design, provide principles and application rules for earthquake resistant design of new masonry buildings as well as specifications to be considered in the case of structural assessment and redesign of existing ones, including masonry buildings of historic importance. Being included into the family of Eurocodes, the design of masonry structures is following the same contemporary design philosophy as the design of any other type of structures. Taking into consideration specific properties of masonry materials, where the probability of not achieving the required mechanical properties is higher than in the case of other structural materials, it is to understand that partial safety factors are higher and load reduction factors are lower than in the case of other structural types.

To make masonry construction competitive, materials of improved strength and thermal insulation properties have been developed and technologies to simplify and speed-up the construction process have been proposed. As preliminary experimental studies indicated, being developed mainly for the intended use in the non-seismic countries, some of these improvements adversely affect the resistance and displacement capacity of masonry structures in seismic situation. Therefore, relevant specifications to limit the use of such materials and construction technologies have been introduced in the standard which covers earthquake resistant design [2]. However, because of the lack of experimental data, the requirements are mainly qualitative, or ranges of values to be used in the design are recommended. It is expected that National Annexes issued by European Union's members states will provide quantitative limitations and narrow the ranges.

To make contribution and provide part of the missing information needed to adequately amend and complement the requirements of the code, experimental research has been conducted also at Slovenian National Building and Civil Engineering Institute (ZAG) in Ljubljana, Slovenia. Some results of this research will be discussed in the following.

2. SEISMIC LOAD REDUCTION: BEHAVIOR FACTOR q

Most masonry structures belong to the category of structures with regular structural configuration, in the case of which the seismic resistance can be verified by simple equivalent elastic static analysis. Design seismic loads are evaluated on the basis of the response spectra, considering the structure as an equivalent single-degree-of-freedom system, and reducing the ordinates of the elastic spectrum by a factor which takes into account the displacement and energy dissipation capacity of the structure under consideration, so called "force (strength) reduction factor". Because of many parameters which influence the values (see for example [4]), the evaluation of force reduction factors for earthquake resistant design of structures is a relatively complex process. However, taking advantage of a number of experimental data regarding the seismic response of different types of masonry buildings, the assessment of possible values has been simplified. Without any additional parametric studies, experimental results have been evaluated to verify the Eurocode 8-1 [2] proposed values. The general definition given in Eurocode 8-1 has been followed.

According to Eurocode 8-1, force (strength) reduction factor is called “behavior factor q”, defined as “an approximation of the ratio of the seismic forces that the structure would experience if its response was completely elastic with 5 % viscous damping to the minimum seismic forces that may be used in the design - with a conventional elastic analysis model - still ensuring a satisfactory response of the structure”. In other words, the behavior factor is the ratio between the seismic force which would develop in an ideal elastic structure (S_e) and the design seismic load (design base shear, S_d):

$$q = S_e/S_d, \tag{1}$$

In the case of seismic resistance verification, each structural element and the structure as a whole should be verified for:

$$E_d \leq R_d, \tag{2}$$

where E_d = design value of action effects, i.e. design load in seismic situation, acting on the element and distributed on the element according to the theory of elasticity, and R_d = design resistance of structural element under consideration. Since the elastic analysis methods do not take into consideration the redistribution of seismic loads after yielding of individual structural elements, and the characteristic material strength values are reduced by partial safety factors, γ_M, the design resistance of structure, R_d, is only an approximation, usually much smaller than the actual maximum resistance, R_{max} (or $R_{max,id}$, obtained by idealizing the actual resistance curve with bi-linear relationship, Figure 1). The ratio between the actual maximum resistance, R_{max} (idealized value $R_{max,id}$), and the design resistance of the structure, R_d, is called reserve strength (overstrength), ρ:

$$\rho = R_{max}/R_d \text{ (or } \rho = R_{max,\, id}/R_{,d}). \tag{3}$$

Assuming that design resistance R_d is equal to design seismic load, S_d, and substituting design seismic load S_d in Eq. (1) by expression for design resistance resulting from Eq. (3), behavior factor q can be expressed in terms of actual maximum resistance R_{max} ($R_{max,\, id}$) and overstrength factor ρ as follows:

$$q = \rho S_e/R_{max} \text{ (or } q = \rho S_e/R_{max,\, id}). \tag{1a}$$

In other words, the behavior factor can be expressed as a product of two parameters, namely factor S_e/R_{max} (or $S_e/R_{max,\, id}$), which is ductility dependent factor [5], and overstrength factor ρ. According to definition given in Eurocode 8-1, the overstrength is implicitly taken into account in the values of structural behavior factor q, required for seismic resistance verification of various structural systems. However, no indication whatsoever is given in the code as regards the amount of the expected overstrength, considered when assessing the values of behavior factor proposed for different masonry construction systems. It has been already shown that, if the resistance of a masonry structure is calculated by means of traditional methods of elastic static analysis, significant overstrength can be expected, depending on the structural type and configuration, as well as the method of calculation [6].

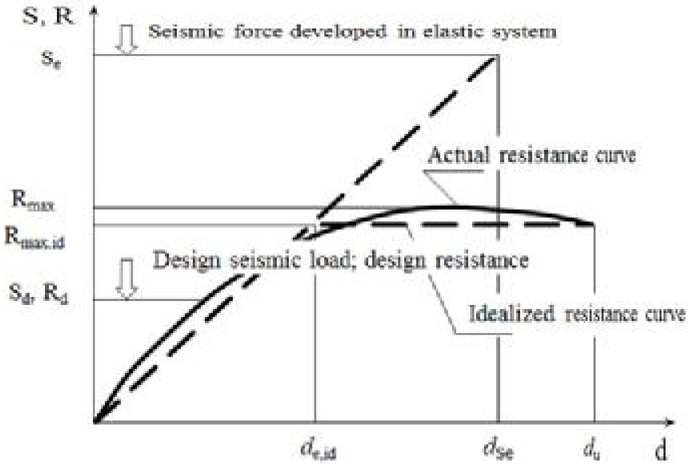


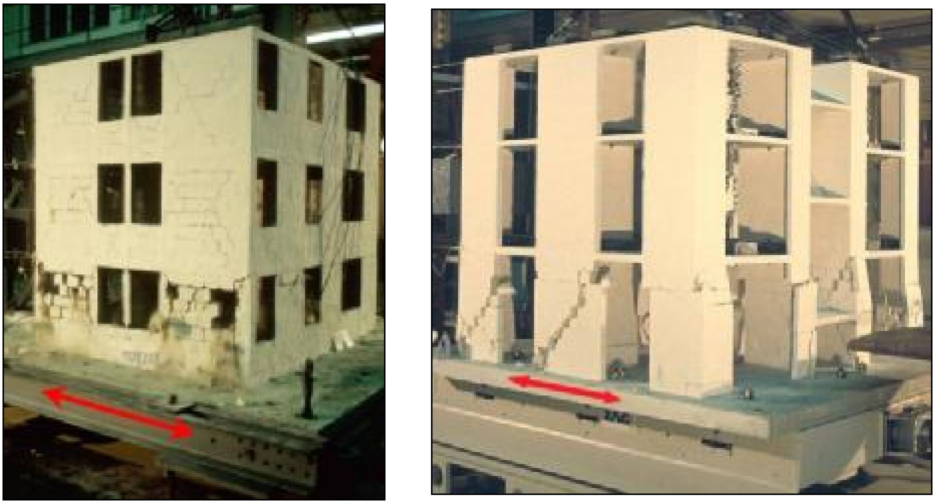
Figure 1. Comparison of ideal elastic and actual (idealized actual) non-linear behavior of a structure

The expression for behavior factor, given in Eq. (1), is based on the assumption that the maximum displacement response amplitudes of an ideal elastic and equivalent non-elastic rigid structures, subjected to the same ground motion, are equal. Using the same basic definition given in Eurocode, but the assumption of equality of energies (equality of areas below the elastic triangle and actual resistance envelope, Fig. 1), the value of behavior factor can be estimated also on the basis of the actual available ductility. For the assessment, the actual resistance curve is idealized as bilinear ideal elastic-ideal plastic relationship. If the assumption of equality of energies is taken into account, structural behavior factor q can be expressed in terms of the global ductility factor of the structure as follows:

$$q_\mu = (2\,\mu_u - 1)^{1/2}, \tag{4}$$

where $\mu_u = d_u/d_{e,id}$, $d_{e,id}$ = the displacement of the structure at the idealized elastic limit and d_u = the displacement at ultimate limit. In other words, Eq. 4 determines the minimum global ductility capacity (ductility demand), which should be ensured if a chosen value of behavior factor q_μ is used for seismic resistance verification. It has been shown [7] that the expression is conservative in the ductility range between 1.0 and 10.0, which is the case of all masonry construction systems. The expression does not depend on the vibration period of the structure, however, it has been proposed that a median adjustment factor 1.2 (varying between 1.05 and 1.34) be used for shear buildings. No adjustment has been considered in this study.

To estimate the possible ranges of values of behavior factor q, the results of a number of shaking table tests of models of confined and unreinforced masonry buildings of different configuration (Figure 2), carried out in the past at ZAG (see for example [8], [9], [10]), have been evaluated.



a.) *b.)*

Figure 2. Confined (a) [8] and (b) plain masonry building models at ultimate state before collapse [9]

The measured base shear-first story drift relationships (resistance curves) of each model, idealized as bilinear ideal elastic-ideal plastic relationships, and Eq. 4 have been used to evaluate the values (Figure 3). To generalize the data, base shear, BS, was expressed nondimensionally in terms of the base shear coefficient, BSC = BS/W, where W = the weight of the structure above the base, and interstory drift in terms of rotation angle, Φ = d/h. In the idealization of the experimentally obtained resistance envelope, story drift at the point where the resistance of the structure degrades to 80 % of the maximum, is usually defined as the ultimate [2]. It is assumed that a ductile structure, although severely damaged, will resist such a displacement without risking collapse (no

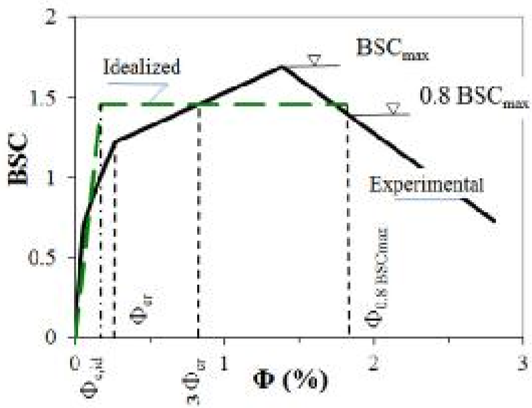


Figure 3. Evaluation of structural behavior factor on the basis of experimentally obtained resistance curve, after [9]

collapse requirement). However, one of the previous studies [11] indicated that the acceptable level of damage to walls (damage Grade 3 according to EMS-98 seismic intensity scale [12]) occurs at interstory drift equal to approximately $\Phi = 3\Phi_{cr}$, where Φ_{cr} = interstory drift angle at the damage limit state. Consequently, besides the usual no collapse requirement, expressed by $\Phi_{d,u} = \Phi 0.8BSC_{max}$, damage limitation requirement, expressed by $\Phi_{d,u} = 3\Phi_{cr}$, is also considered in the evaluation of q factor. The lesser value of the two is taken into account in the evaluation.

System	No. of stories	Materials	$\Phi_{e,id}$ (in %)	Φ_{cr} (in %)	$\Phi_{0.8BSCmax}$ (in %)	$3\Phi_{cr}$ (in %)	$\mu_u = 3\Phi_{cr} / \Phi_{ei,d}$	
							μ_u	q
Confined	3	Clay block	0.17	0.28	2.60	0.84	4.94	2.98
	3		0.17	0.27	1.81	0.81	4.76	2.92
	3	AAC block	0.23	0.28	2.46	0.84	3.65	2.51
	3		0.36	0.48	2.27	1.44	4.00	2.65
	4		0.30	0.44	2.33	1.32	4.40	2.79
	3	Clay block	0.14	0.42	1.36	1.26	9.00	4.12
	3		0.23	0.55	3.16	1.65	7.17	3.65
Plain	3	Calcium silicate	0.07	0.20	0.42	0.16	8.57	4.02
	3	Clay block	0.16	0.33	1.65	0.99	6.18	3.37

Table 1 Values of structural behavior factor q, evaluated from the results of model shaking table tests on the basis of ductility and damage limitation requirements (adopted from [8], [9], [10])

Typical results of such evaluation are summarized in Table 1. The experimental studies indicated that, in addition to construction system (confined, plain masonry), as assumed by the code, seismic resistance and displacement capacity of masonry buildings depend also on the type of masonry materials and structural configuration. Taking this into account, it can be seen that the values of behavior factor q cannot be assessed by means of only ductility tests of individual structural walls and subsequent numerical analysis of seismic response of the whole structure. Numerical simulation based on the input data obtained by testing of individual walls is usually not enough.

It can be seen, however, that in the case where the values of behavior factor are evaluated by taking into consideration damage limitation requirement ($\Phi_{d,u} = 3\Phi_{cr}$), the differences in the values of factor q, evaluated for different structural types, are not significant. It can be also seen that the values, evaluated in such a way, are well within the range of values, proposed by the code for seismic resistance verification of different masonry construction systems [2]:

- For unreinforced masonry: q = 1.5–2.5,
- For confined masonry: q = 2.0–3.0,
- For reinforced masonry: q = 2.5–3.0.

As shown by the analysis of experimental results, the values at the upper limit of the Eurocode 8-1 proposed range of values of structural behavior factor q for unreinforced and confined masonry construction systems, i.e. $q = 2.5$ for unreinforced, and $q = 3.0$ for confined masonry structures, are adequate even in the case where very little or no overstrength, i.e. $\rho \cong 1.0$, is expected. It can be therefore concluded that in the case where elastic analysis methods are used and significant overstrength is expected, these, code proposed values are conservative. Although some studies have been already made to modify code requirements [6], additional research and parametric studies are needed to confirm the proposals.

Pushover methods for calculation of seismic resistance of masonry buildings have been proposed before long ([13] and [14]). As the correlation between experimentally obtained and calculated results indicates, lateral resistance-displacement characteristics of masonry structures are realistically predicted by these methods. If these methods, where no overstrength (ovestrength factor in Eq. (1a) is $\rho \cong 1.0$) is taken into account, are used for seismic resistance verification, the values of behavior factor at the upper limit of the Eurocode 8-1 proposed range can be used to determine the design seismic load (resistance demand) needed in seismic resistance verification. However, displacement capacity of the structure should be also verified and compared with displacement (ductility) demand, at the same time.

If, on the other hand, the seismic resistance verification of the same structure is carried out by using traditional elastic methods, the code specified values of behavior factor q can be increased by the overstrength factor, implicitly taken into account by the code makers when suggesting the values given in Eurocode. In other words, if instead of a model predicting the actual resistance curve (mechanism model, nonlinear pushover method), a conventional elastic analysis model is used in the calculation, the values of behavior factor q, recommended in Eurocode, can be increased by the actual overstrength factor $\rho = R_{max}/R_d$ ($R_{max,id}/R_d$).

3. MATERIAL LIMITATIONS

3.1. Masonry materials and redesign of old buildings

As recommended by Eurocode 8-3 [3], mean, and not characteristic values of mechanical properties of materials are considered in the redesign of existing buildings, determined either by in-situ testing or by testing specimens, taken from the existing structure, in the laboratory. To obtain the design values, the mean values are reduced with the so called confidence factor, CF, the value of which depends on the thoroughness of inspection of the building and reliability of data needed for structural evaluation. In addition, however, the standard requires that partial safety factors for material, γ_M, be also taken into account to calculate the design values of material strength:

$$f_d = \frac{f}{CF\gamma_M}, \tag{5}$$

where f_d = the design value of material strength, f = mean value of material strength, determined by testing, CF = confidence factor, and γ_M = partial material safety factor for masonry.

Confidence factor is a function of knowledge level (KL), which, according to Eurocode 8-3 [3] depends on the thoroughness of inspection of the buildings under consideration and the number of tests which have been carried out to assess the state of the structure and material properties. Three knowledge levels are defined in the code. No reduction, i.e. CF = 1.00, is needed in the case of the complete structural knowledge (80 % of structural elements inspected, 3 material specimens tested in each story), CF = 1.20 is recommended for the case of the intermediate, and CF = 1.35 for the case of the limited knowledge (20 % of elements inspected, 1 material specimen tested in each story).

According to Eurocode 6-1 [1], the values of partial safety factor for masonry, γ_M, depend on the factory production control and inspection of works on the site. In normal situation, the values within the range between 1.5 (optimum production control and severe inspection on the site) and 3.0 (no proof regarding the production control and inspection) are considered. In seismic situation, the chosen value can be reduced by 1/3, however in no case γ_M should be smaller than 1.5.

Whereas the introduction of confidence factor makes sense and stimulates the amount of inspection and testing of structural materials for the assessment of seismic resistance of old masonry buildings, the reduction by partial material safety factor, as defined in Eurocode 6-1 for the new construction, cannot be accepted. Speaking of structural safety, it is not possible to assess the uncertainties regarding the mechanical properties of old, historic masonry, as is the case of the new construction. At the time of their construction, the modern quality control mechanisms have not yet been established. Consequently, the most unfavorable value should be considered in redesign of such buildings, namely $\gamma_M = 3.0$, which in seismic situation can be reduced to $\gamma_M = 2.0$. This means that only one half of the mean value of masonry strength, obtained by testing the actual materials, can be considered in the redesign - at the best.

As an example of consequence of reduction of experimentally determined values of masonry strength by partial safety factor, γ_M, on the decision regarding the necessary strengthening measures, the seismic resistance of a series of stone masonry buildings in the region of Posočje, Slovenia, has been analyzed (Table 2). The buildings, damaged by the earthquake in 1998 (estimated intensity VIII by European Macroseismic Scale, EMS [12]), were strengthened by tying the walls at floor levels and injecting the walls with cementitious grout. Strengthened after the earthquake of 1998, they were subjected to another earthquake of the same intensity in 2004. Most buildings remained undamaged in 2004, some of them suffered only minor damage.

In the analysis, shear mechanism model and experimentally obtained values of the tensile strength of the strengthened, cement grouted masonry, f_t, have been taken into account [15]. Instead of mean, characteristic values, $f_{t,k}$, of the tensile strength have been taken into account. For easier comparison with the code requested value of the design base shear coefficient, BSC_d, the resistance of analyzed buildings is expressed in a nondimensional form of the design seismic resistance coefficient, $RC_d = R_d/W$, where W = the weight of the building.

According to seismic hazard map of Slovenia, design acceleration $a_g = 0.225$ g should be considered in the case of the seismic resistance verification of building structures in the area, built on the firm soil (soil factor S = 1.0). The equivalent value of the design base shear, expressed in the nondimensional form of the design base shear coefficient, is $BSC_d = Sa_g\ \beta/q = 1.0*0.225*2.5/1.5 = 0.375$, where $\beta = 2.5$ = spectral amplification factor for the flat part of elastic response spectra, where typical vibration periods of masonry buildings are located. In the particular case studied, actual ground acceleration records, obtained in 2004 on river deposit, indicated the possibility of even higher seismic loads (Figure 4). However, nonlinear dynamic response analysis of several

Building type*	Wall/floor area (%)		$f_{t,d} = f_{t,k}$			$f_{t,d} = f_{t,k}/\gamma_M$		
	x-dir.	y-dir.	$f_{t,d}$ (MPa)	RC_{dx}	RC_{dy}	$f_{t,d}$ (MPa)	RC_{dx}	RC_{dy}
a	10.9	6.4	0.16	0.30	0.24	0.08	0.20	0.15
a	12.0	9.1	0.16	0.28	0.28	0.08	0.21	0.19
b	6.9	8.6	0.11	0.25	0.33	0.06	0.22	0.25
b	12.1	11.1	0.11	0.42	0.38	0.06	0.33	0.31
b	4.7	14.6	0.11	0.19	0.47	0.06	0.17	0.33
b	7.2	14.3	0.11	0.21	0.47	0.06	0.16	0.31
b	15.1	13.7	0.11	0.40	0.33	0.06	0.29	0.25
b	10.5	9.5	0.11	0.39	0.29	0.06	0.31	0.25
b	10.5	9.9	0.11	0.31	0.34	0.06	0.23	0.26
b	10.3	10.2	0.11	0.28	0.35	0.06	0.22	0.26
b	11.9	10.3	0.11	0.29	0.34	0.06	0.28	0.29
b	9.8	10.9	0.11	0.32	0.34	0.06	0.23	0.26
b	8.8	8.33	0.11	0.31	0.33	0.06	0.23	0.27
b	10.6	12.0	0.11	0.35	0.36	0.06	0.28	0.28
b	9.7	12.0	0.11	0.34	0.47	0.06	0.27	0.34
b	7.9	4.2	0.11	0.35	0.21	0.06	0.26	0.19

*Note: building type a: school, building type b: residential. $BSC_d = 0.375$.

Table 2. Seismic resistance of typical 2-story stone masonry buildings in Posočje, Slovenia, calculated without and with taking into consideration partial safety factor for masonry, $\gamma_M = 2.0$. CF = 1.0 in both cases (adopted from [15])

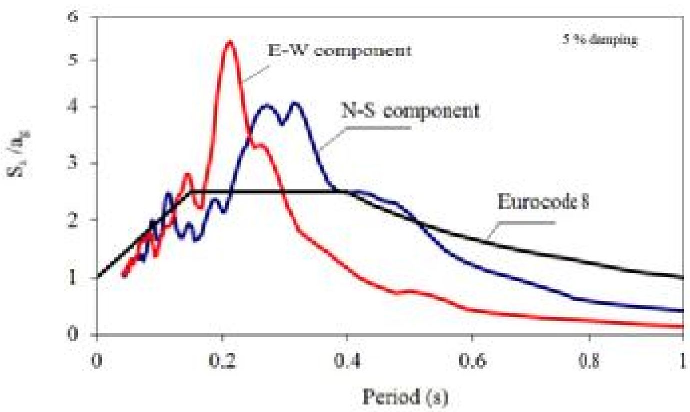


Figure 4. Eurocode 8 response spectrum in comparison with the response spectra of 2004 Posočje earthquake for 5 % damping (earthquake data obtained from IKPIR)

buildings to recorded ground motion, where the same mechanical properties of stone masonry have been taken into account as in the case of this analysis, confirmed the realistic assumption of the code as regards the design earthquake [15].

As the damage observation analysis after the earthquake indicated, the actual resistance of the analyzed buildings was sufficient to prevent the damage, although the resistance values, calculated by taking into account the characteristic tensile strength of masonry, do not fully comply with the requirements of the code. In the case, where the characteristic values of masonry strength were reduced by partial safety factor, γ_M, however, the predicted resistance of the analyzed buildings is much lower than required. The results of such analysis indicate that serious strengthening measures should be applied to the analyzed buildings in order to ensure adequate seismic behavior and prevent damage. However, as the actual situation proved, the additional measures have not been necessary.

On the basis of this and similar past experiences, it can be seen that, in the case of redesign of old masonry buildings, there is no reason that besides the confidence factors, CF, the partial safety factors for masonry, γ_M, be also considered in determining the design values. As regards the code recommended values of confidence factors, CF, belonging to each knowledge level, past experiences show that the code recommended values are too optimistic. The value of CF = 1.0 is acceptable, if mechanical properties of masonry are determined either by in-situ tests or in the laboratory by testing specimens, taken from the building under consideration. In such a case, at least one specimen of the specific masonry type should be tested in the building and the composition of the masonry should be verified by removing plaster at least in one location in each story. CF = 1.35 should be used if the mechanical properties are obtained by testing at least one specimen in the cluster of buildings of the same typology. Identification of a given type of stone-masonry is carried out by removing plaster and opening the walls at least in one location in each story of the building under consideration. In the case that no tests but identification inspection only is carried out, the recommended value is CF = 1.7. In this case, however, the values of mechanical properties for the specific masonry type are taken from the literature, corresponding to the masonry type under consideration.

3.2. Robustness of masonry units

Nowadays, solid bricks are replaced by hollow units, the shape and materials of which have been designed to meet the demanding energy saving criteria for buildings with minimum additional thermal insulation layers. Clay units are usually made of specially developed porous clayey materials. They are shaped to have a large percentage of uniformly distributed holes, which requires thin shells and webs. Whereas the load bearing capacity of masonry walls made of such units is adequate for gravity loads, experimental investigations indicated, that the units exhibit local brittle failure if the walls are subjected to a combination of high level of compressive stresses and in-plane horizontal seismic loads at the same time [16]. As the result of the lack of robustness of masonry units, the behavior of such walls is even less ductile than the behavior of walls built with traditional bricks and mortar (Figure 5a). If masonry is reinforced with steel reinforcement, placed between the units in different ways, and the units are brittle, they are not able to carry the additional compression and shear needed to develop the tension capacity of reinforcement (Figure 5b). Consequently, the design equations, developed on the assumption of solid behavior of units and adequate bond between units, mortar, and reinforcement, do not reflect the actual situation [16]. As a rule, they are misleading, because they are too optimistic as regards both, lateral load bearing and displacement capacity of reinforced masonry walls. Lack of robustness of the units is the main reason for unreliability.

(a) (b)

Figure 5. Brittle shear failure of a highly stressed unreinforced masonry wall (a) and (b) brittle failure of a complete course of concrete hollow blocks which prevented the activation of reinforcement

To avoid local brittle failure of hollow units, requirements are given in most national seismic codes which limit the amount of holes and minimum thickness of shells and webs of the units used for the construction of masonry buildings in seismic zones. In this regard, requirements have been also specified in the draft version of Eurocode 8 [17]. The void ratio was limited to 50 % of the volume of the units, and the minimum allowable thickness of shells and webs of the units to 15 mm. Since such units no longer exist on the market, the present standard [2] requires only that "masonry units should have sufficient robustness in order to prevent local brittle failure." The decision on how to meet the requirement is left to the National Annexes, which "may select the type of masonry units from EN 1996-1: 2004, Table 3.1 that satisfy this requirement." However, the decision is not simple, because according to this table, the units to be selected from, are the units, where the volume of holes varies from 25 % to 55 % of the gross volume of the unit, and the thickness of shells and webs is not less than 8 mm and 5 mm, respectively.

As an attempt to propose such criteria, the influence of shape of the units on the parameters of seismic resistance of the wall has been investigated at ZAG. For the study, 6 different types of hollow clay blocks, currently available on the market (Figure 6), have been selected. Since all brick producers aim at the same goals, the materials, shape and dimensions of units do not vary substantially. The experimental program consisted of two phases. In the first phase of testing, the mechanical and geometrical characteristics of all types of units have been determined by standardized testing procedures. Then, a series of specific tests has been carried out, by means of which the stress state and failure mechanism of a single unit have been simulated for the case in which the units are part of a shear wall, subjected to a combination of vertical load and shear in the case of an earthquake. In the second phase, a series of 28 walls, made with all six types of masonry units, have been tested by subjecting them to a combination of constant vertical load and cyclic shear. Two levels of precompression have been chosen to simulate the possible ranges of working stress levels in masonry walls due to vertical loads in actual structures.

Figure 6. Typical hollow clay blocks tested in the experimental campaign [18]

Since the quality of masonry blocks is declared by their compressive strength, the correlation between the mean compressive strength of units, normal to the bed joints, $f_{b,m}$, on the one hand, and compressive strength, parallel to the bed joints ($f_{b,h}$), as well as the diagonal tensile ($f_{bt,d}$), splitting tensile ($f_{bt,s}$) and shear strength (f_{bs}), on the other, has been analyzed. The results are shown in Figures 7a and b. In Figure 7a, the ratio between the various strength parameters and compressive strength of tested hollow blocks is plotted against the volume of holes. As can be seen, only the ratio between the compressive strength, parallel to the bed joints and compressive strength of units, indicated a trend of increase with the decreased volume of holes.

In Figure 7b , however, the dependence of the analyzed strength parameters, normalized by the compressive strength of units, on the combined thickness of shells and webs is shown, which confirms the expectations that the resistance of units to tension and shear depends not only on the quality of materials, but also on the units' shape and amount of holes.

However, the cyclic shear tests of the walls did not confirm this conclusion. All walls failed in shear, as expected, but no difference in resistance and displacement capacity, which could have been attributed to different shapes of the units, has been observed. The resistance envelopes of all walls, tested at the same precompression ratio, were similar in terms of both, resistance and displacements. The coefficient of variation of resistance values at individual displacement amplitudes was 6.5 % for high and 8.4 %

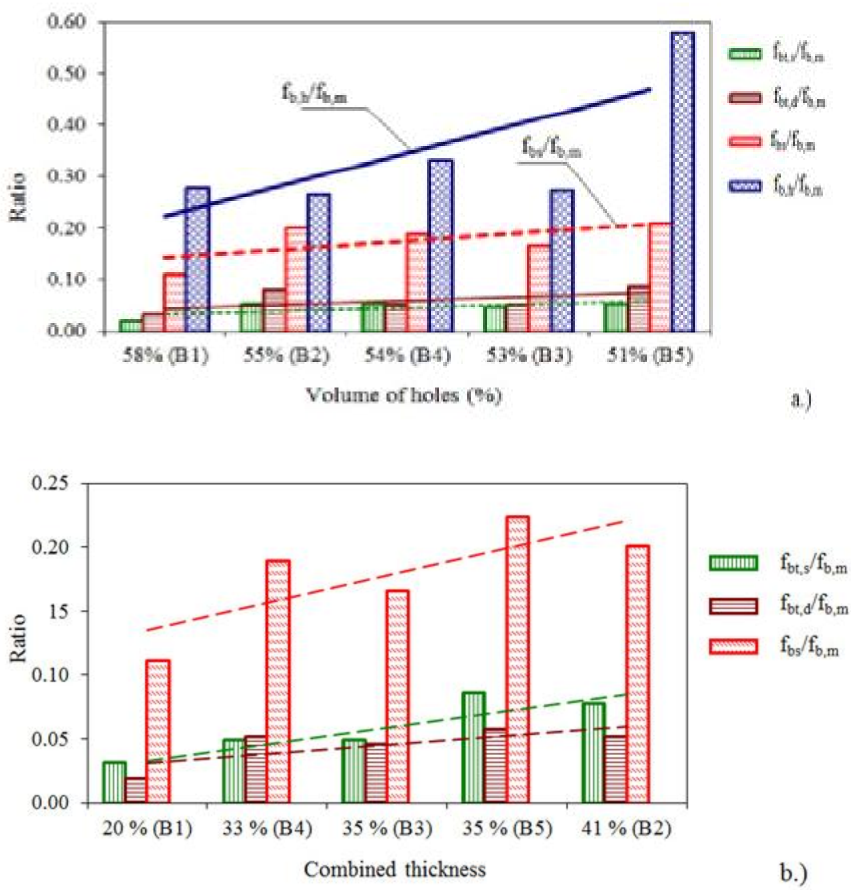


Figure 7. Correlation between strength parameters and the volume of holes, a) and b) correlation between strength parameters and combined thickness of shells and webs [18]

for low level of precompression. As can be seen in Figure 8, where the resistance envelopes, averaged for each precompression level, are presented in a nondimensional form, the precompression ratio determined the displacement capacity of the walls. In the figure, the resistance of the walls has been normalized with regard to the maximum and displacements were expressed in terms of drift angle, $\Phi = d/h$ (in %).

Against expectations, the shape and mechanical properties of individual units did not affect the seismic behavior of walls. When subjected to a combination of vertical and cyclic horizontal loads, the working compressive stress/compressive strength of masonry ratio, turned out to be the governing parameter. The behavior of units, which exhibited monolithic behavior at low level of precompression, became brittle when subjected to a combination of higher level of precompression and cyclic in-plane shear loads (Figure 9).

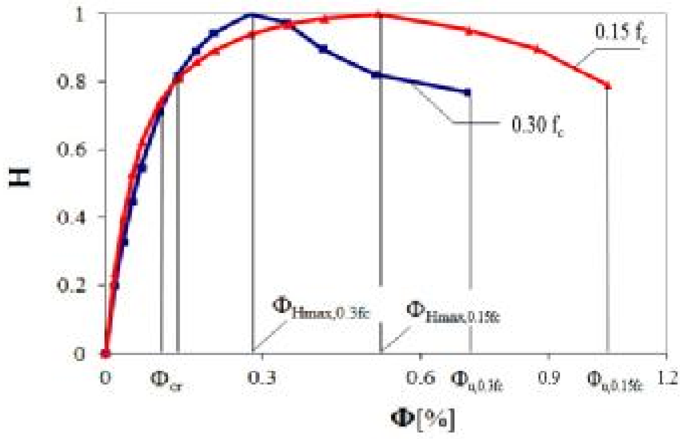


Figure 8. Comparison of average lateral load - rotation angle envelope curves, obtained by testing the walls at high and low precompression ratio. Lateral resistance is normalized with regard to maximum (after [18])

(a)

(b)

Figure 9. Damage to the wall at ultimate state at low (a) and (b) high precompression ratio [18]

In other words, the study indicated that the working stress level in structural walls is the governing parameter. Same units will behave adequately at low, but will exhibit brittle behavior at high level of precompression. Taking into account the same damage limitation criterion as in the case of assessment of behavior factor q, it can be proposed that hollow clay units with sectional properties (volume of holes, thickness of shells and webs) at the upper limit of range of values, specified for Group 2 units in Eurocode 6 [1], can be used in seismic zones, if the precompression ratio does not exceed the range of

0.15–0.25. In the case of quasi monolithic units of Group 1, however, the recommended highest precompression ratio can be increased to 0.20–0.25.

4. CONCLUSIONS

On the basis of recent experimental research in seismic behavior of masonry walls and models of buildings, an attempt has been made to quantify the design parameters, for which, due to the lack of experimental data, qualitative requirements are given in Eurocode 8 or too conservative values of design parameters are recommended. Taking into consideration the damage limitation criteria and actual displacement capacity of masonry walls and structures as a basis for the evaluation, the study indicated that the following modifications can be suggested:

The values at the upper limit of the range of values of structural behavior factor q, recommended in Eurocode 8-1 for unreinforced and confined masonry buildings, can be used if equivalent static analysis is used for seismic resistance verification. They are slightly conservative in the case where traditional elastic methods are used to assess the design resistance, but adequate in the case where mechanism models and pushover methods are used. In that case, however, not only global resistance, but also global ductility capacity should be verified at the same time;

In the case of assessment of seismic resistance of existing, historic masonry buildings, there is no need that the experimentally determined properties of masonry materials be reduced by partial material safety factor for masonry, as defined in Eurocode 6-1 for the new construction. The reduction by confidence factor with suggested modification of values for different knowledge levels will provide reliable information;

In order to avoid local brittle failure of contemporary hollow clay masonry units and ensure adequate displacement capacity of structural walls, the precompression ratio should be limited to 15−20 % of the compressive strength of masonry.

ACKNOWLEDGMENT

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DRUŠTVO ZA ISPITIVANJE I ISTRAŽIVANJE MATERIJALA I KONSTRUKCIJA SRBIJE

International Conference "**Marking 60 Years of Operation of DIMK**
Belgrade, October 19, 2012.

Roko Žarnić[1], Vlatka Rajčić[2], Lidija Krstevska[3],
Ljubomir Taškov[4], Miroslav Stepinac[5], David Antolinc[6] UDK:624.011.1.072.33.042.7

DEVELOPMENT OF SEISMIC RESISTANT WOODEN FRAMES WITH LAMINATED GLASS INFILL

Summary: *The paper presents an insight in on-going research of behaviour of wooden frames infilled by glass panels. Researchers from three institutions, University of Ljubljana, University of Zagreb and IZIIS Skopje joint their efforts to develope an innovative structural element for use in contemporry buildings. Because the research is in relatively early stage only typical results showing the response of tested specimens on vertical, racking and shaking load are presented. In continuation of the research programme the computational model will be developed, verified and validated by the experimentally obtained data on describing the behaviour of tested types of structural elements.*

Key words: *wooden frame, laminated glass, testing, vertical load, racking, shaking*

RAZVOJ SEIZMIČKI OTPORNOG DRVENOG OKVIRA SA ISPUNOM IZ LAMELIRANOG STAKLA

Rezime: *U radu su predstavljena ispitivanja ponašanja drvenih okvira ispunjenih panelima iz lameliranog stakla. Istraživački rad je udruženi napor istraživača sa tri institucije: Univerziteta u Ljubljani, Univerziteta u Zagrebu i Instituta IZIIS u Skoplju. Ispitivanja su još uvijek u toku, pa je stoga u radu prikazan samo dio konačnih rezultata ispitivanja na verikalno, cikličko horizontalno i seizmičko opterećenje. U nastavku istraživanja razvit će se računski model, koji će se verificirati i ovrednotiti pomoću rezultata dobivenih laboratorijskim ispitivanjima.*

Ključne riječi: *drveni okvir, lamelirano staklo, ispitivanje, vertikalno opterećenje, cikličko opterećenje, potresno opterećenje*

[1] Prof. Dr. Roko Žarnić, University of Ljubljana, Faculty of Civil and Geodetic Eng.
[2] Prof. Dr. Vlatka Rajčić, University of Zagreb, Faculty of Civil Engineering
[3] Prof. Dr. Lidija Krstevska, UKIM IZIIS, Skopje, Macedonia
[4] Prof. Dr. Ljubomir Taškov, UKIM IZIIS, Skopje, Macedonia
[5] Miroslav Stepinac, B.Sc., research assistant, University of Zagreb, Faculty of Civil Eng,
[6] David Antolinc M.SC., research assistant, University of Ljubljana, Faculty of Civil and Geodetic Engineering

1. INTRODUCTION

Timber frames infilled with vertically load-bearing glass sheets represent an innovative structural element that is suitable as a load-bearing panel in prefabricated timber houses. It can be easily fixed to structural elements made of other kinds of structural materials (timber, concrete, steel) and connected to each other timber panels by simple steel fasteners. The series of tests of timber-framed glass panels started jointly at University of Ljubljana and University of Zagreb in order to study load-bearing capacity of laminated glass panels and racking performance of timber-framed glass panels. In addition, dynamic shaking table tests on full-scale box-type model made of two timber frames with glass infills exposed to real earthquake excitation has been carried out at the Institute of Earthquake Engineering and Engineering Seismology IZIIS, Skopje, Macedonia. It was a part of the Croatian-Macedonian bilateral research program, where the partner from Croatia was University of Zagreb.

The idea of development new structural element based on structural glass and simplified computational model to be used in future codes emerged from the cooperation of authors of this paper in wide international group of expert engaged for justifying of needs for further codes related to use of glass products in civil engineering works (Ref.1). They reported about the initial experimental work in Workshop held in JRC ELSA in Ispra in 2010 (Ref.2).

2. TEST SPECIMENS AND TEST PROCEDURES

Specimens for racking testing were designed as timber frames with glass infills. Infills were made of a pair of two 10 mm sheets of toughened glass laminated together. The length of infills was 2900 mm and height 2400 mm. Two types of timber frames were used. The first was made of cross-laminated timber and the other of glue-laminated timber. The first type of frames has corner joints fixed by a double steel bolts while the second type with single bolt and punched metal plates in each corner of timber frame. The dimensions of frame were 3220 mm in length and 2720 mm in height.

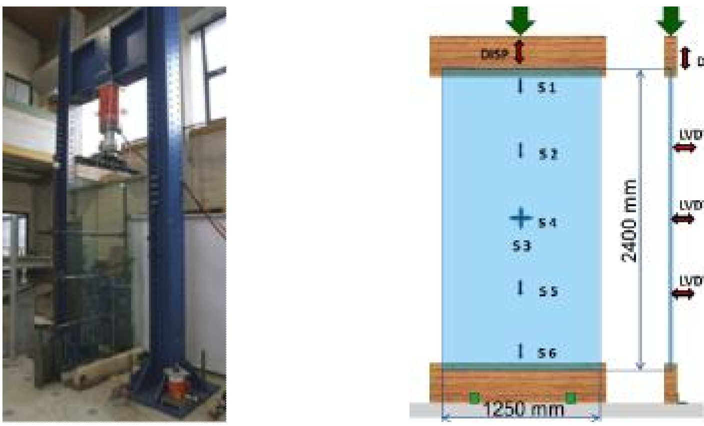


Figure 1. Vertical load test of laminated glass sheet and scheme of instrumentation

Laminated glass sheets, 1250 mm wide and 2400 high were tested to failure by vertical force (Fig. 1) to obtain data of their load-bearing capacity and deformability. Altogether three specimens were tested. Two of them were exposed to monotonous vertical load to failure and one by cyclic vertical load to failure.

Glass infilled timber frames were tested by combined constant vertical load of 25 kN/m’ and cyclic horizontal load (racking load) up to displacement equal to 2% of panel height. The test-setup that is installed in the laboratory of University of Ljubljana (Fig. 2) enables testing of panels exposed to three different configurations that simulate three different boundary conditions. Those are (Ref.3):

1. **Shear cantilever**: one of horizontal edges of panel is supported by the firm base while the other can freely translate and rotate
2. **Constrained rocking**: one of horizontal edges of panel is supported by the firm base, the other can translate and rotate as much as allowed by the ballast; ballast can translate only vertically without rotation.
3. **Shear wall**: one of horizontal edges of panel is supported by the firm base while the other can translate only in parallel with the other edge while it’s rotation is fully constrained.

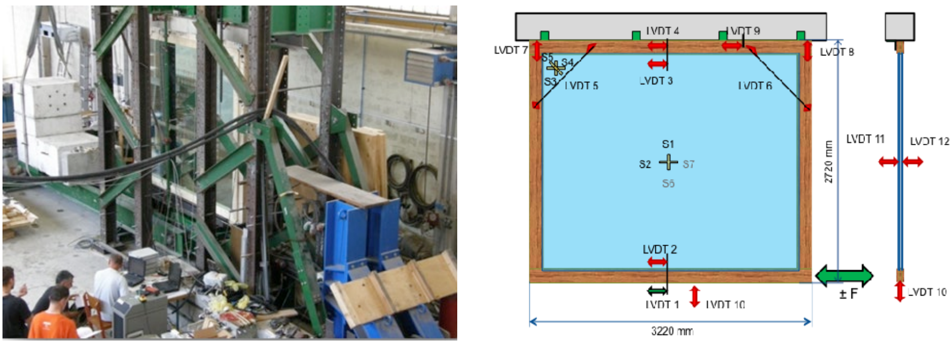


Figure 2. Cyclic racking test of glass-infilled timber frames and scheme of instrumentation

Glass-infilled X-laminated frames were tested with all three above described boundary conditions. Because it was found that the second boundary condition does not influence on significantly different response of tested element in comparison to the first boundary condition (Ref. 4), the frames of the second group are to be tested only by variation of two boundary conditions: shear cantilever (1) and shear wall (3). Until now only the shear cantilever tests were performed. In addition to glass infilled frames two glue-laminated frames without infill were also tested applying two boundary conditions: shear cantilever and shear wall. In those two frames joints were fixed by single bolt and punched metal plates.

The objective of racking test was to obtain data for development of computational model of tested type of structural element that can be used for prediction of inelastic response of buildings made of glass-infilled timber frames on seismic action. To obtain dynamic parameters and study the phenomena of response of this type of structures on seismic action, shaking table tests were carried out (Fig. 3). The table is installed at the IZIIS Laboratory in Skopje, Macedonia. It is constructed as a pre-stressed reinforced concrete slab having 5x5 m in plan. It is used for simulation of different types of dynamic motion: random, harmonic, impulse, earthquake etc. Four vertical hydraulic actuators support it. The working frequency range of the shaking table is 0.1-80Hz, and the maximum mass of a model is limited to 40t. The max accelerations are 0.7g in horizontal and 0.5g in vertical direction, and the max displacements are 0.125m in horizontal and 0.05m in vertical direction. The shaking system controls five degrees of freedom of the table, two translations and three rotations. This three-variable control system (MTS) is capable to control displacements, velocities and accelerations, simultaneously. For earthquake generation and data acquisition modular PXI system is used

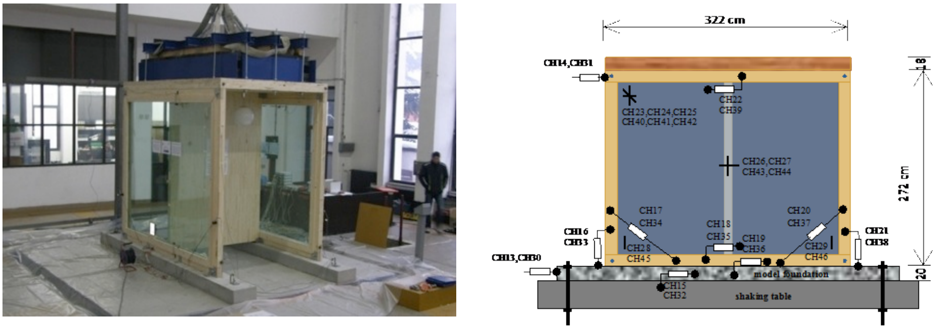


Figure 3. Shaking table test of box-type model and scheme of instrumentation

Box-type models were constructed of two glass-infilled timber frames made of simple laminated wood and corner joints fixed by single bolt and punched metal plates. The mass of 9,6 tons was added atop of model. Four types of real earthquake actions were subsequently applied to model: El Centro 1940, N-S, California, USA; Petrovac 1979 Montenegro; Kobe 1995 E-W, Japan, and Friuli 1976 E-W, recorded in Tolmezzo, Italy. The results of shaking table tests are presented in (Ref. 5).

For the seismic tests, the model was instrumented for measuring the input as well as the response at characteristic points. The both panels had the same instrumentation - 10 LVDT's and 7 SG. At the top of the model there were 4 accelerometers, one at each corner. 2 LP's were placed at the level of the foundations and 2 at the top to measure the absolute displacement of the model. Considering that the connection between the glass and the wooden frame is of crucial importance for the stability of the panel during the seismic action, as well as the location where the energy is dissipated during the strong shaking, several LVDT's for measuring the slippages and deformation were placed at the critical points. To obtain information about the strains in the glass panels 14 strain gages were used. The total number of channels was 44, as presented on Figure 3. The real time recording of the model response was performed by 72-channel high-speed data acquisition system.

3. TEST RESULTS

3.1. Laminated glass sheets

The average loadbearing capacity of two specimens of monotonously tested laminated glass sheet was 142.1 kN/m’ with mid-height horizontal displacement (buckling) of 53.7 mm. In case of cyclic vertical loading the achieved loadbearing capacity was 101.3 kN/m’ and mid-height displacement 52.8 mm.

The constant vertical load applied to timber-framed glass panels induced load to laminated sheets in magnitude of approx. 12% of their load bearing capacity in the case of cyclic vertical loading. That caused only up to 10% of buckling deformation.

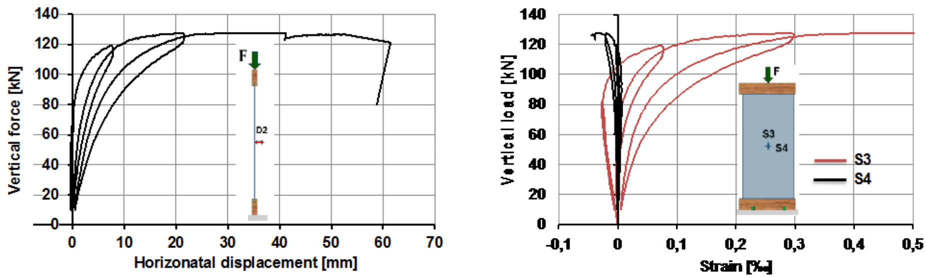


Figure 4. Response diagrams of two-layer laminated glass sheet (Spec #3) to cycled vertical load

Specimen #	F_u/1.25 (kN/m')	$D2_{Fu}$ (mm)	$S3_{Fu}$ ($^{0}/_{00}$)
1	138.14	21.42	0.29
2	146.30	13.33	0.34
3	102.14	21.04	0.29

Table 1. Main data obtained from testing of all three specimens

3.2. Monotonous load test of frames with and without glass infill

Cycling testing of each specimen preceded with monotonous push test to obtain parameters needed for creation of protocol of cyclic test. In Fig. 5 are compared response diagrams for five different specimens and test arrangement (regarding boundary conditions). The main load-bearing parameters are presented in Table 2 below.

As a first, the effect of glass infill on behaviour of frame itself (specimen F1) is obvious. Further on, there is a strong influence of type of frame composition. The glulam frames (F7, F9, F11) are less resistant than X-lam frames (F3, F5). Also the effect of punched metal plates is obvious (F9, F11). The influence of boundary conditions is well seen from the comparison of behaviour of specimen F3 and F5 and F9 an F11. In general, the observed differences were expected, but only experimental results can give the insight in their magnitude. Also it will be one of crucial set of data for development, verification and validation of complex and simplified computational model.

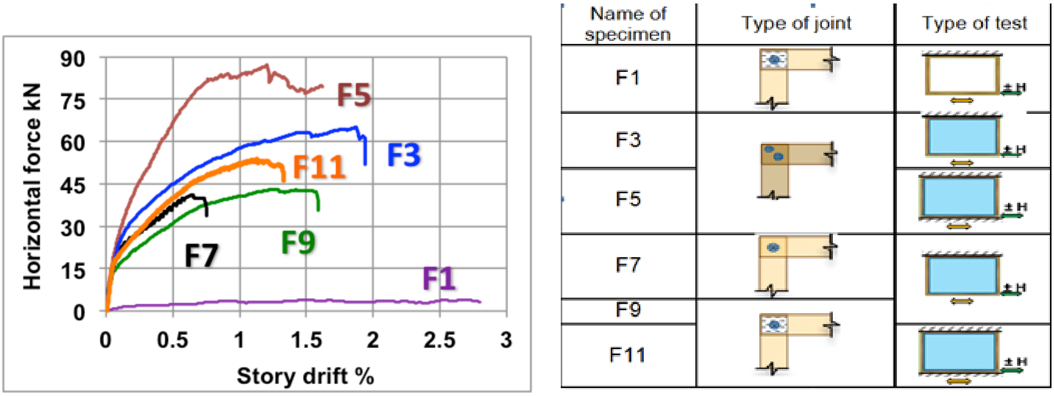


Figure 5. Comparison of diagrams of different frames with and without glass infills

Name of specimen	Maximal horizontal force (kN)	Corresponding story drift (%)
F1	4.0	2.66
F3	65.0	1.87
F5	87.1	1.21
F7	41.2	0.64
F9	43.0	1.24
F11	53.9	1.13

Table 2. The load-bearing characteristics of different frames with and without glass infills obtained by monotonous tests

3.3. Cyclic test of glue-laminated frames

The cyclic response of glue-laminated bare frame with bolted joints strengthened by punch metal plate does not much depend on boundary conditions as seen from diagrams in Fig. 6 below. Diagrams are showing only a part of response. Diagram in red is response of frame on monotonous load as is the case in all other diagrams of cyclic response in this paper. Frames were tested to the level of large deformations achieved at story drift of 7%. Damages of joints were reparable and deformed frame could be returned by horizontal pushing to the initial geometry. It is to be mentioned that the constant vertical force acting on 160/90 mm columns was 40 kN, practically equal to 10 times of value of the achieved horizontal load bearing. From the hysteresis loops calculated amount of energy dissipated by joints will enable estimation of energy dissipated by wood to glass interaction in the cases of infilled wooden frames. But due to different mechanism of response of frame joints in bare and infilled frame the frame deformation range in which comparison has sense should be defined by further analysis of test results and computational modelling.

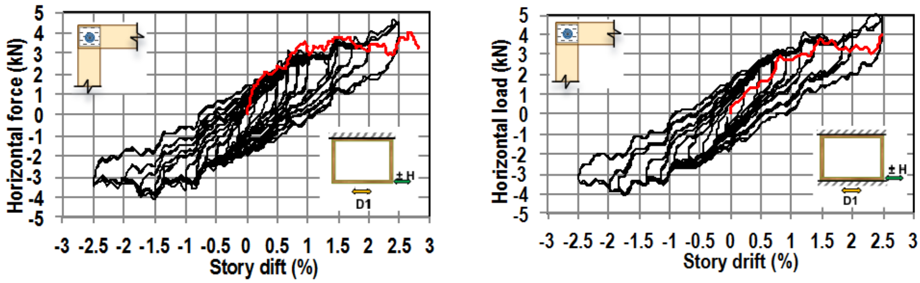


Figure 6. Response diagrams of glulam frames exposed to racking load with two different boundary conditions (shear cantilever and shear wall)

3.4. Cyclic test of frames with glass infill

Hysteretic response of tested specimens content the information on ductility of structural element, deterioration of strength due to repeating of horizontal load to equal displacement, cycle to cycle stiffness degradation and energy dissipation due to viscous damping of tested structure that passes different stages of gradual damaging of its parts. Tested type of structural elements is highly dissipative, where the main dissipation is caused by glass to wood interaction. Part of dissipation is caused by development of damages in joints and in some extends also by plastic deformations of frame anchoring elements to concrete base. Evaluation of stiffness degradation is explained in Fig. 9, and the evaluation of viscous damping in Fig. 12.

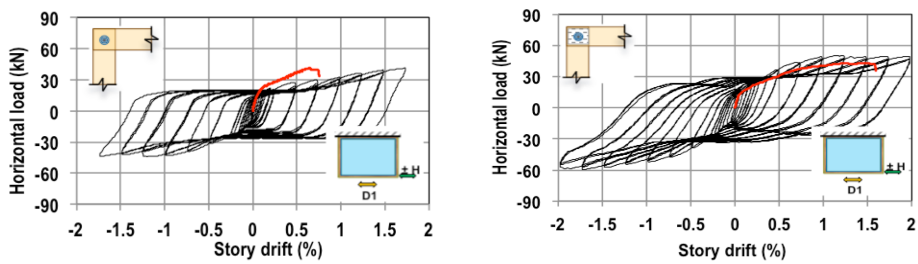


Figure 7. Response diagrams of glulam frames with glass infill exposed to racking load with two different joint configurations (bolted joints without and with punched metal plates)

The observed differences in response of glue-laminated frames (Fig. 7) are mainly the consequence of joint configuration. The punched metal plates limited the propagation of shear damages in joints increasing their load bearing capacity and ability to dissipate energy. It reflects in the all over increase of load bearing capacity and ability to dissipate energy through the glass-wood interaction. The effect of boundary condition to response of this type of frames will be observed in the future tests, but regarding the experience gained from testing of X-laminated frames (Fig. 8) they can be of significant magnitude.

The influence of difference in wood composition quality and joint configuration on hysteretic response of glass infilled frames is obvious from the comparison of diagrams presented in Fig. 7 and Fig. 8. From the Fig. 8 the influence of boundary condition is clearly recognisable. The same type of structural element can achieve much higher load-bearing capacity if supported firmly along the both horizontal edges (shear cantilever). In the case of tested specimens the specimen tested as shear wall has 46% higher load bearing capacity than one tested as shear cantilever. The difference is calculated as average value of maximal load achieved in opposite directions of racking

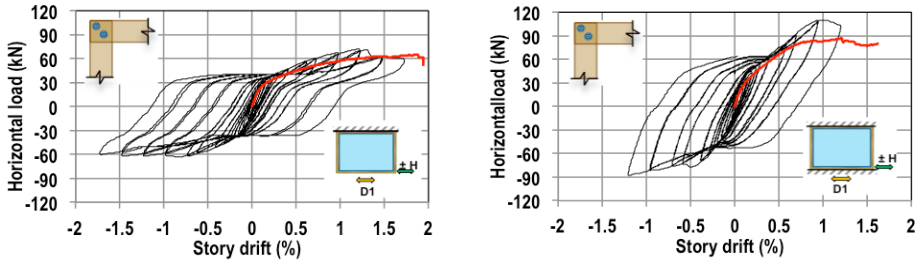
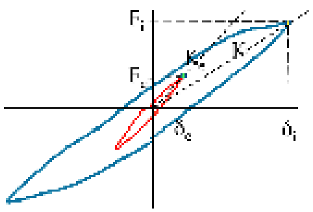


Figure 8. Response diagrams of X-lam frames with glass infill exposed to racking load with two different boundary conditions (shear cantilever and shear wall)

As mentioned before, one of main parameters describing the hysteresis response of structure is stiffness degradation. It can be calculated from the stiffness of the chosen loop and effective stiffness of the specimen in the early elastic stage. In the case of tested structural elements the effective stiffness (Ke) was calculated from the inclination of the several first hysteresis loops in the elastic range of response (Fig. 9). From the coordinates of the subsequent hysteresis turning points (δi, Fi) the inclination (stiffness Ki) of corresponding loops was calculated. The diagram of stiffness degradation can be mathematically defined by equation 1 below, where the parameter Ck is named “stiffness degradation factor”. Since its values are calculated from hysteresis response of tested specimens, it can be considered as their own, unique characteristics. Ck is very useful parameter that can be well employed in process of validation of computational models, when the experimentally obtained and calculated hysteresis responses are compared.



$$K_i = \frac{F_i}{\delta_i} \qquad K_e = \frac{F_e}{\delta_e} \qquad \frac{K_i}{K_e} = \frac{C_k}{\frac{\delta_i}{\delta_e}} \qquad (1)$$

K_e effective stiffness
K_i stiffness of corresponding loop
C_k factor of stiffness degradation

Figure 9. Definition of stiffness degradation factor C_k

In Fig. 10 and 11 below, the experimentally obtained data on cycle-to-cycle changing of degradation factor Ck are presented. The shape of curves is of the same kind, but values depend on the own hysteretic properties of each structural element. The values of Ck show a level of structural degradation expressed by lowering of its stiffness.

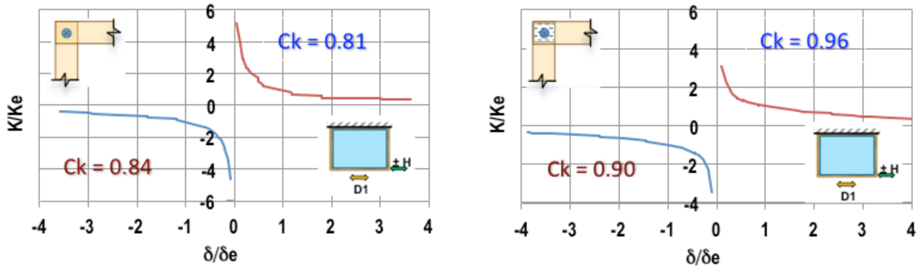


Figure 10. Experimentally obtained stiffness degradation factors Ck of glass infilled glulam frames

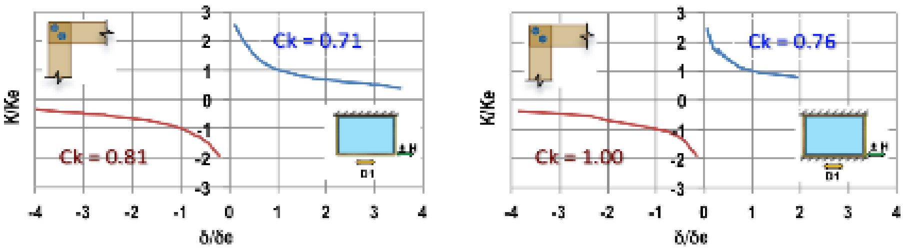
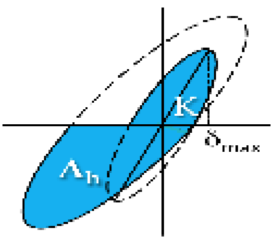


Figure 11. Experimentally obtained stiffness degradation factors Ck of glass infilled X-lam frames

It is obvious that due to damages of structure at certain level of deformation its stiffness at achieved equal displacement in the opposite direction of excitation is lower than when it was achieved for the first time. This ”softening” effect reflects in the values of Ck, which are in most cases lower in the direction of opposite excitation. The magnitude of differences in values of Ck shows in which extend of symmetry is development of damages in structural elements. In the case of tested types of structural elements the symmetry is much higher than in the cases of other types of structural elements as, for instance, reinforced concrete frames with masonry infills.

Another important parameter that quantifies hysteresis response of structural elements is the equivalent coefficient of viscous damping (ξ It can be calculated from hysteresis response as explained below and formulated by equation 2.



$$\xi = \frac{A_h}{4 \cdot \pi \cdot K \cdot \delta_{max}^2} \qquad (2)$$

ξ	equivalent coefficient of viscous damping
A_h	surface of the corresponding hysteresis loop
δ_{max}	maximal displacement of the corresponding loop

Figure 12. Definition of the coefficient of viscous damping

In figures 13 and 14 the envelopes of hysteresis loops and calculated coefficients of viscous damping are compared. The envelopes also show the amount of load bearing deterioration at repeating of deformations (three cycles to the selected displacement). In the case of X-lam frames the deterioration was insignificant, while in the case of glue-laminated frames it was relatively low.

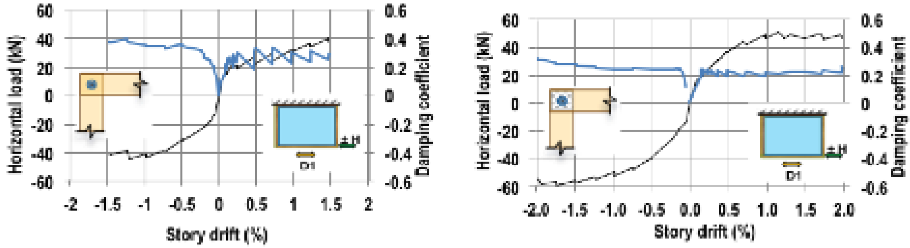


Figure 13. Experimentally obtained viscous damping coefficients of glass infilled glulam frames

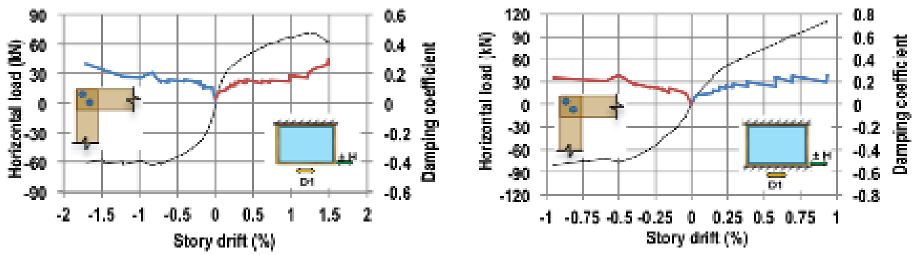


Figure 14. Experimentally obtained viscous damping coefficients of glass infilled X-lam frames

Diagrams, showing the values of coefficients of viscous damping in relation to The displacements can be observed as an illustration of hysteresis dumping in different stages of deformations of tested structure. In all cases the coefficient was in range of 0.2 to 0.4. In the case of x-lam frames it was up to 0.2, while in case of glue-laminated frames up to 0.4. It can be concluded that the configuration of joints had a significant influence. In two-bolted joints the coefficients were lower (Fig. 14) than in single bolted joints (Fig. 13). Punched metal plates influenced lowering of damping. Difference of viscous damping of glue-laminated frames in comparison to X-laminated frames was governed mostly by behaviour of joints. Joints behaved differently because of both wood mechanical characteristics and type of connecting frame components.

3.5. Shake table test of box-type model

Before the seismic testing, the dynamic characteristics of the model were obtained by measuring the ambient vibrations at selected points and processing the records by use of the Artemis software. The seismic excitations selected for the shake-table testing of the model were four representative accelerogrames recorded during the following earthquakes: El Centro (amax=0.34g), Petrovac (amax=0.47g), Kobe (amax=0.58g) and Friuli (amax=0.31g). The idea was to investigate the seismic behaviour of the model under several types of earthquake, considering their different frequency content, peak acceleration and time duration. The tests were performed in series, with increasing intensities until the damage occurrence of the model. The applied input intensities in a series were decided to be around the same percentage of the max acceleration (full scale) of the applied earthquake. The model frequencies were checked after each series of test by random excitation or by sine-sweep test. The final tests were performed by using the most unfavourable excitation, i.e. the earthquake Kobe, because it produced very intensive shaking and response of the model. The last 4 tests were performed by harmonic excitation having frequencies equal to the frequencies of the model after the seismic tests accomplishment, f = 4.0 Hz and f = 6.0 Hz, in order to see the effects of the resonance conditions.

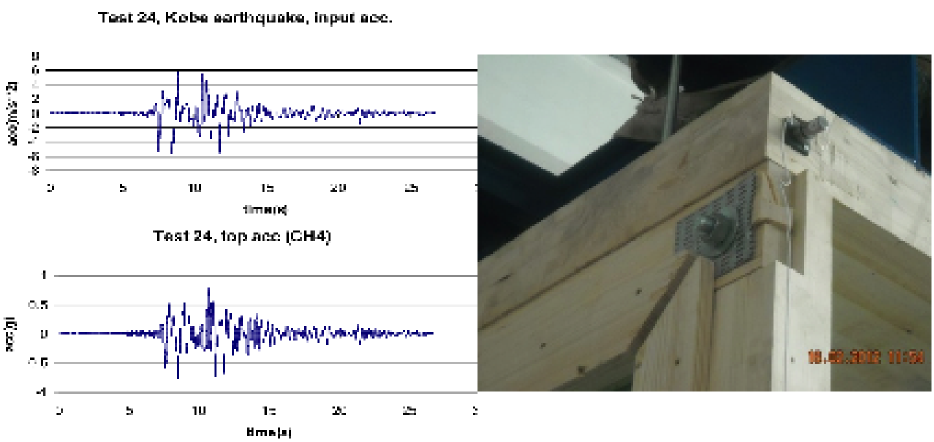


Figure 15. Excitation (max. input acc. = 0,6g) and response of model Kobe 1995 E-W, Japan earthquake (max. response acc. = 0.8g) and damages of upper frame joints after testing

The inelastic behaviour of model was achieved after application of full scale Kobe earthquake that was applied last in subsequent application of other three full-scale earthquakes. The damages caused by Kobe earthquake were limited to upper joints of frame, but their extent was much lower than in the case of racking load at its ultimate stage.

The performed tests showed clearly the behaviour of the glass infilled wooden frames and failure mechanism under strong earthquake motion. It is manifested by slip of the glass along the wooden frame and permanent deformations of the wood, without any damage in the glass. The panels dissipated energy trough sliding of the glass, development of damages in frame corners and activating of the still connectors that anchor frame to r. c. fundaments.

The seismic tests proved that the innovative composite panel could be considered as promising structural system, in which the load-bearing structural glass and the wood are working together, conforming to each other in beneficial manner. The dynamic tests results showed very good agreement with the results obtained during the racking tests of the panels.

ACKNOWLEDGEMENT

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SOCIETY FOR MATERIALS AND STRUCTURES TESTING OF SERBIA
DRUŠTVO ZA ISPITIVANJE I ISTRAŽIVANJE MATERIJALA I KONSTRUKCIJA SRBIJE

International Conference **"Marking 60 Years of Operation of DIMK**
Belgrade, October 19, 2012**.**

Radomir Zejak[1]

UDK:620.1:691]:628.4(497.16)

SOME CURRENT RESEARCH OF MATERIAL BASED ON INDUSTRIAL WASTE IN MONTENEGRO

Summary: *The aim of this work is to appropriately present part of current research on materials based on industrial waste in Montenegro. Immobilization of industrial waste, with less energy consumption. is one of the important projects on environmental protection. Its evaluation in the field of building materials in the world, though, has been launched. Use of industrial waste, in addition to raw materials for the production of inorganic binder, sintered products, based geopolimerised processes etc.. could significantly affect the protection of the environment in Montenegro, which is otherwise declared an ecological state.*

This paper presents selected results of research by a group of Civil Engineering and Metallurgy and Technology, University of Montenegro, Podgorica, noting that this kind of cooperation is of particular importance, especially when designing formulations for new products. Generally, it is a well-known but under-explored mechanisms for the formation of the specific raw materials.

Keywords: *Sintering, geopolimerization, structure, strength, fly ash, red mud.*

NEKA AKTUELNA ISTRAŽIVANJA MATERIJALA NA BAZI INDUSTRIJSKOG OTPADA U CRNOJ GORI

Rezime na maternjem jeziku: *Cilj ovoga rada je da se na prikladan način prezentira dio aktuelnih istraživanja na materijalima baziranim na industrijskom otpadu u Crnoj Gori. Imobilizacija industrijskog otpada, uz što manju potrošnju energije,. predstavlja jedan od važnih zahvata na očuvanju životne sredine. Njegova valorizacija u oblasti proizvodnje građevinskih materijala u svijetu odavno je pokrenuta. Korišćenjem industrijskog otpada, kao dodatka sirovinama za proizvodnju neorganskih veziva, sinterovanih proizvoda, proizvoda na bazi geopolimerizacionih procesa i sl. moglo bi se značajno uticati na očuvanje životne sredine u Crnoj Gori koja je, inače, deklarativno ekološka država. U ovom radu su prikazani odabrani rezultati istraživanja grupe autora sa Građevinskog i Metalurško-Tehnološkog fakulteta Univerziteta Crne Gore u Podgorici, uz napomenu da je ovakav vid saradnje od posebnog značaja, pogotovu kada se projektuju recepture za dobijanje novih proizvoda. Generalno, radi se o poznatim ali nedovoljno istraženim mehanizmima kod formiranja strukture materijala sa konkretnim sirovinama.*

Ključne reči: *sinterovanje, geopolimerizacija, struktura, čvrstoća, leteći pepeo, crveni mulj.*

[1] PhD, University of Montenegro, Faculty of Civil engineering, Džordža Vašingtona b.b. Podgorica, rzejak@t-com.me

1. INTRODUCTION

Laboratory of materials and structures from its inception (1980) is located at the University of Civil engineering, University of Montenegro in Podgorica. It was formed primarily for the practical training of students, which is essential in the construction of high-quality implementation of the curriculum.

Laboratory had from the beginning diverse (new) equipment which was current in the world at the time, and useful for testing materials and structures. Laboratory equipment is continually upgraded with various structural elements (Test Apparatus, devices for long-term load, etc..) in which they were able to examine different design prototypes. In that way it was possible to organize a high-quality scientific research experiment in which, as in other technical disciplines is a very important method.

Simultaneously with the expansion of construction in Montenegro growing need for standardized testing of construction product and construction, and was very broad and successful cooperation with the industry in which the Laboratory for Materials and Structures forefront.

As a result of these activities, it is safe to make the following statement.

For more than thirty years, all generations of students of Civil Engineering in Podgorica had a quality job training and the opportunity to check the theoretical training the experimental method on different models. Well-designed experiments with high-quality equipment were used to create three master's theses and sixteen doctoral dissertations.

It should, in fact indicate that the addition of Montenegrin researchers, experimental research for doctoral dissertation carried out in this laboratory, and two candidates from Serbia and one from the Republic of Serbian (Bosnia and Herzegovina) and China.

In the framework of international programs (IAESTE, CEEPUS II, ...), Laboratory of Materials and Structures in Civil Engineering in Podgorica was the host for several months of practical training for about 25 students from all over the world (Argentina, USA, Lebanon, Poland, ...).

In the field of cooperation with industry is important to emphasize testing and regular check-traditional materials and construction materials (stone, wood, brick, concrete, steel, ...).

Significant number of experimental trial testing and various construction and structural elements such as bridges, panels, crane paths, elements in buildings and other, which are necessary for the release of these facilities into operation.

It is important to note that in addition to standard laboratory tests, this laboratory do field testing of deals (in situ) as a geomechanical tests, current status and quality of building material and so on., using combined methods of destructive and non-destructive design.

Laboratory has achieved significant results in the field of certification and quality control of various materials and building products, which are imported from abroad.

Special attention is understandably devoted to scientific research and experiments that were organized for this purpose. This paper provides a brief review relating to an actual test material, since testing of structures and structural elements has a special place and represents a separate and much broader topic to handle.

Very fruitful and good cooperation with the Laboratory of Metallurgy at the University of Technology in the field of materials research is of particular interest for conservation and environmental protection in Montenegro should be especially noted.

Experiments were part of research projects, which is organized with the support of the Government of Montenegro in accordance with contracts signed with the University of Montenegro, Podgorica.

2. INDUSTRIAL WASTE IN MONTENEGRO

Besides the usual waste that accompanies every modern society in Montenegro there are significant amounts of industrial waste that could be valorized through the production of construction materials.There were done many different studies and researches considering it, there are also international experiences in the application of these waste materials in various forms of construction.

It is well known that in the process of manufacture Aluminium Plant Podgorica generates about 370 tons / year of red mud (Figure 1a). Regardless of what is not classified as hazardous waste it requires special treatment because it contains dangerous impurities. In addition, the capacity of the pool for its disposal, has long been obsolete, and with multiple pools of these upgrades is estimated that they have about 7.5 million tons of red mud.

Steel mill in Niksic is designed to produce 300 t / y of high-grade steel. In addition to other waste, slag is deposited within 3 miles of the factory in the quantity being measured millions of cubic meters. It should be noted that the landfill is located next to the river and has built an impervious surface, which is required for this type of waste.

Thermal power plant Pljevlja produces about 280,000 t / y. of ash, produced by burning coal. Although it is not classified as hazardous waste it requires special handling. Dam Maljevac (Figure 1b) which was expanded several times, is a very problematic structure whose collapse could produce disastrous consequences, and it is estimated that it deposited about 10 million tons of fly ash and slag.

Figure 1. Industrial waste: a) Aluminium plant Podgorica; b) Dam Maljevac (ash)

At the site of the former Repair Institute Arsenal in Tivat whose conversion is into a final stage, for decades conducted navy war ship repair, and within are large amounts of grit which is classified as a hazardous waste.

The shipyard in Bijela in Kotor Bay, where it continues to ship repair, also deposited large amounts of hazardous waste.

Flotation tailing Gradac,in the area of Pljevlja should be mentioned and a large overburden dump marl from coal mines. Flotation tailings in Mojkovac is finally sealing repaired, and the final arrangement of the terrain along the river Tara is underway.

Designated hot spot for ecological state of Montenegro is a serious problem, and the management of these wastes requires special attention.

Research on specific industrial waste should provide information on the possibility of its use as a raw material, where the specific conditions of the technological process of production, combined with other materials in order to obtain a product with properties relevant for use in construction. This could contribute significantly to economic progress. On the other hand, it is possible making permanent immobilization of certain industrial wastes and therefore contribute to the preservation of the environment.

3. REVIEW OF REALISED RESEARCHES

Forward to give some of the selected research results obtained by various authors from the Civil Engineering and Metallurgy - Technology Faculty in Podgorica. Seeing that their names are mostly found in the works of literature they will not be mentioned here. It should be noted that this type of cooperation is of particular importance, especially when designing recipes for the producti In fact, here the term new product, should not be understood as pretentious, since it is a well-known but under-explored mechanisms for the formation of the raw materials with concrete.

3.1. Sintered product on the basis of electrofilter ash [11].

Electrofilter ash contains silicates, carbonates and phosphates of calcium, magnesium, iron, aluminium and other elements [9]. Illite-kaolinite clays, apart from illite and kaolinite minerals, contain mostly also α-quartz, Fe_2O_3 and $CaCO_3$ [20]. The chemical and mineral content of the components of the raw material mixture causes, depending on the sintering temperature, the reactions in solid state, polymorphic transformations of quartz and liquid phase formation [12]. The firing regime and mineral content of raw materials have important influence on the relations between particular microstructural elements [3]. The acceleration of the solid state reactions is caused by the liquid phase formation (diffusion coefficient in such systems increases up to 1000 times) [13]. The compounds formed by solid state reactions during sintering process, i.e., new crystal phases, apart from above mentioned factors, are determined as well by the mineral and chemical content of clay [7], [10]. The samples are heated at the temperature sufficient for the oxidation of free carbon, present in ash, which could cause surface defects and the decrease of the sintered product strength. In the following phase the samples are heated to the sintering temperature to obtain the products with satisfactory characteristics regarding porosity and strength [14]. The content of electrofilter ash in the raw material mixture can be different (20-60%), depending on the additives, shaping method and sintering temperature [6].

3.1.1. Experimental

The raw material mixture for the production of samples on the basis of electrofilter ash was formed with the addition of illite-kaolinite clay (“BP”) as a binder. On the basis of preliminary results it was defined that the satisfactory characteristics of the sintered product regarding total porosity, volume shrinkage and compression strength could be obtained with the ash content of 30 mass % in the raw material mixture, if the pulverization of ash as well as addition of other components in raw material mixture were not performed.

The samples were shaped by plastic shaping in a mould corresponding to a parallelepiped with dimensions of 7.7 cm×3.9 cm×1.6 cm [5].

The characterization of the components of the raw material mixtures was performed with the determination of mineral content, chemical content and grain size distribution by granulometric analysis. For raw, unfired products, linear and volume shrinkage during drying in air [15] to the constant mass and drying in a dryer at the temperature of 110 °C were determined. The samples were fired at the temperatures of 800, 900, 1000, 1100 and 1200°C, respectively.

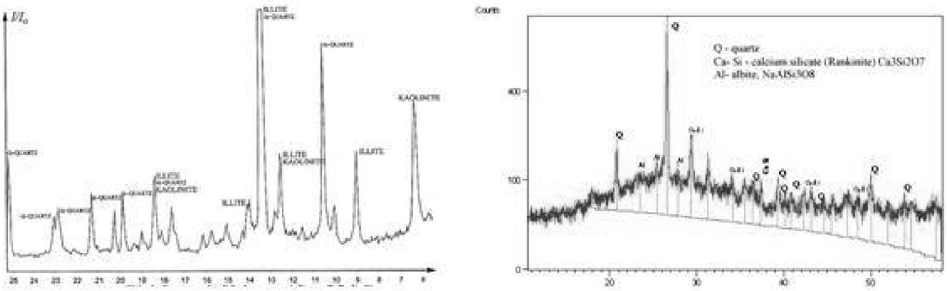


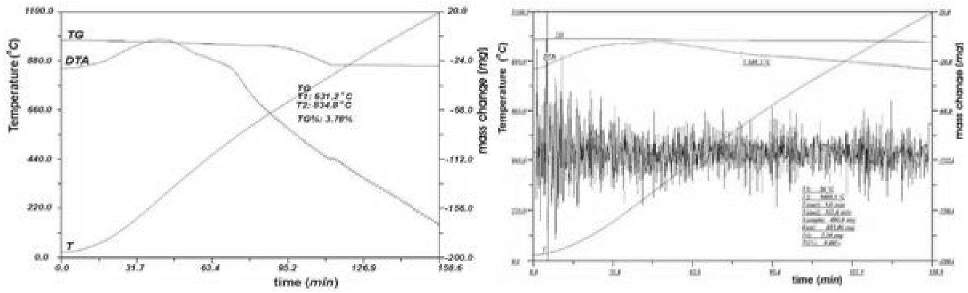
Figure 2. X-ray diffractogram of a) “BP” clay, b) electrofilter ash.

For the products fired at different temperatures there were determined: total porosity; linear and volume shrinkage during sintering; compression strength; microscopic and x-ray analysis of sintered products, as well as DTA and TG analysis.

3.1.2. Discussion of results

The mineral content of “BP” clay (Figure 2a) determined by x-ray analysis shows that the clay is an illite-kaolinite type, with presence of α-quartz. The X-ray analysis of electrofilter ash (Figure 2b) shows the presence of quartz, rankinite and albite. The chemical analysis results of “Bijelo Polje” clay and electrofilter ash show higher amount of Al_2O_3 in electro-filter ash (21.77 mass %) compared to the amount of Al_2O_3 in clay (10.98 mass %). The amount of SiO_2 is particularly lower in electro-filter ash (49.45 mass %) compared to the amount of SiO_2 in clay (71 mass %).

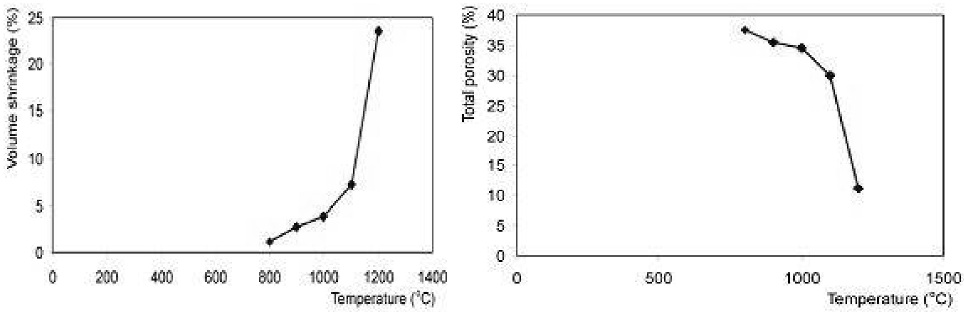
“Bijelo Polje” clay does not contain the following oxides, present in electrofilter ash: TiO_2, ZnO, MnO and P_2O_5. The DTA analysis of electrofilter ash (Figure 3a) does not show precisely defined “peaks” which correspond to endothermic and exothermic reactions. The changes in heating were registered in the form of slight inflections, where the first was registered in the temperature area of 305 °C-520 °C ($MgCO_3$ dissolution), and the other with the endothermic effect at the temperature of 728 °C as a consequence of CO_2 formation by thermal dissociation of $CaCO_3$. The most important mass change, according to the results of the TG analysis (Figure 3a) was registered within the interval from 654.9 °C to 827.3 °C, which corresponds to thermal dissociation of $CaCO_3$, according to the results of the x-ray analysis. The mass loss for this temperature interval was 3.78%.



a) b)

Figure 3. DTA and TG analysis of a) electrofilter ash, b) of sintered product (+ DTG).

On the basis of the results of granulometric analysis it can be concluded that electrofilter ash has higher average grain size (109 μm) compared to average grain size of clay (7.10 μm). In electrofilter ash the most common fractions are: 99 μm-114 μm (13.7%); 114-131 μm (14.3) and 131-150 μm (12.4%).



a) b)

Figure 4. Products during sintering a) volume shrikage; b) Total porosity.

In “Bijelo Polje” clay the most common fractions are 7.22-8.29 μm (4.7%); 6.29-7.22 μm (4.7%) and 5.48-6.29 μm (4.7%).The granulometric content of electrofilter ash and clay has important influence on linear and volume shrinkage of the products during sintering. The values of volume shrinkage during sintering increase with the increase of temperature (Figure 4a). The volume shrinkage increase is caused by solid state reactions and formation of new minerals as well as the formation of liquid phase (because the diffusion coefficient in such systems increases up to 1000 times). The alkali content in the components of the raw material mixture affects the increase of the liquid phase content, which accelerates the solid statev reactions that cause the formation of new structures (i.e., compounds). Electrofilter ash contains TiO_2 and MnO which have the role of mineralizers and have influence on polymorphic transformations of quartz.

During sintering the present carbon oxidates, surface defects appear and sample strength decreases. Total porosity during sintering (Figure 4b) decreases with the increase of temperature and has the lowest value at the temperature of 1200 ºC (liquid phase formation). Apart from the temperature, mineral and granulometric content of the components of the raw material mixture as well as shaping method also have the important influence on porosity. During formation of the raw material mixture the flux was not added, but the alkali content accelerates the solid state reactions as a result of the increase of diffusion coefficient.

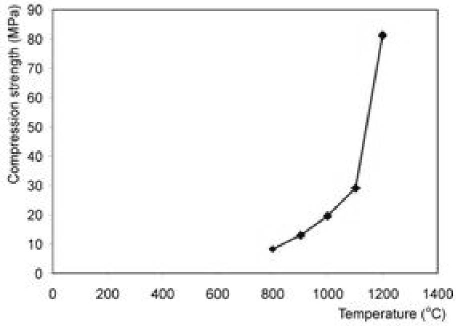
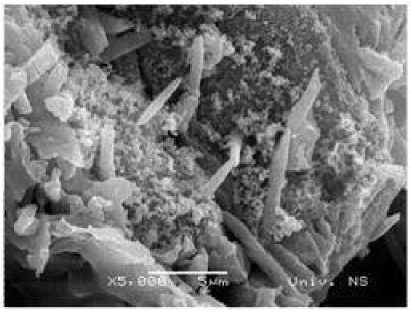
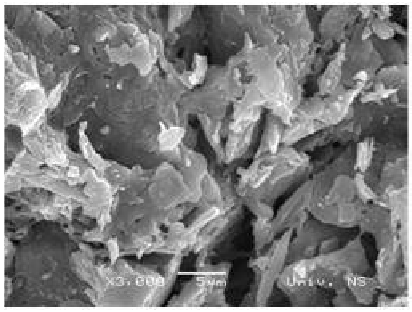


Figure 5. Compression stregth of products depending on temperatere of sintering

The values of compression strength increase with the increase of temperature (Figure 5). The relations between particular microstructural elements depend on the sintering level of ceramic mass and they can be influenced by the firing regime. At the increased temperatures the porosity values decrease (liquid phase formation) and the values of volume shrinkage increase.





a) *b)*

Figure 6. Microstructure of sintered product (t=1000 °C): a) enlarge 5000x; b) enlarge 3000x

The comparative x-ray diffractogram of the products sintered at 1000 ºC and 1100 ºC [15] does not show important differences in the content of formed minerals (albite, chematite, quartz). However, higher presence of liquid phase at the temperature of 1100 ºC has more important influence on the solid state reactions. The DTA and TG analysis of the sintered product (Figure 3b) show the total firing loss within the temperature interval from 25 to 1191.7 ºC, 0.46% in relation to the starting sample mass. The only visible change at DTA curved line is slight inflection with endothermic maximum at the temperature of about 588.3 ºC and it can correspond to phase transformation of beta quartz to alpha quartz. The microstructure of the sintered product (30% of ash and 70% of clay) at the temperature of 1000 ºC was shown in Figure 6a (increase by 5000 times) and Figure 6b (increase by 3000 times).

3.1.3. Conclusions

The investigation of the influence of the temperatureon the properties of the sintered product on the basis of the raw material mixture of electrofilter ash and illite-kaolinite clay shows that: Satisfactory characteristics of the sintered product regarding volume shrinkage, total porosity and compression strength, can be obtained at the temperatures of above 1000 ºC (optimum temperature 1100 ºC).

At the temperatures of above 1100 ºC, because of liquid phase formation, the difficulties may appear, therefore they are not recommended.

The investigations were performed without pulverization of electrofilter ash and addition of other components, which would certainly affect the properties of the sintered product.

3.2. Utilisation of electrofilter ash as a building material through geopolymerization process [17]

Fly ash is coal combustion product and it is recognized as by-product material from coal fired power station. Production of large quantity of fly ash impose the finding the solution of fly ash utilization. Production of elecrofilter ash and slag in coal fired power station – Plevlja in Montenegro is 44t /h and 5 t/h, respectively, and currently they are disposing to the landfill, leading to the environmental problem. A possible solution from the standpoint of environmental protection could be the utilization of fly ash, through the geopolymerisation process in a useful material that could be convenient for application in a civil engineering. Geopolymerization involves the chemical reaction of alumino-silicate oxides with alkali solution yielding inorganic polymeric material called geopolymer. These materials and geopolymerization process in general have attracted more attention since the 1990 s as a possible way of utilization of alumo-silicate based wastes.

The empirical formula of geoplymer is: $Mn\ [(-SiO_2)z\text{-}AlO_2]n \cdot wH_2O$ where Mn is a cation, usually an alkali (Na, K, Ca), n is a degree of polycondensation, $w \leq 3$ and z is 1, 2 or 3, [16]. Mixing of alumosilicate source and alkali solution generate geopolymer paste. This paste mixed in certain ratio with a sand or aggregate result in a geopolymer mortar or concrete, respectively. Mechanical and physic-chemical properties of geopolymer mortar and concrete mostly depend of the properties of geopolymer paste. So, fly ash based geopolymers are currently considering as a possible replacement for cement as binders in production of concrete and mortars. In such way materials traditionally used cement as a binder, could be replaced with a fly ash based geopolymers.

Investigation of geoplymerisation process is mainly focused on the investigation of multiple source materials with the aim to develop a methodology to optimize reaction parameters. These approach contribute to the environmental protection with respect to the fact that production of cement generate about the 5-7 % of global CO_2 production, [5], and the geoplymerisation process yield the environmentally friendly materials which do not emit the green house gases during the polymerization process.

The most important factors affecting properties of the geopolymers are: curing conditions (time and temperature) and type and concentration of alkaline solution (which determine in fact the ratios between main oxides, Na_2O/Al_2O_3 and SiO_2/Al_2O_3) as well as a solid / liquid to (S/L) ratio.

Besides, geopolymerization process may be successfully used for immobilization of hazardous waste (heavy metals, hazardous organic components, radioactive waste), [1], [18], [8].

The aim of this research was investigate the influence of reaction parameters, solid/liquid ratio, temperature of curing on the compressive strength and porosity of electrofilter ash based geopolymer. Besides, possibility of immobilization of used sandblasting grit into the geopolymer matrix has been investigated, as well. The spent sand blasting grit is solid waste generated in cleaning operations of tankers in the dock yard and it usually contain heavy metals which act as anti-fouling and anti-corrosion agents. Utilization of both, electrofilter ash and used sand blasting grit into the useful product would have a great significance from the standpoint of environmental protection.

3.2.1. Experimental

Fly ash that was used to obtain geopolymer paste is obtained from coal fired power station – Pljevlja. Chemical composition of fly ash is given in the Table 1.

Component	%
SiO_2	49.45
Fe_2O_3	5.23
Al_2O_3	21.77
TiO	0.66
CaO	13.34
Na_2O	0.46
ZnO	4.5×10^{-3}
MgO	1.29
MnO	0.02
P_2O_5	0.24
K_2O	1.40
Loss on ignition	4.35

Tabela 1. Chemical composition of electrofilter ash

Alkali solutions were prepared by mixing of 10M NaOH solution and sodium silicate solution (Na_2O = 8.5%, SiO_2 = 28.5%, density of 1.4 kg/m^3) with a ratio water glass / 10M NaOH of 2. Sodium hydroxide solution is obtained by dissolving of a proper amount of solid NaOH pallets in distilled water and sodium silicate solution was a commercial water glass supplied by Galenika Magmasil, Beograd. Electrofilter ash and alkali solution were mixed in mass ratio liquid/solid (S/L) of 0.5, 1, 1.25 and 1.5. Geopolymer paste obtained at the S/L ratio 1.5 were not considered because the bad workability of geopolymer mixture and impossibility of moulding.

A comparison in terms of liquid-to-electrofilter ash ratio was made to replicate ratio typically used to quantify compressive strength in ordinary Portland cement. Optimal theoretical cement-to-water ratio 2.5 (i.e. water-to-cement 0.4), which would correspond to the ash ratio/alkali liquid, couldn't be achieved because a bad workability of geopolymer paste. Experiments related to the investigation of possibility of used sand blasting grit immobilization into the geopolymer paste have been carried out at the same conditions but with addition, 10, 20 and 30% of grit. Used sand blasting grit is obtained from the shipyard Bijela. Geopolymer paste were casted in 28 mm x 8 mm a plastic moulds and cured in an oven at the specific temperature for 24 h. Curing temperatures were 65^0C and 75^0C. After 24 h the moulds have been demolished and the specimens were left at the ambient temperature for a next 6 days. After that, the specimens were cut to the high of 5 mm and tested for a compressive strength. Besides, initial and final setting time and total porosity of geopolymers were determined. A Vicat needle was used measure the geopolymer setting time. Total of porosity (TP) is determined from the following equations:

$$x_{1,2} = \frac{r - r_v}{r} \cdot 100, (\%) \tag{1}$$

Where, ρ is volumetric density and ρ_v is sample density

3.2.2. <u>Results and discussion</u>

Photographs of samples obtained at different solid/liquid ratio at the temperature of 75^0C are shown in the Figure 7(a-d). Geopolymer sample obtained at the ratio S/L=0.5 (Figure 7a) has not be considered due to the appearance of cracks after two days of curing at the ambient temperature. This behaviour of obtained sample is similar to the cement materials cured with water excess.

(a) (b) (c) (d)

Figure 7. Geopolymer samples obtained at different solid/liquid ratio cured at 75^0C for 24h; (a) S/L =0.5, (b) S/L =0.75, (c) S/L =1, (d) S/L =1.25

Change of compressive strength depending on S/L ratio is given in the (Figure 8a). It is evident that compressive strength of geopolymers increasing with increasing of S/L ratio, i.e. with increasing of proportion of electrofilter ash fraction in a geopolymer paste. Increase of compressive strength may be explained by the reduction of water content in a geopolymer mixture with increase of S/L ratio.

The change of compressive strength of geopolymer paste with change of S/L ratio, i.e. with a change of water content is the same as in the case of cement paste. Decrease of compressive strength with decrease of S/L ratio may be explained by the increase of total porosity of obtained geopolymers (Figure 8b). Increase of temperature of curing from 65^0C to 75^0C slightly increases the compressive strength, as well.

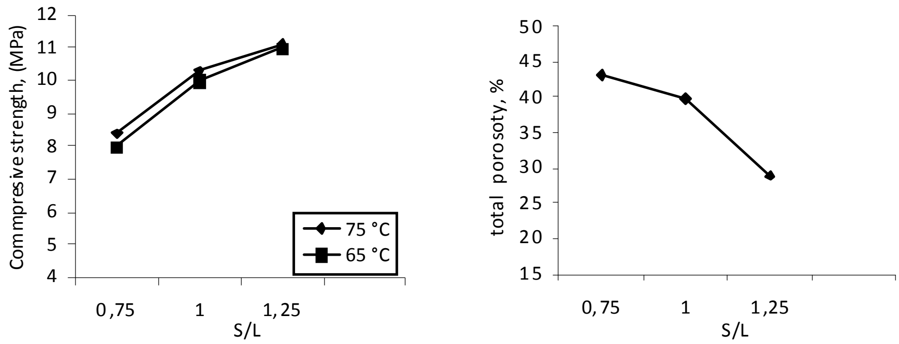
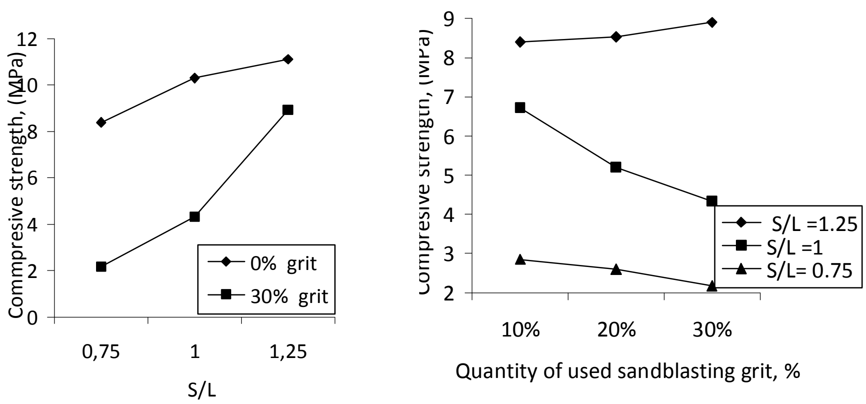


Figure 8. Influence of L/S ratio on the compressive strength
(a) And total porosity (b) of electrofilter ash based geopolymer

Investigations of possibility of immobilization of spent sandblasting grit into the geopolymer paste have shown that it can be successfully immobilised into the geopolymer paste. Compressive strength of geopolymer paste in this case is lower compared to the compressive strength of pure geopolymer paste obtained at the same S/L ratio (Figure 9a). Influence of quantity of added grit on the compressive strength of geopolymer paste is shown in the Figure 9b.



(a) (b)

Figure 9. a) Influence of used sand blasting grit addition on the compressive strength of geopolymers at the S/L= 1.25; b) Influence of spent grit quantity on the compressive strength of geopolymers

It is evident that the change of compressive strength with a quantity of added grit is different depending on the S/L ratio. At the S/L ratios of 0.75 and 1, decreasing of compressive strength with the increase of grit quantity is observed, while at the S/L ratio 1.25, slightly increasing of compressive strength with increase of grit quantity, is observed. This change in strength of obtained geopolymers can be correlated with the amount of liquid phase in the mixture. Namely, the grit contains the toxic components (heavy metals, organic toxic compounds) and their successful immobilization means dissolution and chemical bonding into the geopolymer paste. In this sense, quantity of liquid phase in a geopolymer mixture may have a significant impact on the effect of immobilization of spent grit. At the S/L ratio 1.25, quantity of liquid phase may be insufficient for dissolution of soluble components of grit and they remain physically trapped into the geopolymer paste. In that case metallic pallets (steel or cast iron) strongly contributed to the compressive strength of geopolymer and with increase of spent grit quantity the compressive strength is increased.

Important factor in a synthesis of geopolymers is satisfied workability of prepared geopolymer paste which enables casting of prepared paste into the molds. The water content controls the paste workability, but also the porosity of obtained geopolymers. Higher water content means the better workability but also a higher porosity and worse mechanical properties. In an experiments were the used sand blasting grit is added in a mixture for preparation of geopolymers, improvement in a workability of prepared geopolymer paste is visually observed. Increase in grit content added in the geopolymer matrix result in an improvement of paste workability. On the other side, initial and final setting time of geopolymer paste with addition of used sand blasting grit not significantly changed. Initial and final setting time of ash based geopolymer paste and paste with addition of 10, 20 and 30 % of grit were, 30, 30, 35, 35 min and 50, 55, 60, 60 min, respectively.

3.2.3. Conclusions

Investigations of geopolymerization of electrofilter ash have shown that:

Mechanical and physical properties, of electrofilter ash based geopolymers are strongly dependent upon the chemical composition of geopolymer mixture. Increase of solid/liquid ratio decrease the total porosity and increase the compressive strength of obtained geopolymers. Compressive strength is increasing with increase of curing temperature, as well.

Geopolymerization of electrofilter ash can be used for immobilization of used sand blasting grit. Compressive strength of ash-spent grit based geopolymers is lower compared to the compressive strength ash based geopolymer.

Addition of grit improves workability of obtained geopolymer and doesn’t change the initial and final setting time of geopolymers.

3.3. Geopolymerization process based on red mud*

In this study the geopolymerization process for obtaining construction materials based on red mud was used. The aim of this study was to define the most favorable conditions which would enable the utilization of geopolymerization process in production of construction materials based on red mud as the by-product of alumina production. For this purpose, physicochemical and mechanical properties of the obtained construction (geopolymers) materials were tested. On the basis of the results the optimal conditions of geopolymerization and the effect of the main synthesis parameters, from the aspect of satisfactory mechanical and other properties of the obtained materials, were determined. The inorganic polymeric materials produced by the geopolymerization of red mud developed satisfactory compressive strength which leads to the conclusion that these materials may be used in the sector of construction materials.

The geopolymerization process is based on heterogeneous chemical reaction that occurs between the solid, aluminosilicate-rich materials and highly alkaline silicate solution. The basic part of this process is hardening of geopolymers which is base on polycondenzation reactions of alkali precursors formed from the dissolution of active silicates and aluminosilicate solid materials in alkali hydroxide solution. Polymeric network as the result of polycondenzation process hardens rapidly acting as the gluing component [4].

The geopolymerization is an exothermic reaction that takes place on atmospheric pressure and temperature bellow 100˚C with the resulting formation of compact, solid materials, typical for their three-dimensional polymer structure. Such materials are called geopolymers [2]. The first stage in this reaction is the formation of hydroxyl complexes of silicon and aluminum with polymer bond type Si-O-Si and Si-O-Al, followed by the formation of three-dimensional aluminosilicate networks containing SiO_4 and/or AlO_4 tetrahedral, alternatively linked through a common oxygen ion. The last stage in the process is wrapping of the insoluble solid particles in the geopolymer [19].

The aim of this investigation is to define the optimal combination of relevant parameters which would enable using of red mud from alumina production process to be the dominant raw material in combination with activator and binder for the production of geopolymers. Geopolymerization creates the favorable conditions in promoting the red mud as the base for the development of new class of construction materials, inorganic polymers-geopolymers. The most important demands to construction materials are good physicochemical and mechanical characteristics, dimensional stability as well as good fire resistance and aggressive-environment resistance. The presence of hydroxyl Fe oxides in bauxite (goethite) compromises their use in conventional construction materials because of their dehydroxylation-hydroxylation activities generating dimensional instability.

Geopolymerization lowers the level of water absorption because of the amorphous or semi-crystal structure, lowers micro porosity, enables higher values of specific mass, compressive strength etc. The final objective of this investigation is to define the influence of relevant parameters affecting the geopolymerization. The compressive

* The paper (Utilization of geopolymerization for obtaining construction materials based on red mud, Authors: Vukčević M., Turović D, Krgović M., Bošković I., Ivanović M., Zejak R.) has been accepted for publication in Materials and Technology ISSN:1580–2949, UDK 669 +666 +678 +53.

strength, apparent density and microstructure of polymeric materials were investigated to define the optimal conditions of polymeric materials synthesis.

3.3.1. Experimental

For the production of construction materials the following raw materials were used:

– red mud obtained as a byproduct of the Bayer process of obtaining alumina. (Aluminum Factory, Podgorica),

– sodium hydroxide of analytical grade (Merck, anhydrous pellets),

– metakaolin, which provides initially the geopolymeric system with soluble silicon and aluminum that are essential for aluminosilicate oligomers formation and progress of the geopolymeriztion,

– sodium-silicate solution (Merck Na_2O: SiO_2 = 3.4, Na_2O 7,5-8,5%, SiO_2 25,5-28,5% and d=1,347g/cm^3)

Deionized water for the synthesis of polymeric material.

Red mud was originated from Aluminum metallurgical plant in Podgorica, Montenegro, as the by-product of alumina production known by the presence of hydroxylation Fe oxides, dried to constant mass at the temperature of 105˚C, and then sifted through a sieve with the diameter hole φ = 1mm.

Metakaolin is the dehydroxylation product of the industrial mineral kaolin in the temperature range between 650º and 850ºC. The thermal de hydroxylation of kaolin increases its solubility in alkaline media and was performed at 750˚C. The basic material was mineral kaolin from the site "Bijele poljane" in Montenegro. Metakaolin is dominantly amorphous material with minor crystallineconstitutes.

As an alkaline activator for the process of geopolymerization a combination of sodium water glass and sodium hydroxide was used. The activator solution was prepared by mixing the previously mentioned components 48 hours before the geopolymer production. Different concentrations of NaOH (C_{NaOH} = 3 mol/dm^3, 7 mol/dm^3 and 10 mol/dm^3) and concentration of Si in Na-silica (1 mol/dm^3, 1.5 mol/dm^3 and 3.5 mol/dm^3) were used. The level of substitution of red mud with metakaolin in solid phase was 4 wt% and 8 wt% and 15%wt.

The process of samples production was performed as follows:

– mixing of solid and liquid phases (solid and liquid phases ratio **-** 2.5 g/ml) until a fine, thick pulp,

– mass transfer in a rectangular mould with the cover,

– settling mould on the shaker for 10 minutes to displace the residual air,

– keeping the samples at the room temperature for 48 hours and

– keeping the samples in a dryer at the temperature of 100˚C for 72 hours,

– aging of the samples for 14 days.

3.3.2. Results and discussion

The chemical content of red mud was shown in Table 2, while the chemical content of metakaolin was shown in Table 3. The content of SiO_2 in kaolin was 59,87%, Fe_2O_3-3,12%, Al_2O_3-19,45%, water was the rest.

oxides	wt %
Fe_2O_3	40.78
Al_2O_3	17.91
SiO_2	11.28
TiO_2	10.20
Na_2O	6.9

Tabela 2. Chemical composition of electrofilter ash

The XRD analysis of red mud shows the presence of hematite Fe_2O_3, gibbsite $Al(OH)_3$, akdalaite $4Al_2O_3 \cdot H_2O$, lapidocrocte FeO(OH) and calcite $CaCO_3$.

The value of specific mass of red mud was $\rho_{CM} = 2.777$ g/cm^3, and for metakaolin the value was $\rho_{MK} = 2.474$ g/cm^3.

The investigation of chemical, mineralogical content as well as thermal characteristics was performed on several geopolymeric samples obtained under the different synthesis conditions. (Table 3).

The X-ray diagrams indicate that the treatment is characterized by dissolution of starting material and formation of amorphous and crystalline aluminosilicate phases and stable phases of leucite and kalsilite. The existence of non-dissolved solid particles of red mud is also indicated.

The unidentified peaks in XRD diagrams represent the residual unreacted kaolinite or sodium aluminosilicate phase. The selected diagrams show the existence of amorphous phase in the system (baseline noise) of aluminosilicate material. As from the diagram it is clear that increasing concentrations of NaOH as well as increasing participation of binder in solid phase, lead to formation of more pronounced peak of the new phase, i.e. sodium aluminosilicate.

oxides	wt %
SiO_2	52.26
Al_2O_3	42.83
Fe_2O_3	1.01
CaO	0.02
MgO	0.09
Na_2O	0.02
K_2O	1.56
TiO_2	0.13
ZnO	< 0.01

Table 3. The chemical content of metakaolin

The FTIR spectroscopy was used to determine the changes in structure during the treatment of starting material using the concentrated solution of NaOH and Na-silica.

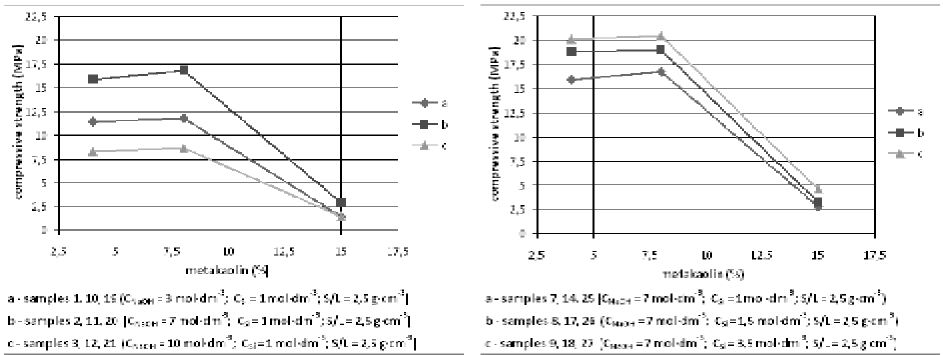
The TG - analysis was performed within the temperature range of 20 - 1200˚C.

The TGA results show that the mass loss occurs in two steps. In the first step, at the temperature under 150°C, the absorbed water is released in pores and on the surface. In the temperature range of 150-600°C, the weight loss is associated with the pre-dehydration process, where there is a reorganization of the octahedral lattice. In the second step, the dehydroxylation of the starting material occurs within the temperature range of 350-800°C.

The results show that geopolymer specific mass, depending of geopolymer synthesis conditions, was within the range from 2.215 g/cm^3 to 2.296 g/cm^3.The values of compressive strength of geopolymer in dependence of concentration of activator (NaOH - Na-silicate) and presence of binder are shown in Figure 10. (a,b respectively).

The factors affecting the increase of compressive strength are different concentrations of alkaline activators. The results show that with increasing concentrations of NaOH (3 mol/dm^3, 7 mol/dm^3, 10 mol/dm^3) the compressive strength increases up to the NaOH concentration of 10M. There, a reduction of the compressive strength occurs. The explanation lies in the fact that initial increase in NaOH concentration results in the increasing of silicon and aluminum dissolution from the solid phase [2].

Increased Si and Al contents in the aqueous phase are essential for initiating the oligomers formation and polycondenzation. The decrease of the values that occurs under higher NaOH concentrations is the consequence of the fact that dissolved silicon and aluminum remain almost constant while the free NaOH increases, resulting in lower SiO_2/Na_2O ratio in the aqueous phase. Therefore, the monosilicates and oligomeric speciaes are predominant in favor of polymers and consequently the polycondenzation is slower [2].



a) b)

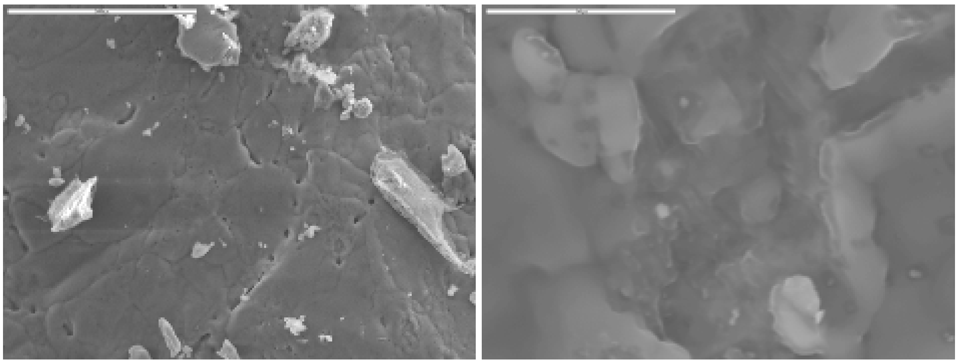
Figure 10. The values of compressive strength of geopolymer samples
a) Influence of NaOH conc. on compressive strength as the function of binder percentage b) influence of Si conc. on compressive strength as the function of binder percentage

Change in silicon concentration in Na-silica (1 mol/dm^3, 1.5 mol/dm^3, 3.5 mol/ dm^3) influence the compressive strength of geopolymer samples to increase. With increasing concentration of alkaline activator the amount of dissolved silicon in the reaction mixture increases. Silica originating from the sodium silicate has an important role because it starts the reaction of geopolymerization by allowing faster and more complete dissolution from the raw material [2].

Higher silicon concentration leads to the formation of silicate species with complex polymeric structure, thus giving the possibility to three-dimensional polymeric framework to rise.The soluble silica fasters the polycondenzation. Under higher initial silica concentrations, the surface cracks were noticed. This might be because of entrapped free water of the aqueous phase. (The water for the Na-silicate dissolution).

The level of substitution of red mud with metakaolin (4% wt, 8%wt and 15%wt) influences the in increase in compressive strength value up to the level of 15% wt. The lower level of compressive strength with the percentage of metakaolin of 15% and higher can be explained by lack of NaOH for dissolving of such a quantity of metakaolin or by the fact that high level of polycondenzation (because of the excessive metakaolin percentage) can create surface, non-permeable membrane which entrapped water from the liquid phase (water for dissolution of Na-silicate) [5].

The microstructure of the synthesized inorganic polymer material was investigated by scanning electronic microscopy and it is shown in Figure 11.



a) b)

Figure 11. The SEM microphotograph of inorganic polymer materials:
a) Increase by 500 times; b) increase by 5000 times

SEM microphotographs shows that the obtained materials are compact, with no discontinuity, which is confirmed by the mechanical properties. The isolated pores which are noticed inside the material are in the range up to 200 um. The presence of new amorphous phase can be seen in Figure 11a. In Figure 11b. under high magnification a gelatinous phase around the particles of starting material is identified.

3.3.3. Conclusion

This investigation shows that the red mud obtained as a byproduct of the Bayer process for obtaining aluminum in Aluminum Factory Podgorica according to its physicochemical properties represents good-quality alum silicate material appropriate for geopolymer formation.

Under the optimum synthesis conditions (S/L ratio 2,5g/ml, concentration of NaOH 7 g/mol^3, percentage of metakaolin 10%wt), the red mud/metakaolin-based polymeric materials develop satisfactory compressive strenght. Such the value of mechanical properties were attributed to the formation of amorphous phase which bonded the non-dissolved particles of the raw solid materials in a good manner. The presence of this phase was also revealed by the XRD, TG and FTIR analysis as well as SEM analysis. From the aspect of density all geopolymer samples can be used for construction materials.

4. BRIEF REVIEW OF THE PRESENTED RESEARCHES

In addition to the specified tests, whether preliminary or detailed, a large number of influential parameters were analyzed. For some materials other relevant properties were examinated in addition to the basic structure and physic-mechanical properties to choose its usefulness-purpose. On the sample of geopolymers with addition of fine aggregate, tensil strength in bending, Modulus of elasticity, RDM, frost resisrance were studied. Tests were made with samples of coarse aggregate, geopolymers concrete, where depending on the type of aggregate and additives used and the degree of workability obtained on a very different behavior of these materials. For the design mix for geopolymerisation natural mineral bauxite was used from the area Bijela Poljana near Niksic. It should be noted that at the Laboratory for Materials and Structures Civil Engineering in Podgorica, research was conducted on samples of classic concrete, which are used as an additional component of grit, slag, fly ash, red mud.

All these tests represent the basis for further research work in order to obtain useful products or technological processes for the immobilization of large amounts of industrial waste in Montenegro.

5. CONCLUSION

Use of industrial waste, in addition to raw materials for the production of inorganic binder, sintered products, based geopolimerised processes etc.. could significantly affect the protection of the environment in Montenegro, which is otherwise declared an ecological state.

Based on preliminary tests, given the information that can be obtained from the relevant sources of world scientific literature, it was shown that such industrial materials can be used to obtain modified products in construction .Three options were provided here for the presentation, which deserve special attention according to the authors.

Sintering of clay as the main raw material, based on a tinge of fly ash could be obtained clay products and satisfactory performance of physic-mechanical properties.

Geopolymerisation based on fly ash with a relatively cheap products of chemical industry may also produce the desired effects and appropriate products and useful construction material.

Geopolymerisation based on red mud from the corresponding products of chemical industry and without excessive energy consumption, it may be meaningful chemical reaction in the formation of the corresponding structure of the final material – geopolymers

Experiments were part of research projects, which is organized with the support of the Government of Montenegro in accordance with contracts signed with the University of Montenegro, Podgorica.

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Damir Zenunović[1]
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ANALYSIS OF PRECAST CONNECTION OF RC MONOLITHIC WALL AND RC PRECAST OR MONOLITHIC SLAB

Abstract: *In the paper was given review of comparative experimental and numerical research in order to defining behaviour of RC monolithic wall and RC precast slab, which was proposed as typical connection of prefabricated building system named „MMS", developed in Tuzla 80's. Within experimental research 3 precast specimens and 3 comparative monolithic specimens are tested. Within numerical research mathematical model is proposed taking into procedure behaviour of connection. Also, the Strut & Tie model is proposed for engineering practice. At the end of the paper reliability analysis of the obtained results was given.*
This paper presents an overview of the papers published from 2007 to 2012.

Key words: *precast structure, precast connection, semi-rigid connection, modified stiffness matrix, strut and tie model, reliability*

ANALIZA MONTAŽNOG SPOJA ARMIRANOBETONSKOG MONOLITNOG ZIDA I ARMIRANOBETONSKE MONTAŽNE ILI MONOLITNE PLOČE

Rezime: *U radu je dat pregled uporednih eksperimentalnih i numeričkih istraživanja sa ciljem definisanja ponašanja montažnog spoja armiranobetonskog monolitnog zida i armiranobetonske montažne ploče, koji je predložen kao tipična veza kod montažnog sistema građenja MMS, razvijenog u Tuzli 80-tih godina. U sklopu eksperimentalnih istraživanja ispitana su 3 montažna modela i tri uporedna monolitna modela. U sklopu numeričkih istraživanja predložen je matematički model uz uzimanje u obzir ponašanja spoja. Isto tako je predložen približni štapni model za inžinjersku praksu. Na kraju rada data je analiza pouzdanosti dobijenih rezultata.*
Ovaj rad predstavlja pregled niza radova objavljenih u periodu od 2007 do 2012.godine.

Ključne riječi: *montažna konstrukcija, montažna veza, polukruta veza, modifikovana matrica krutosti, štapni model, pouzdanost*

[1] Prof., University of Tuzla, Faculty of Mining, Geology and Civil Engineering, Univerzitetska 2, Tuzla, Bosnia and Herzegovina, email: damir.zenunovic@untz.ba

Foreword

During the 60 years of the former JUDIMK, and now DIMK Serbia number of authors from Bosnia and Herzegovina has given his contribution to the field of testing and research materials and structures. Results of their research are published by the conferences, symposia and journals in these areas.

In the field of materials research is important to mention: Academician Julije Hahamovic, Sahzija Dzonlagic-Dreca, Borislav Stanivukovic and Nadezda Knezevic-Vuksanovic.

In the field of structure research included the following researchers:

- Theory of structures: Svetislav Milic, Ranko Smiljanic, Nina Verbic, Ognjen Jokanovic, Branislav Terzic, Adam Ibrahimbegovic
- Concrete structures: Vjekoslav Marendic, Mehmed Beganovic, Muhamed Zlatar, Vahid Hasanovic
- Steel structures: Sead Ferusic, Slobodan Studen, Bogdan Kuzmanovic
- Geotechnics: Branko Krsmanovic, Hamid Dolarevic, Zlatko Langof

1. INTRODUCTION AND LITERATURE REVIEW

1.1. Introduction

Prefabrication of concrete started at the beginning of the last century, and its rapid development experienced after World War II, when the reconstruction of housing and infrastructure demanded faster and industrialized construction. According to [56] first prefabrication in the former Yugoslavia was in 1948 at the highway Belgrade - Zagreb, prefabrication of bridge girders on the bridge over Bosut. With introduction of prestressed concrete, prefabrication and assembly are being used more intensely in construction practice. Larger building companies were developing a variety of prefabricated building systems. Great contribution to the prefabrication of concrete structures has been achieved through the construction of bridges. Further development of prefabrication was application in building construction.

Prefabricated buildings can be divided into: panel (with large-sized elements) (Fig.1), frame, spatial (cellular) and mixed (combined).

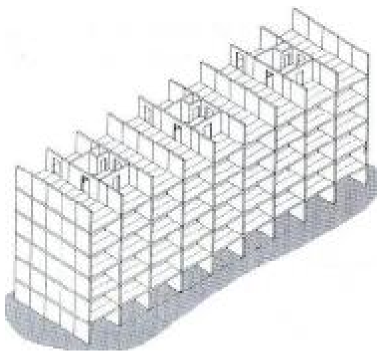

Fig. 1. Large panel prefabricated bulding system

Most prefabricated building systems applied in residential and public buildings in the former Yugoslavia were panel building systems. Application of industrialization in residential buildings construction started in late fifties. The first solution industrialized production of residential buildings in the panel system was developed by company „Jugomont“ Zagreb, which later merged with the well-known large companie in our region, „Industrogradnja“. Same year, the company „CAMUS“ started industrial production in France. From the very beginning until today, a number of building companies is built using prefabricated building panel system, such as Trudbenik Belgrade, Adriamont Rijeka, Karpos Skopje, Vranica Sarajevo, Kongrap Belgrade and other.

At the end of the seventies of the last century company "Rad" Belgrade purchased license for prefabricated bulding system the French firm "Balency." This prefabricated building system is slightly modified for local conditions and applied under the name "Balency - Rad" [2]. Modification of the prefabricated building system "Balency" was the result of experimental and analytical studies of the Institute IZIIS Skopje and Faculty of Civil Engineering Belgrade. As a result of the joint research was suggested new modified connection system, which allows the correct behaviour of ductile structures in the nonlinear region, The research was an example of a comprehensive research work on improving the building system [46].

Results of the research were used as a basis for development prefabricated building system named „MMS“ in 80th of the last century in Tuzla, which is scheduled for construction of residential buildings. Bearing structure is designed with monolithic reinforced concrete walls (MRCW) and prefabricated reinforced concrete slab (PRCS). Connection of MRCW and PRCS is dislocated in the span, in order to prevent damage of wall-slab joint during the earthquake (Fig.2). The prefabricated building system has been applied experimentally in the construction of a residential area in Tuzla. In order to define the mechanism of the proposed precast connections experimental studies and numerical analysis were performed. This paper presents the results of research. Some results of the research were presented in the papers [20], [49] to [55].

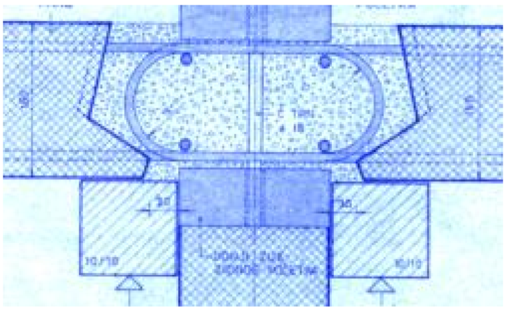

Fig. 2. MMS connections

1.2. Literature review

In modern civil engineering prefabricated building structures becomes a significant share of building structures. Generally, prefabricated structure is composed from two sets: elements and connections. Basic difference between monolithic and precast RC structures consist of large number of precast elements connections. Their function is to transfer load from one to another connected element and to provide structure integrity with suitable reliability. Selection of system of connections influences not only on the structure safety, but on durability, production and erection procedure. Behavior of precast concrete structures in great degree depends from their quality. Existing precast building systems are improved in a way to achieve the quality with large reliability. Joints/connections are the most sensitive parts of the precast structures. Precisely defining the behaviour of the precast connection is a fundamental problem in the analysis of prefabricated structures as well as creating a numerical model of real behaviour. Also, there are various prefabricated building systems, and it is the problem of uniformity of numerical models.

In general, connections can be divided into wet and dry. Most common solution of bearing elements connections is with site concrete filling, i.e. monolithization (grooving). Due to different characteristics of materials in and out of joint, joint area of individual elements and elements itself have different behavior. Therefore differences appeared between real and designed static system. Different behavior of connections relatively to elements is introduced through notion - connection yielding. When calculating monolithic RC structures, an integral structural work is assumed, with stiffly connected structural elements and the requirement that the failure of structural elements occurs before the failure of their connection. In Europe, such calculation concept is used also in the numerical modeling of connections of prefabricated RC elements. By calculation procedure connections are treated ideally rigid or of the hinges (pinned). In reality, all connections are semi-rigid. Total precast frame analysis may therefore be carried out by substituting rigid joint connections with ones of finite strength and rotational stiffness. The behaviour may be described in terms of the well known moment-rotation data, but in the case of precast slab - monolith wall connections, the semi rigidity is due to both material and large deflection effects.

The real mechanism of operation of prefabricated connections can be determined only experimentally. For this reason, many authors have implemented their experiments in individual prefabricated systems, in order to define calculation models. This experimental database and the associated numerical analyses provide guidelines for the description of working mechanisms and failure mechanisms in prefabricated connections. Here, some particularities and representative research results were specified that indicate the behavioral properties of prefabricated joints and connections, which were the basis for the definition of certain aspects of methodology for the analysis of such connections.

Experimental and numerical researches of behavior of precast connection started in fifties of 20th century. Some of the researches results are presented in [40] from a group of authors that worked on testing of connection applied in precast building systems in ex Soviet Union. Examination of large panel structures, performed by Pommeret and Robinson, were presented on CEB session in Ankara 1964. Base on performed researches CEB have published recommendations for large panel structures. Researches related to shear transfer through connection and yielding were performed in sixties on Washington University by Mattock and by Cranston. Experimental research results obtained at large panel connection models and forms for determination of shear capacity were published by Robinson and Foure [39]. In order to define the stress transfer in prefabricated reinforced concrete elements, Guillaud and Morlier [24] carried out an experimental study of prefabricated concrete-filled connections with a variety of shapes and positions of prefabricated joints and joint infill. They processed the results by modifying the terms for monolith structures with correction factors, which take into account the actual behavior of the connection. “PCI Special Research Project No. 1/4”, was performed for the purpose of investigations of precast connections and their bearing capacity [13]. In April 1991 PRESSS research program was started with goal of definition behavior of precast structures imposed to seismic load. Program was started parallel in USA and Japan. The part the entire program was connection classification. In the papers [24] and [41] research results and evaluation of particular connections are presented. In [8] an overview of experimental researches of precast beams and columns imposed to seismic load, performed 1987 in National Institute of Standards and Technology in Gaithersburg, Maryland. In [15] some researches results were given aiming to modify the connector in order to achieve precast connection ductility. Also, in books [11] and [16] the procedure for calculation of bearing capacity and ductility of structural elements and connections were given. Those calculations were based on theoretical knowledge gained so far and performed experimental researches on reinforced concrete (RC) and prestressed concrete (PC) monolithic frame. In [32] research results obtained on 18 models (samples) with intention to define behavior of 2 types of precast beam-column connection recommended by PCI Committee and Australian Prestressed Concrete Group. In paper [27] authors presented results from experimental researches performed at Technical University in Aachen. Testing was performed on 24 models of inside and outside beam-column connection. On the basis of performed research models of predicting load failure and failure mode were developed. In papers [12] and [13] the classification of experimental researches on 4 different types of precast connection is presented. In paper [12] the overview of experimental researches results is presented, while in paper [13] analytical forms for determination of bearing capacity of treated precast connections are presented taking into consideration connection yielding. Evaluation of connection yielding was compared with beam-line. Based on of performed researches, analytical studies, result, proposals of modified forms for calculation procedure of precast connections were proposed.

Paper [1] presents the overview of analytical forms for calculation of bearing capacity of precast connection exposed to compression, shear and bending, for different connection systems with monolithization on site and with steel protruding bar (dowel) and profiles. Also, in book [5] design procedures of precast concrete structures were given. In [33] are published research results giving the classification of connections, mechanisms of load transfer through connection and proposing FEM model for connection stiffness with springs. RILEM-CEB-CIB symposium “Mechanical and Insulating properties of joints of precast reinforced concrete elements” held in 1978. One part of entire work session of cited Symposium was related to large panel and characteristics of large panel connections. Among numerous papers at Symposium was a very interesting [43]. Forms for determination of shear bearing capacity of precast connection based on performed experimental researches were presented in that paper. In [47] an overview of 40-year experiences in design of precast structures was given, as well as recommendations for increasing of precast structure stiffness. In 1995 Sucuoglu in the paper [42] gives forms for determining the moments at the ends of precast beam, ie, at the junction of precast beams and columns. The paper discusses the effect of joint stiffness on seismic response of precast structures. Issue of relocation of potential plastic hinges away from the vertical elements (columns and walls) Restrepo, Park and Buchanan [38] are discussed and presented in their paper, published in 1995, where they gives the procedure for calculating frames with short beams where the junction displaced into the middle of the range.

Special attention is given on definition of behavior of precast connections exposed to seismic load. According to that, CEB published Draft guide for the design of precast wall connection [10]. The same year UBC guide for the design of connections in seismic region was adopted. At 9th European Conference in Rotterdam Tsoukantas presented a paper [44] where the behavior of prefabricated connections under seismic load was analyzed, and the behavior of monolith connections compared. He proposed analytical expressions for predicting the response of connections exposed to seismic actions. At 10th European Conference in Vienna paper [35] was presented, which compared behavior of monolithic and precast connections imposed to seismic action and proposed analytical forms for prediction of precast connection response on seismic action. Work mechanism of connection exposed to compression and shear, local ductility, and mechanism of energy dissipation also was described in the mentioned paper. At 10th European Conference on Earthquake Engineering. The Report of EAEE work group 4 under the title “Precast structures in seismic regions” was presented, where trends of researches in Bulgaria, Italia, Russia, Romania, Turkey, Former Yugoslavia, Japan and USA were surveyed.

In FIP 7, Task 6.4.7, page 60, is given recommendations for design of monolithic and precast concrete connections. In the paper [30] rotational spring stiffness-connection ratio relation is explained and revealed. The finite element analysis on 4 types of precast connections which are pinned, rigid and semi rigid is presented in the paper [17]. The stiffness of connections is obtained from the slope of the total load versus deflection graph in the elastic range. Model of precast joint using 3D solid elements and surface-to-surface contact elements between the beam/column faces and interface grout in the vicinity of the connection was presented in the paper [26]. Results from the finite element analysis correlate fairly well with experimental results. Iterative process of nonlinear analysis with introducing stiffness of connections presented by Carol and Murcia in [6]. In each iteration step is taken redistribution of shear forces and new stiffness as a result of the previous iteration.

In paper [28] Jirasek presented modified matrices for the stiffness of beam element, as constituents of frames, where the effects of connection yielding were taken into account. The yielding of connections is introduced through modification of stiffness matrix. Yi, Yun and Feng presented in the paper [48] modelling of beams with semi-rigid connections and modified stiffness matrix. The stiffness matrix for the element with elastic yielding ends was presented in the paper [21] and the book [45]. Modeling plastic deformation by the use of finite elements is formulated by Ramm and Kompfner in [37].

Experimental researches are necessary for defining the characteristics of connection yielding. A lot of researches have been performed and it represents considerable data base of characteristics of precast connections. Some of them are mentioned. According to the findings modifications of monolithic structure calculation are made. However, problem of unification and application of uniform analytical forms for different precast connections still exist. In order to attain a goal it is necessary to increase the data base for connection characteristics, with further experimental and analytical researches.

Experiences of the above-mentioned experimental researches are used in the preparation of the experimental program for the researches presented in the paper.

2. EXPERIMENTAL RESEARCH

2.1. Experimental program

Connection of MRCW and PRCS of MMS prefabricated system is dislocated in the span. The analysis of dislocated connection of the walls and slabs is based on the experiences of previous empirical and theoretical basis. The program of experimental and numerical research of these connections was made in the period from 2004 to 2007. Connections of two slab types: 1-precast, 2-monolithic and monolithic wall are studied. The research objective was to define connection rigidity and investigate its behaviour in relation to monolithic connection. Preliminary the results of the experimental studies were published in the paper [49], [50] and [51], and the research was described in details.

In accordance with experimental program the research program consists of comparative experimental and numerical research. Experimental investigations include testing of three specimens with precast connection (Fig. 3) and three comparative monolithic specimens. The aim of tests is the comparison of precast and monolithic connection behaviour, as well as definition of connection yielding. Thickness of PRCS was 16 cm; while monolithic part of structure (MRCW) had thickness 15 cm. Monolithic specimens are performed with the same geometrical parameters. Recommendations given in [10] are used in selection of geometrical relation of specimens (Fig. 4).

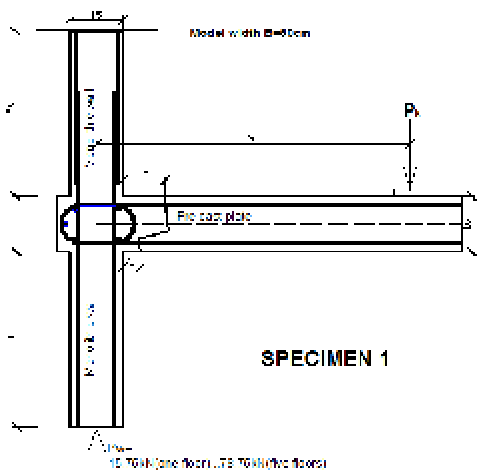

Fig. 3. Specimen with precast slab and monolithic wall

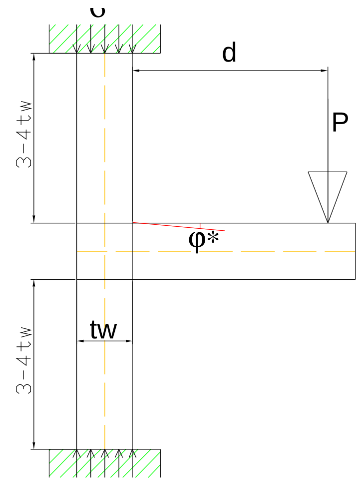

Fig. 4. Adopted geometric relationship after [31]

Based on proposed relation of wall segment height and thickness *(3-4)* t_w wall height 50 cm was adopted. Cantilever part of model (PRCS) was exposed to load at distance 90 cm from wall axis. This distance was determined on the basis of experiences gained in similar experiments, which confirmed that corbel effect run out at relation a/h = 5.0. In this case a/h = 90/16 = 5.625. In accordance to proposal of authors of treated precast system, connection of precast and monolithic part of structure was performed with tooth. Connection bottom was dislocated 5.0 cm from wall internal surface, while connection top was dislocated 12.0 cm, due to technological reasons. Adopted width of the model was 50 cm. Monolithic parts of the structure (MRCW and joint) are performed from concrete class C 25/30, while PRCS is made of concrete C 30/37. Reinforcement (rebar) S 400/500 (f_{yk} = 400 MPa) is used in the construction. Longitudinal rebar (loops) are Φ12/15 cm, and distributing rebar Φ10/30 cm. Bars for loops stiffening are Φ12. Walls are reinforced with mesh f_{yk} = 500 MPa (Fig. 5).

During the manufacture of the specimens, samples of in-built materials (concrete and steel) were tested in order to determine their mechanical properties (Fig. 6).

Fig. 5. Reinforcement

Fig. 6. Concrete and steel samples

The specimens are placed into a claw press of maximum capacity of 6500 kN, which is used to restraint the specimen. The walls of specimen are pressed by loading that causes normal stresses of $0.4f_{ck}$. The degree of constraint is controlled by deflectometer at the measurement points U_6 and U_7 (Fig.7). The samples are loaded by a mobile hydraulic press of 100N measurement accuracy. Loading of specimens was applied in 4 phases, up to the service load P_{serv}. The service loading of specimen (P_{serv} = 17.18 kN) is defined as the loading at the wall/slab junction caused by internal forces of the same intensity as in a real structure with the slab range of 6.15m. Namely, the distance of the bearing walls of MMS systems is 6.15m.

After the phases P/2 and P specimens were unloaded. After that, they were loaded up to failure. The following measurements were performed for every load phase:

- Strain of reinforcement loops. Steel bolts welded on bars were used (Fig. 5 and Fig.7). Measurements were executed with mechanical extensometer type Demac with measurement base of 250, 150 and 50 mm and measurement accuracy of 0.001 mm. Point of measurements, marked as D_a, is presented in Fig.8;
- Strain of concrete is marked as D_n (n = 1,2,3,...) (Fig.8);
- Deformation (deflection and rotation) of walls, joints and slabs with deflectometers of 50 mm range with accuracy of 0.01 mm and 10mm range with accuracy of 0.001 mm is presented (Fig. 7).

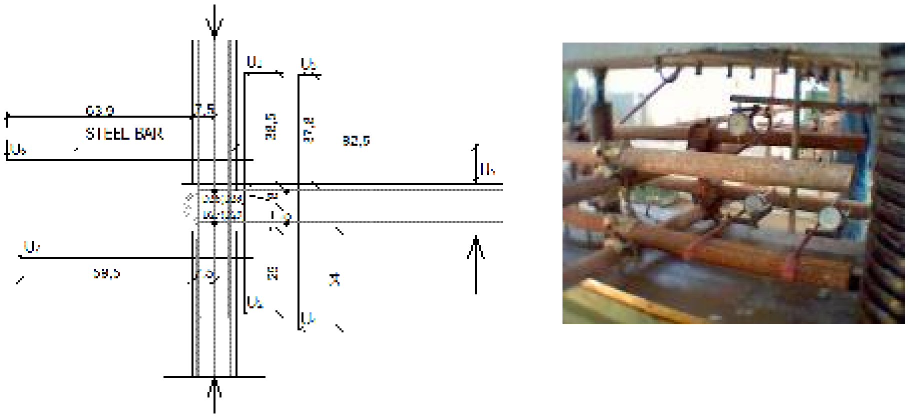

Fig. 7. Arrangement of deflectometers

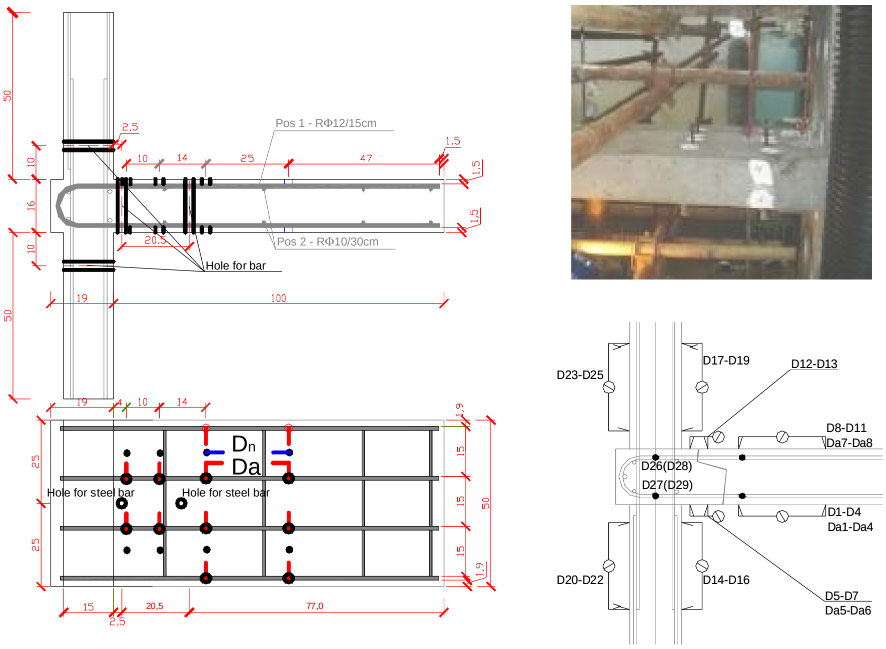

Fig. 8. Arrangement of measurement points

2.2. Experimental results

During manufacture of models, samples of in-built materials (concrete and steel) tested in order to determine mechanical characteristics as follows: concrete pressure strength and modulus of elasticity (Fig. 9), and reinforcement yield and tension strength (Fig. 10). Results are presented in Table 1.

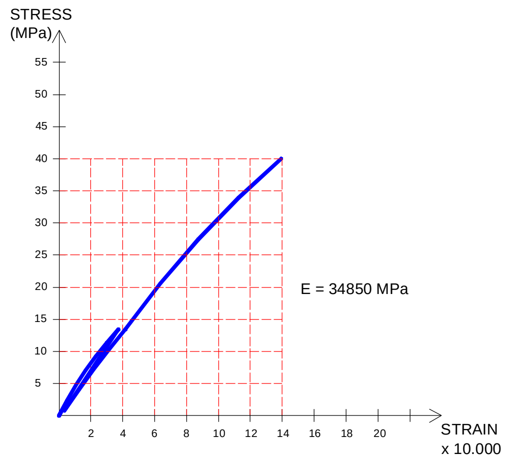

Fig. 9. s - e curve of prefabricated plates concrete

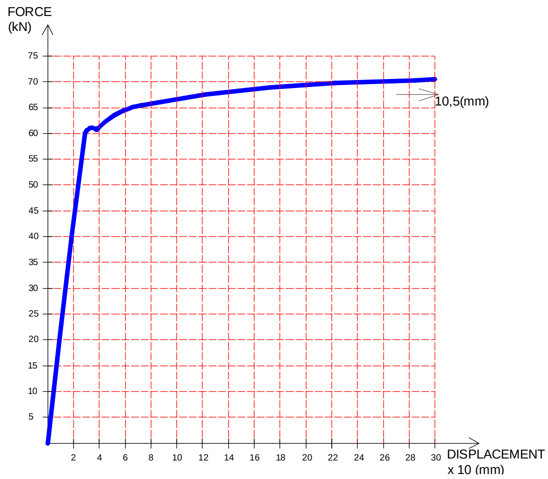

Fig. 10. s - e curve of reinforcement

Tests of characteristics		Average values
Concrete compressive strength (MPa) (evaluated on cubes 15x15x15cm)	Precast plate	47.37
	Monolithic wall	37.55
	Monolithic model	49.63
Modulus of elasticity (GPa) (evaluated on cylinders 15x30cm)	Precast plate	34.85
	Monolithic wall	33.05
Reinforcement strength (MPa) (evaluated on samples 150mm in length)	Yield strength	568
	Tension strength	698

Table 1. Average testing values of concrete and reinforcement samples

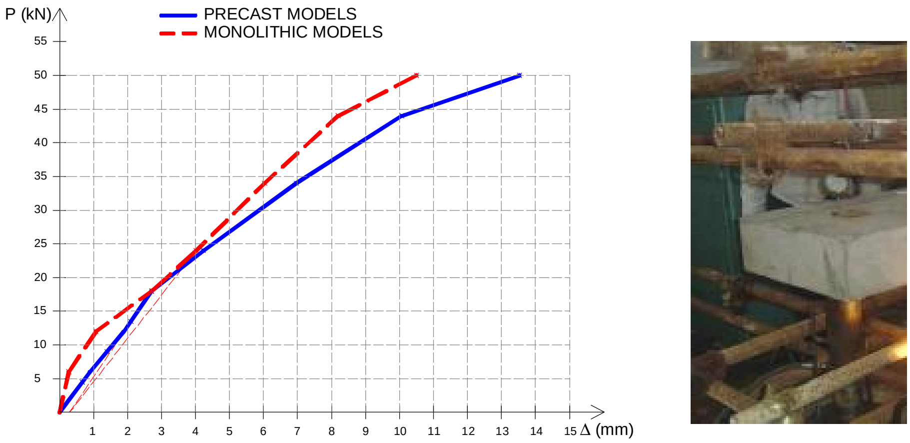

Fig. 11. P-D average curves over applied load

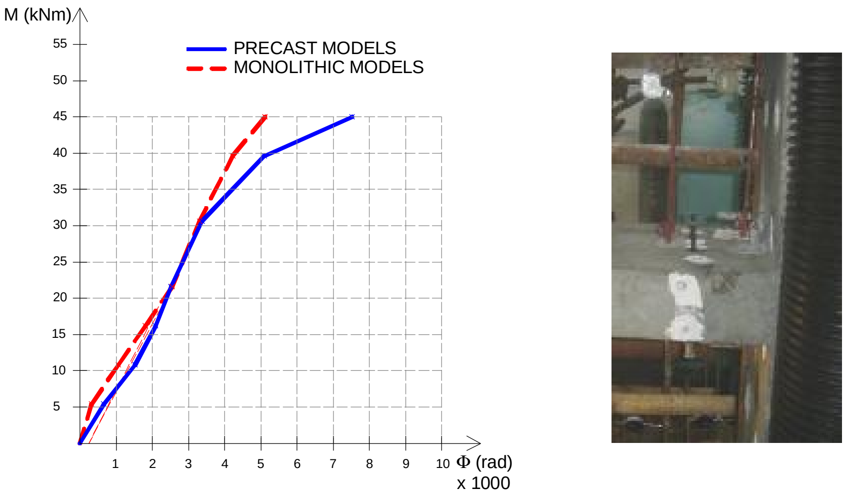

Fig. 12. M-F average curves of plate cross-section close to wall

The results of measured deflection are presented in Fig. 11, Fig. 12 and in Fig. 13. Force-displacement curve determined at measurement place U5, located over applied load is presented in Fig. 11. Cross-section rotation, 20.5 cm from internal wall surface, was measured with deflecto-meters at measurement places U1 and U2. Those measurements were used for obtaining ***M-F*** curve, presented in Fig. 12. Rotation on distance 20.5 cm from wall was measured with deflecto-meters at U3 and U4 (Fig. 13). Deflecto-meters measurements places U6 and U7 were used for control of model fixity and determination of values of joint and plate rotation relatively to wall.

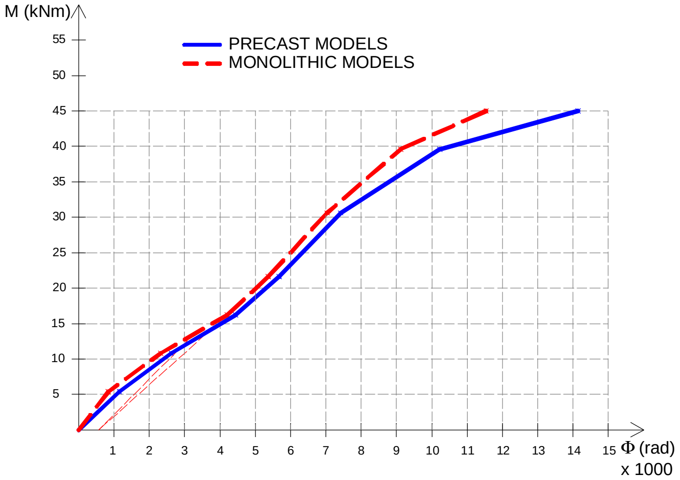

Fig. 13. M-F average curves on distance 20.5 cm from wall

During the experiment, the crack opening mechanism and the state just before failure were observed. Micrometer was used for measuring the width of the crack.

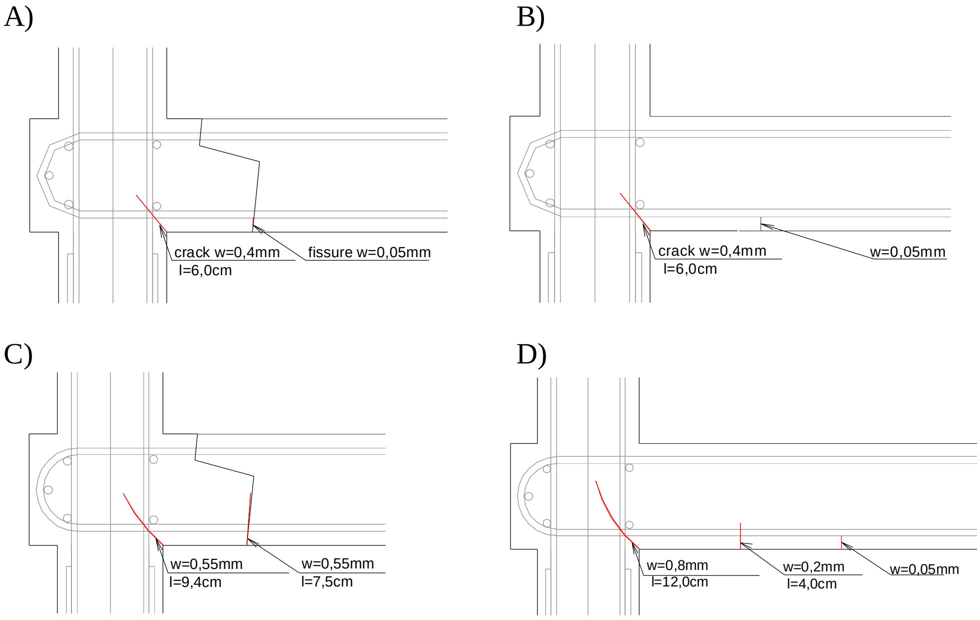

Fig. 14. Crack opening mechanism: A) Precast specimens-opening of 2nd crack; B) Monolithic specimens- opening of 2nd crack; C) Precast specimens-development of cracks; D) Monolithic specimens-development of cracks;

First crack on the monolithic and precast specimens opened at the slab – wall joint with the size of the force of 18-24 kN, i.e. the size of the bending moment ranges from 15 to 19.8 kNm. The angle of the crack was approximately 450. After reaching a length greater than 3cm the direction of the crack was shifted towards the joint, i.e. branch of the crack made the angle with the vertical axis less than 45 degree. When the intensity of the load P ≈ 34.0 kN, i.e. the size of the bending moment M ≈ 28 kNm crack in slab-wall joint, by precast specimens, has the width of 0.4 mm (Fig.14A). This width is for monolithic specimens, at the intensity of the load P ≈ 38.0 kN (Fig.14B.) After the opening of above crack-widths, further cracks opened. At precast specimen the crack opened at the precast connection of the MRCW and PRCS (Fig.14A), while on the monolithic specimen the second crack opened at a distance of 15-16 cm from the wall – slab joint (Fig.14B). With the increase of load, the length and width of the crack at the wall-slab joint is also increasing, changing also the crack pattern inclination towards the vertical axis (Fig. 14C and 14D). After the opening of a second crack, the crack opening mechanism in the precast specimens has the property that the second crack in the precast connection opens more intensely than the crack at the wall-slab joint, so that for the load intensity of P = 40 kN, i.e. for the moment of M = 33.0 kNm, the width of each crack is equal, i.e. 0.55 mm (Fig.14C). In monolithic specimen, with the further increase of load, the next crack opens at a distance of 15cm from the second crack (Fig. 14D).

A) B)

Fig. 15. Failure mechanism of precast specimens: A) Crack pattern; B) Precast specimens by yielding of reinforcement

At the load of P = 48 kN, i.e. the size of the bending moment M = 40.0 kNm crack at the junction of the wall-slab for the precast specimen reaches a maximum value of width w = 0.8 mm and length l = 12cm. At the end, its path is nearly vertical. At the same time cracks in the dislocated precast connection has a width of w = 1.0 mm. The next crack open on the regular grid of 12 to 13cm. Further cracks open following the pattern of the regular grid from 12 to 13cm. At the above load intensity, reinforcement yielding occurs. Cracks at the joint of the wall-slab are still not developed and gradually close with further development of the crack in the precast connection (Fig.15A). Maximum implemented load on the precast specimen is P_{max} = 52 kN, while on the monolithic specimen is P_{max} = 58 kN. Thus the experimentally determined failure load for the monolithic connection is for 11.5% higher than for the prefabricated connection. At these load intensities, reinforcement yielding, structural deterioration and decline of force at hydraulic press occur.

The damage of the model shows the characteristics of ductile fracture. In fact, registered at the fracture site the opening of the cracks have the width w_{max} = 20mm and strain of reinforcement 60‰. This ductility is the result of double reinforcement section, because according to [11], fig. 10.22, pp. 350, can be seen that the increase the percentage of reinforcement in the compressed zone increases the ductility and, for example, for the relationship $\rho'/\rho = 0.75$ is $\Phi u / \Phi y > 16.0$.

On the basis of the experiments in [27] the beam-column joint failure was classified as:

– beam failure,

– joint failure.

Experiments on the models designated as RK1 and RK4, where the joint reinforcement is consistent with that in the specimen presented in this paper that the beam failure occurs at the geometric relation of $h_{beam}/h_{column} \leq 1.25$, while the joint failure occurs at $h_{beam}/h_{column} \geq 1.50$. Crack paths immediately before the failure are presented in [27], p. 657.

The crack-opening mechanism, as identified by experiments on precast and monolithic models, indicate a slab failure instead of a joint failure, which corresponds to the relations given in [27]. Namely, the geometric relation is $h_{slab}/h_{wall} = 16/15 = 1.07$. This conclusion may be confirmed also by comparing the experimentally identified crack paths (Fig. 14 and 15) with Figure 4a in [27].

A similar type of failure can be seen in [24], p.132, type "O". On page 133 of this paper there is a conclusion that the cracks opening mechanism depends on the relation $m_E = E_{b,p}/E_{b,j}$, where $E_{b,p}$ is the elasticity modulus of the concrete of prefabricated elements and $E_{b,j}$ is the elasticity modulus of concrete in the joint. With the identified value of relation m_E of $0.9 \leq m_E \leq 1.12$ the crack opens either in the concrete of slab or in the concrete of joint, instead of the location of failure, i.e. at the joint of the prefabricated and monolithic part of the structure. If $0.9 > m_E > 1.12$, the crack opens at the joint. On the tested precast specimen, the relation m_E is 1.05. Also in [24], p.132, it was noted that in the vast majority of experiments, the first cracks open at the joint.

With the introduction of the relation $\beta = \varepsilon_r/f_\varepsilon$, where ε_r - dilation at the failure, ε_f - dilation at the opening of the first crack, after extensive experimental research in [24] the following possible forms of failure were identified:

– $5 < \beta < 8$ an elastic crack opens with concrete crush

– $8 < \beta < 12$ simultaneous failure

– $\beta > 12$ a plastic type of crack opens, failure of the reinforcement or failure of the reinforcement anchorage.

On the tested precast and monolithic models, the relation between failure dilation and the dilatation at the first crack opening is $\beta \approx 28$.

It is important to emphasize that the testing was done unilaterally by quasi-static load, and therefore should take into account the effects of dynamic loads, which will be the subject of further research.

In order to determine connection yielding relative rotation of precast slab and monolithic wall has been analyzed. Processing of the experimental results is very difficult, due to the fact that sometimes it is impossible to separate relative rotation inside connection from flexible slab rotation. In order to overcome mentioned problem relative rotation is determined, by using the pattern given in papers [13], [14] and [23], as the sum of:

– flexible slab rotation, i.e. joint rotation, along the plastic hinge;

$$\Phi = \frac{M \cdot l_p}{E_c \cdot I_{I(II)}} \tag{1}$$

– rotation of slab-wall contact surface;

$$\Phi = \frac{s_s \cdot l_e}{E_s \cdot d} \tag{2}$$

– rotation by relevant cross-section deformability.

$$\Phi = \frac{M}{A_s \cdot E_s \cdot z_{II} \cdot (d - x)} \tag{3}$$

where: l_P – length of the plastic joint, E_c – concrete elasticity modulus, $I_{I(II)}$ - the moment of inertia of the non-cracked and the cracked cross-section, σ_s – reinforcement stress, E_s - reinforcement elasticity modulus, l_e – anchorage length or length over which a stress distribution along the rebar is uniform, d – effective depth, A_s – reinforcement area, z_{II} – lever arm of internal forces.

Joint stiffness quantification was performed by slab. Moment - relative rotation experimental curves, calculated curves and plate-line with geometrical characteristics and characteristics of materials used in experiments are given in Fig. 16. Geometrical parameters are as follows: b = 50 cm, d = 16 cm, As = As' = 4.52 cm^2. Points on curves, A and B, were determined for slab span 6.15 m, loaded with serviceability load. Point A presents moment of support with assumption of ideal rigid connection. Point B presents rotation on support with assumption of joints connection. Points C and D were determined for slab span 6.15m, loaded with ultimate load. Points E1 and E2 are on intersection of slab-line and experimental curves for precast and monolithic models. As can be seen from the Fig. 16 curves intersect slab-line, and that mean that stiffness of connection can be taken in to consideration in the calculation procedure. In case of connection failure before intersection of slab-line, connection may be considered as joints.

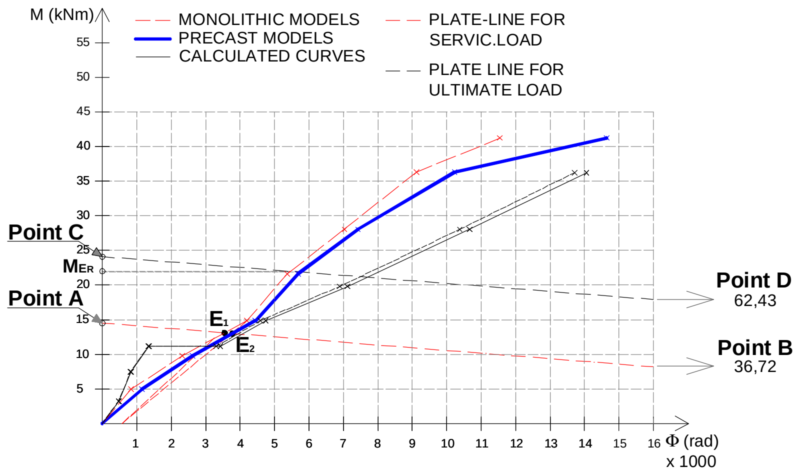

Fig. 16. Comparative curves of moment - relative rotation

Analyzing presented curves it may be concluded that monolithic connection can be considered semi-rigid. Performed experiments have shown obtained that monolithic and precast models had similar behavior up to load of $1.33P_{service}$. This mean that in the mentioned range of load intensity tested precast connection can be considered monolithic.

Factor of connection yielding exposed to serviceability load was obtained according to recommendations in [10], [12], [13] and [14]. Results are presented in Table 2.

After	Monolithic model	Precast model
[10]	$\alpha/\alpha+2 = 0{,}91$	$\alpha/\alpha+2 = 0{,}89$
[14]	$\gamma = 0{,}89$	$\gamma = 0{,}87$

Table 2. Average degree of connection yielding

Researches performed on monolithic models show that even monolithic structure can also be considered semi-rigid. Degradation of investigated precast connection start at load intensity greater then serviceability load, and it is insignificant up to ultimate load. Bearing capacity of precast connection is proved to be at the same value as monolithic. Yielding of precast connection is in increase mode relatively to monolithic connection for the range over 1.33P. Based on experimental tests on 3 precast and 3 monolithic samples it was also proved that designed precast connection enable dislocation of joints opening outside of wall and dissipation of seismic energy with similar bearing capacity as monolithic.

3. NUMERICAL ANALYSIS

Modeling the mechanism of behavior of reinforced concrete (RC) elements and connections is a complex mathematical problem. In fact, reinforced concrete is a composite, highly heterogeneous material with its influential parameters having a stochastic nature, which can only be treated through reliability analysis based on theory of probability. Theoretical considerations of modeling the behavior of RC elements are constrained to certain simplifications and the introduction of a series of assumptions due to formulate a mathematical model of acceptable complexity and accuracy. Experimental studies are the basis for the development of theoretical settings in the civil engineering. Mechanisms of behavior of reinforced concrete monolithic elements and joints are more discussed than those in prefabricated structures. Moreover, in the proposed mathematical models, the adhesion properties of concrete and reinforcement, the crack initiating mechanisms and the condition immediately before the fracture were treated. The modern approach to the analysis of structures is reduced primarily to the concept of ductile structures, which was presented in [36]. Therefore, current norms for the RC elements and structures are primarily related to provisions for achieving sufficient ductility, which especially refers to structures loaded with horizontal variable load (wind and seismic loads).

When calculating monolithic RC structures, an integral structural work is assumed, with stiffly connected structural elements and the requirement that the failure of structural elements occurs before the failure of their connection. In Europe, such calculation concept is used also in the numerical modeling of connections of prefabricated RC elements. During analysis connections are treated as absolutely rigid or joints. To describe a realistic response of prefabricated structures it is necessary to analyze the stiffness of connections. Here is the analysis of connections with the presented experimental results. The numerical model and approximate engineering model (Strut and Tie Model) are given. At the end reliability analysis of experimental results and the results of numerical analysis are presented.

3.1. Numerical models

During the analysis of the experimental results, the definition of connection stiffness presented in papers [19], was used, starting with the base formula,

$$K_{jr} = M / j \tag{4}$$

According to [12], the connection stiffness or the average module of deformability was identified based on the experimental curves presented in Fig.16. With the identified parameters of connection deformability, it is possible to formulate conditions for the compatibility of Force Method or the conditions for the balance of the Displacement method. The application of the Force Method was described in details in paper [18]. This paper presents the application of the Displacement Method.

The analysis of the discrete model consists of the element analysis and the system analysis. The element analysis starts from basic equations of the theory of bar, that is the theory of slab, and establishing the relation between generalized forces and generalized displacements in joints at the end of the elements.

$$\{P\} = [K]\{d\} \tag{5}$$

In order to analyse semi-rigid connections the modification of stiffness matrix is necessary,

$$\lfloor K^* \rfloor = [K] \cdot [C] \tag{6}$$

throughout the implementation of yield system with semi-rigid joints matrix [C] in the equation,

$$[C] = \begin{bmatrix} 1 & 0 & 0 & 0 \\ -\frac{k_{21}^*}{S_f} & 1-\frac{k_{22}^*}{S_f} & -\frac{k_{23}^*}{S_f} & -\frac{k_{24}^*}{S_f} \\ 0 & 0 & 1 & 0 \\ -\frac{k_{41}^*}{S_f} & -\frac{k_{42}^*}{S_f} & -\frac{k_{43}^*}{S_f} & 1-\frac{k_{44}^*}{S_f} \end{bmatrix}, S_f = \frac{M}{f} \text{ experimental connection stiffness} \tag{7}$$

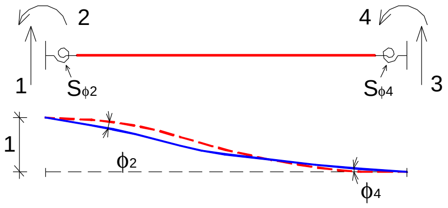

Fig.17. Geometrical-statical meaning of first vertical line of yielding matrix

Through matrix transformation equation (6) can be transformed in equation:

$$\left[K^*\right]=[K]\cdot\left([I]-\left[S_f\right]\left[K^*\right]\right) \tag{8}$$

Modified matrix of interpolation functions is,

$$\left\{f^*\right\}^T=\{f\}^T\cdot[C] \tag{9}$$

so that vector of equivalent loads is,

$$\left\{P_e^*\right\}=\int_0^L\left\{f^*\right\}^T\cdot p(x)dx \tag{10}$$

Working diagrams shown in Fig. 18 are used for modeling the connection zone of the monolithic RC wall and a prefabricated RC slab. Yielding of the tested prefabricated connection compared to the monolithic connection is defined through the relative rotation, as shown in Fig. 19.

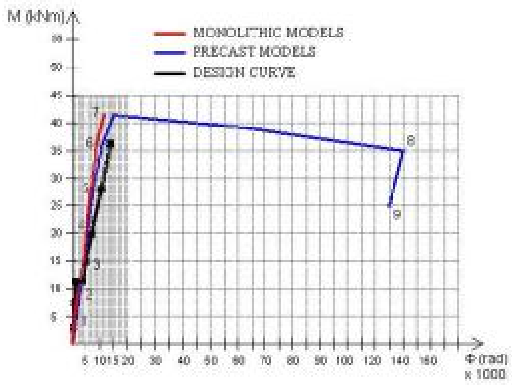

Figure 18. Comparative moment-relative rotation curves with extension of the experimental curve for the prefabricated model

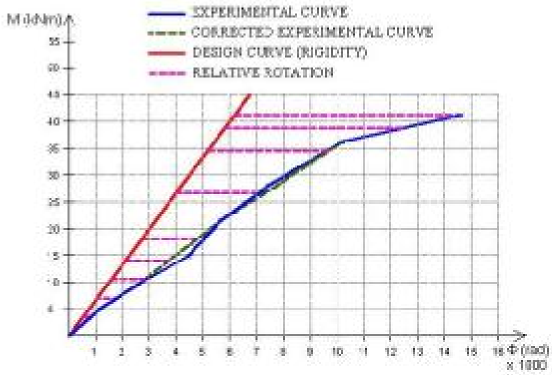

Figure 19. Values of relative shift of the prefabricated connection

For modeling the structural failure, the concept of residual stiffness, presented in [29], is introduced. For the prefabricated model residual stiffness is defined as shown in Figure 20.

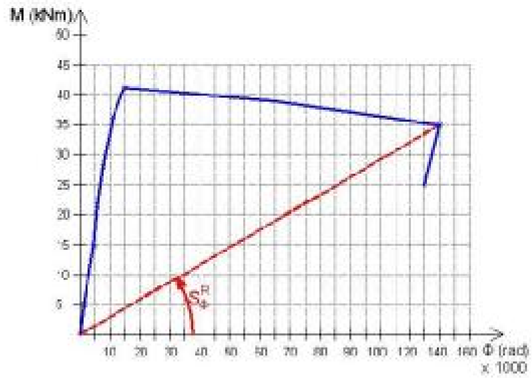

Figure 20. Residual stiffness of the prefabricated connection - S_{Φ}^{R}

The numerical analysis of real behaviour of PRCS and MRCW connection was done on the models where slab and wall were modelled with BEAM and SHELL elements, whereas the connection behaviour is modelled with PLASTIC LINK elements. Testing of proposed models was performed during modelling, and it was proved that finite elements model with side ratio 1:1:1 gave the best results. The model with finite elements without modelling of connection work mechanism which is generally applied in design practice was used as a starting comparative model. Afterwards, models with slab-wall connections modelled with one plastic link element (monolithic yielding model) were tested (Fig.21a). Plastic link element was modelled with the application of experimentally obtained M-ϕ curve for monolithic model. The precast model was made in the same way. The behaviour of precast models in the failure area was modelled with two plastic connection elements being serially joined (Fig.21b). First linked element was modelled by monolithic yielding models, whereas work mechanism of the second link element was modelled by applying the moment – relative rotation curve of the precast model.

a) b)

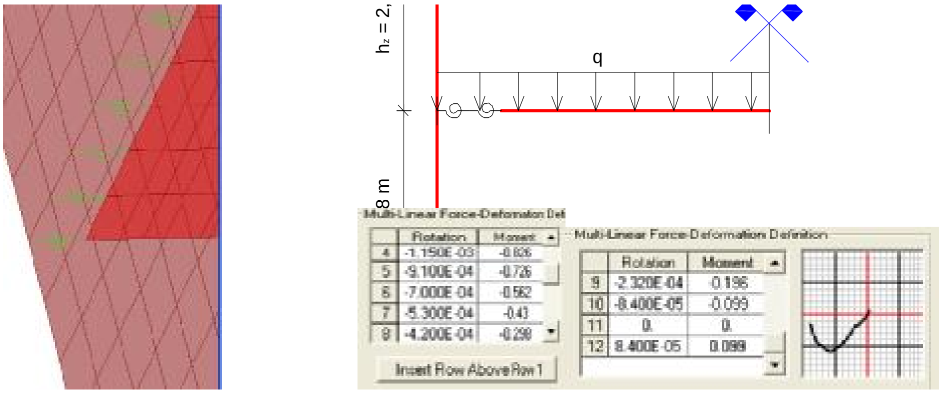

Fig. 21. Modelling of slab-wall connection: (a) Monolithic yielding model; (b) Precast model

Comparative support moment – load curves for proposed numerical models are given in Fig.22.

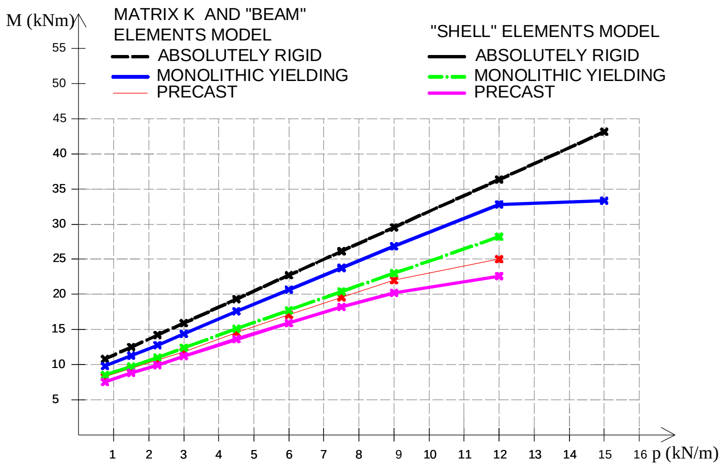

Fig.22. Calculation curves support moment – load (M-p) for analyzed numerical models

Values of the yielding ration for precast connection in relation to monolithic connection are presented in table 3.

Load	**Numerically obtained values**	**Experimentally obtained values**
Serviceability load	0,92	0,907-0,913
Ultimate load	0,90	0,897-0,903

Table 3. Experimentally and numerically obtained values of connection yielding - g

The previously performed numerical analysis and comparison with the analysis of experimental results show good coincidence of results and sufficient accuracy of presented mathematical model for the calculation of researched connection. The deviation of calculation values in relation to the experimental, for serviceability load, is 0.77 – 1.43%, and for ultimate load is 0.33%.

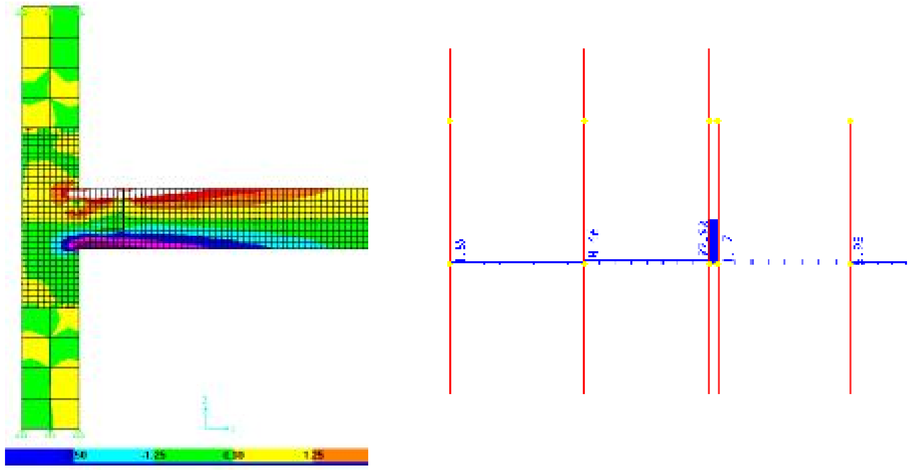

Fig.23. FEM model:(a) stress distribution,(b) stress concentration in reinforcement-crack location

3.2. Approximative models – Strut & Tie models

During the experimental research of monolithic and precast specimens, mechanism of crack opening was registered. The mechanism of crack opening, registered during the testing of precast specimens, was modelled with finite elements (Fig.23).

FEM model was used as a basis for making Strut and Tie model. The analysis started from the preliminary Strut and Tie model (STM) for the monolithic specimen (Fig.24).

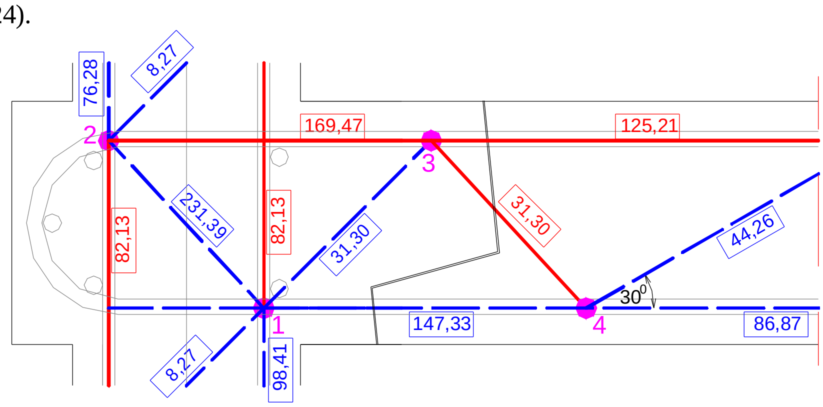

Fig.24. Preliminary STM for monolithic specimen

Naturally, in this model the effects of local stress and behaviour mechanism of the dislocated precast connection were not covered. So, the preliminary Strut and Tie model served as a comparative model. In order to define the Strut and Tie model, with sufficient accuracy for design practice, a detailed analysis of stress inside the precast connection was performed through FEM model (Fig.25). The proposal of Strut and Tie model for researched precast specimen is given in Fig. 26.

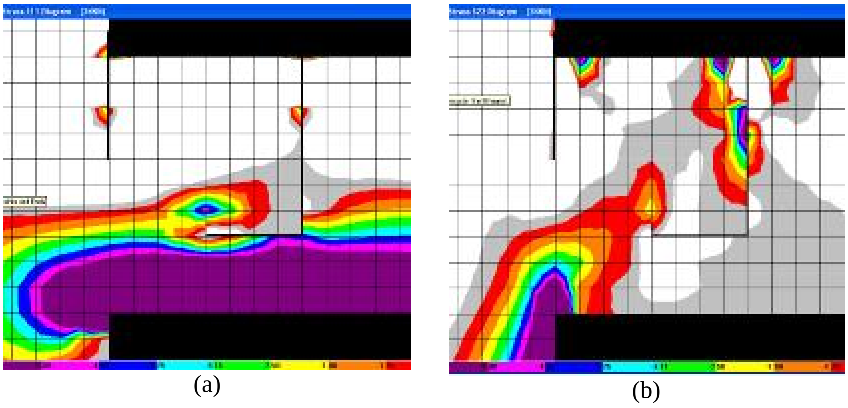

Fig.25. FEM model, principal stresses: (a) s_1, (b) s_2

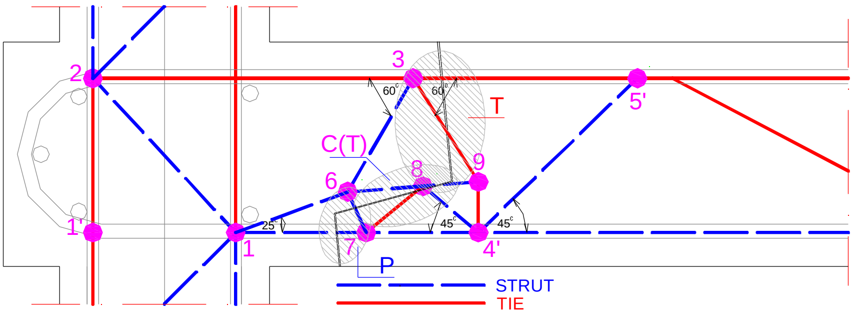

Fig.26. Proposal of STM for Precast specimens

In the connection area, three different areas of stress trajectory departure can be separated regarding the deviation in comparison to the monolithic model, marked in Fig. 26 as areas C, T and C (T). Area T is the area of crack opening in dislocated precast connection. In that area there is a stress concentration in reinforcement, which is distributed in the surrounding non-cracked concrete. In the vicinity of upper turning point of precast connection tension area passes in the area where the connection is partially stressed with tension and partially with compression (area C (T)). Tensioned bars were presented with tie 3-5'. Shear in connection, that is the transfer of shear load, was modelled with tie 3-9. Transferred area C (T) was modelled with struts 8-9, 4'-8 and tie 7-8. The concentration of compression stress near the lower turning point of precast connection, area C, was modelled with strut 6-7. The proposed Strut and Tie model represents the approximation of complex stress state in the vicinity of precast connection with resultant forces of stress fields. The accuracy of the model depends on the number of struts and ties, and the level of model detailing. The proposed model describes basic properties of the designed precast connection work mechanism. Axial loads in struts and ties of the proposed model are presented in Fig. 27. The effect of strength connection yielding was obtained through the introduction of the effects of crack opening in dislocated precast connection, with respect to the stress trajectory obtained by applying FEM model. Therefore, the force in tensioned bars (tie 2-3) is smaller by 7% in relation to the monolithic model. Also, compression in joint 1 is smaller by 16%. The mentioned values are approximate and on safe side, and can be applied for preliminary numerical analysis of work mechanism parameter of the researched precast connection.

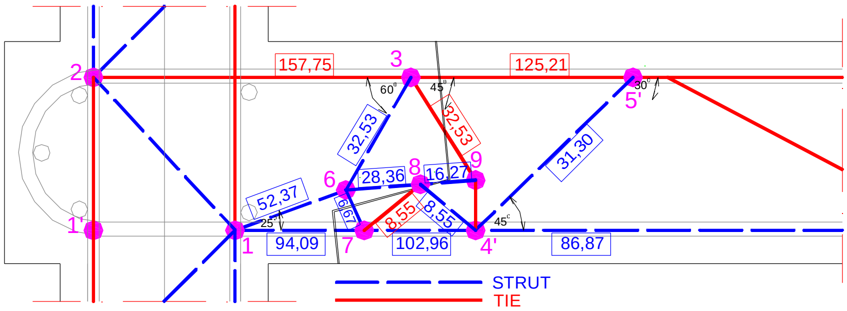

Fig.27. Axial loads in compressed bars and ties of proposed STM

3.3. Reliability analysis

To verify the presented results is necessary reliability analysis of the obtained characteristic values with the aim of defining the safety degree (factor) of analyzed prefabricated building systems. Reliability analysis of yielding degree of researched MRCW and PRCS connection, defined by expression given in the paper [8],

$$a = \frac{S_f \cdot L_{pl.}}{B_{pl.}} = \frac{S_f \cdot L_{pl.}}{E \cdot I_{pl.}} \tag{11}$$

was performed. The analysis is done assuming a normal probability density function for the influential parameters. The span of plate was adopted L_{pl} =6,15m, and coefficient of distribution distortion β_1=0. Therefore, the yielding degree as a function of three random variables $a = f\left(S_f, E, I\right)$ was analyzed.

Because the testing performed on the 3 monolithic and 3 precast specimens, that was not enough data for statistical data processing and reliable determination of scattering results parameters. Therefore, the obtained results of experimental research compared with results of so far performed research and recommendations presented in [5], [16], [25], [33] and [34].

On the basis of extensive experiments Ellingwood 1980's recommended a representative coefficient of variations, for concrete elements exposed to bending, from 8 to 9.5%. The same author has defined for the beam-column connection coefficient of variation of 10%. Mehlhorn in [33] presented extensive studies of smooth shear connectors, where for the 176 experiments obtained coefficient of variation 16%. The same results were obtained Hansen, Olesen, Fauchart and Cortini. Pommeret got a coefficient of variation 12-15%, Laing 17 ÷18 %, Pume 13 %. The paper [24] presented a statistical analysis of the testing results with 95 models, for which the obtained coefficient of variation of 12%.

Based on these previous data and results of experiments, analysis for the coefficient of variation from 10 to 25% was performed. The experiment obtained mean value $S_f = 3507{,}94\ kNm/rad$ was adopted as the expected value.

By production of precast elements, size of permitted deviation depends on the type of structures, production procedure and erection procedure, and often is within the limits of 3 to 10mm. For the purposes of the definition of reliability in this paper, tolerance for the width of panel models was adopted ±10mm, ie., $b_{pl.} = 500 \pm 10\ mm$, and for a plate thickness ±10mm, ie., $d_{pl.} = 160 \pm 3\ mm$.

Assuming standardized normal distribution and typical 5%-fractil values of permitted deviations, coefficients of variation are:

– for width of the model CV = 1,22 % (adopted 1,5%)

– for plate thickness CV = 1,13 % (adopted 1,5%)

Difference of value of modulus of elasticity of precast and monolithic specimen concrete determined with experiment is 4.78%. This difference is reflected in different deformation characteristics. However, conducted experimental and numerical studies have shown that defined size of differences had no significant effect on the comparative analysis of the yielding degree of monolithic and precast models. Since the expressions for determining the modulus of elasticity in the function of compressive strength of concrete, here is the lack of sufficient data to take the recommended values of coefficients of variations, based on extensive research of individual authors. Ellingwood in 1980. proposed for reinforced concrete elements loaded in bending coefficient of variation of 14% [25]. Bljuger during the analysis of compound walls adopted coefficient of variation CV = 12.8% [5]. Based on the results of research that is done Entroy 1960., Murdock 1953., Rusch 1969., Mirza 1979., Melchers is presented in Table 8.7. [34] coefficient of variation or standard deviation ranging from 2.8 MPa for the excellent concrete to 5.6 MPa for the bad concrete. These are the results of compressive strength fc> 28 MPa. For compressive strength with a nominal value 30 MPa the coefficient of variation is CV = 9.33% for the excellent concrete, and CV = 18.66% for the bad concrete. For reliability analysis in this paper have been adopted the coefficient of variation as for connection stiffness from 10 to 25%. Expected value is adopted experimental mean value $E_c = 34975\ MN/m^2$.

For solving random variable systems have been developed 3 methods of solution:

1. Accurate method, which requires as input known probability distribution functions for the variable. In the process of resolving apply numerical integration and Monte Carlo methods. This procedure requires extensive computer analysis.

2. FOSM (First order second moment method), where in the process of solving the use Taylor's series.

3. PEM (Point estimate method) that is most used in the analysis because it allows a gradual development of solutions.

Analytical integration is possible only in special cases. Numerical solution of the problem is simpler, but when it is integral more than two random variables numerical integration can not in all cases provide sufficient accurate solution. With the increasing number of variables significantly increasing the calculation requirements, because in these cases integration areas are complex geometric figure, n-dimensional sphere.

Therefore, solutions based on numerical integration for the specific methods were developed, such as Monte Carlo simulation, which can be applied only to developed systematic methods of numerical sampling of basic variables X, or Markov chain simulation method presented in the paper [3]. Application of Monte Carlo simulation methods for RC structures are presented in the papers [4] and [22].

Application of Taylor's series requests in formulating and solving the derivative, which is for multi-variable problems demanding task, especially when the function is implicit in the form of the default curve, a graph or as a FEM solution. Examples of simplified reliability model are presented in the papers [9], [25] and [31].

For reliability analysis in this paper was used PEM method developed by Rosenblueth eighties last century. Method is based on the analogy between probability distributions and distributed vertical load on the horizontal solid beam. Expected value corresponds to the position of action that balances the load force (unit value) or the center of gravity of the load. Rosenblueth proposed that the parameters of the expected value and standard deviation determine according to the analogy with the simple beam (Fig.28).

Reactions p- and p + are two points of equilibrium of distribution function f(x). Applying the theory of reliability can set the following equation:

The condition of equilibrium: $p_+ + p_- = 1$ (12)

Expected value: $p_+ x_+ + p_- x_- = E[f(x)] = \bar{x}$ (13)

The measure of dispersion $p_+(x_+ - \bar{x})^2 + p_-(x_- - \bar{x})^2 = s\,[f(x)]^2 = s^{\,2}[x]$ (14)

Asymmetry of distribution: $p_+(x_+ - \bar{x})^3 + p_-(x_- - \bar{x})^3 = b\,(1)s^{\,3}[x]$ (15)

Solutions of equations (12) to (15) are:

$$p_+ = \frac{1}{2}\left[1 \pm \sqrt{1 - \frac{1}{1 + \left[\frac{b\,(1)}{2}\right]^2}}\right];\ p_- = 1 - p_+;\ x_+ = \bar{x} + s\,[x]\sqrt{\frac{p_-}{p_+}};$$

$$x_- = \bar{x} - s\,[x]\sqrt{\frac{p_+}{p_-}} \qquad (16)$$

On the basis of certain points of random variable x and the functional dependence x and y get the value of the function y(x), y + and y-, using the expression,

$$E\left[y^M\right] = p_- y_-^M + p_+ y_+^M \qquad (17)$$

where M corresponds to the number of known moments for random variable x.

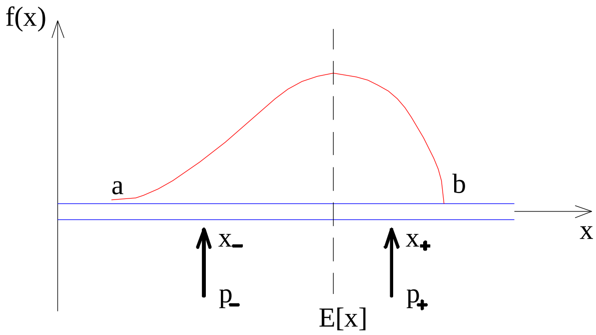

Fig.28. Distribution of probability – solid beam analogy

For the function of three random variable $y = y(x_1, x_2, x_3)$ is valid:

$$y_{\pm\pm\pm} = y(\overline{x_1} \pm s[x_1], \overline{x_2} \pm s[x_2], \overline{x_3} \pm s[x_3]) \tag{18}$$

Weight coefficients p are:

$$p_{+++} = p_{---} = \frac{1}{2^3}(1 + r_{12} + r_{23} + r_{31});\; p_{++-} = p_{--+} = \frac{1}{2^3}(1 + r_{12} - r_{23} - r_{31})$$

$$p_{+-+} = p_{-+-} = \frac{1}{2^3}(1 - r_{12} - r_{23} + r_{31});\; p_{+--} = p_{-++} = \frac{1}{2^3}(1 - r_{12} + r_{23} - r_{31}) \tag{19}$$

Expected value is:

$$E[y^M] = p_{+++}y^M_{+++} + p_{++-}y^M_{++-} + \ldots + p_{---}y^M_{---} \tag{20}$$

Applying PEM method analyzed several possible statistical cases. Details were given in the paper [54]. For the cases degree of the researched connection yielding. Results of the analysis are given in Table 4.

Statictical cases	Moment of inertia I (m^4)		Modulus of elasticity E_c (MN/m^2)		Connection stiffness S_ϕ (kNm/rad)		Degree of yielding γ after [15]	
	E[I]	V(I)%	$E[E_c]$	$V(E_c)$%	$E[S_\phi]$	$V(S_\phi)$%	$V(\gamma)$%	$\gamma_{0,05}$
Case (a)	0,00003392	6,0	34975	10,0	3507,94	10,0	7,68	0,888
Case (b)	0,00003392	6,0	34975	15,0	3507,94	15,0	11,31	0,878
Case (c)	0,00003392	6,0	34975	20,0	3507,94	25,0	26,17	0,839
Case (d)	0,00003392	6,0	34975	25,0	3507,94	25,0	20,61	0,860
Case (e)	0,00003392	6,0	34975	10,0	3507,94	25,0	15,90	0,869
Case (f)	0,00003392	6,0	34975	25,0	3507,94	10,0	21,91	0,860

Table 4. Analyzed statistical cases

For maximum expected variation of 27%, ie., maximum value from Table 4 moment on support is, $E(M_{osl.,0.05}) = 0{,}835 \cdot \overline{M}$, with variation $V(M_{osl.,0.05}) = 7{,}22\%$.

4. DISCUSSION AND CONCLUSIONS

Researches performed on monolithic slab-wall models show that even monolithic structure can also be considered semi-rigid. Based on experimental tests on 3 precast and 3 monolithic samples it was proved that designed precast connection enable dislocation of joints opening outside of wall and dissipation of seismic energy with similar bearing capacity as monolithic. Degradation of investigated precast connection start at load intensity greater then serviceability load, and it is insignificant up to ultimate load. Yielding of connection rigidity increase in the range over 1.33 P.

Application of Displacement Method, with modification of stiffness matrix through the introduction of yielding of joints, can be defined by a mathematical model that describes the mechanism of structures made of monolithic walls and precast slabs. The paper presents a modified stiffness matrix obtained in the experiment by introducing stiffness of MRCW-PRCS joint. By the application of the proposed mathematical model, the degree of yielding of the investigated precast connection was 17.95 - 20%. Relative yielding of the precast joint compared to the monolithic is up to 10%.

For the design purpose, the proposed Strut and Tie Model, which analyzed the yielding of the joint, can be used in practice. The accuracy of calculation depends on the complexity of the model and is achieved through refinements of Strut & Tie model.

Analysis of civil structures includes multivariable problems, where almost all stochastic variables. Reliabaility analysis is done for experiment determined the mean and variation, as well as variations recommended by authors who have dealt with issues under consideration. For the analysis adopted a normal distribution of random variable. Correlation coefficients between individual random variable adopted for the experiment conditions. Due to an insufficient number of tested samples for implementation of reliable statistical procedure, the correlation coefficients were reduced to the expected minimum values. Adopted is to reduce correlation coefficients r_{I,S_f} *and* r_{E_C,S_f} of 20%. It is considered that random variables have a strong correlation since the correlation coefficient is $\rho \geq 0,50$. Since the correlation between moment of inertia and connection stiffness, and connection stiffness and modulus of elasticity of concrete strong for researched precast connection, in the discussions were considered correlation coefficients reduced to a value that is at the lower limit of good correlation.

Accordingly, the correlation coefficients were adopted:

$r_{I,E_C} = 0,4;\ r_{I,S_f} = 0,72;\ r_{E_C,S_f} = 0,48$

Variation of degree of yielding and moment on support are:

$V(a) = 29,03\%$

$V(M_{osl.}) = 8\% < V(M_{osl.,\mathrm{exp.}}) = 10\%$

Reliability index of researched precast connection is $\beta = 5,98$.

Variation of moment capacity, for the expected minimum correlation coefficient, is less than the adopted for the experiment conditions. Based on performed analysis is recommended for defining the reliability of the precast connection, which behave similarly as researched precast connections, use the following minimum correlation coefficient:

– correlation between the moment of inertia and modulus of elasticity of concrete – $\rho_{I,Ec} = 0,40$,

– correlation between the moment of inertia and connection stiffness $\rho_{I,S\phi} = 0,72$,

– correlation between the modulus of elasticity of concrete and connection stiffness $\rho_{Ec,\,S\phi} = 0{,}48$,

– correlation between width and thickness of the element $\rho_{b,\,h} = 0$.

This recommendation applies to the analysis of the reliability of structures that are not implemented or are in small scale experimental research was conducted. With large scale experimental research recommendations may be conservative.

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