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FACULTY OF TECHNICAL SCIENCE
Institute for Civil Engineering

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iNDiS 2006

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10th National and 4th International scientific meeting

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R. Folić, V. Radonjanin, M. Trivunić

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INDIS 2006

The time we live in sets new tasks to building construction. Its wider context incudes the problems of planning, project designing, building and buiding restoration, all of which are the subject of this convention. Such a wide area of activity requires the involvement of a variety of experts: space planners, urbanists, architects and civil engineers of all profiles, plan designers, building contractors, executors of installation and finishing works in civil engineering, but also of other professions whose activity is related to architecure, civil engineering and built environment, such as sociologists, economists and others.

In contrast to the first conference held in 1976 on the topic „Industrial construction of apartments"and the subsequently held conferences: „Industrialization in civil engineering", the subject was gradually broadened, so that the last two meetings and this one also should have this form. As a result, there are more papers presented at this meeting compared to the conference held in past.

It is our pleasure that a number of members of the Internacional Scientific Committee have taken an active part during the preparation of the Conference and wrote papers that are published in these proceedings. These, as well as some works of authors, contain a variety of ideas and results of experimental and theoretical research, which became the basis for formulating adequate calculation patterns or models of structure behavior under various influences, as well as models used in other filds of civil engineering and enviroment protection. We would like to express our sincere gratitude for their contibution.

Following the "INDIS 2006" Conference two books of proceedings will be published, one of which is in Serbian and the other in English, which shows our desire to present our results and activities to the world and the results of our coleagues abroad to our professional public. We are delighted that these meetings are becoming our connection to the world, i.e.increasingly international.

Novi Sad, November 2006. godine *Authors*

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FOURTH INTERNATIONAL
SCIENTIFIC MEETING

iNDiS 2006

NOVI SAD, NOVEMBER 22-24,2006.

John Ermopoulos[1]

OLYMPIC GAMES 2004 IN ATHENS

Summary: *In the frame of the Olympic Games 2004 a lot of new structures (sport halls, buildings etc) were planed and constructed in Greece and especially in the area of Athens city. Most of them were designed and executed by Greek architects, engineers, contractors and manufacturers. During that period there was an "explosion" in the use of steel in Greece, since almost all the new structures were designed on the basis of this excellent material. In this presentation some of these structures are described in brief.*
Key words*: Olympic Games, Steel Structures, Eurocodes.*

OLIMPIJSKE IGRE U ATINI 2004

Rezime: *U okviru olimpijskih igara 2004 godine veliki broj novih objekata (sportskih hala i zgrada, itd.) je projektovan i izveden u Grčkoj, a pogotovo u okolini Atine. Većina objekta je projektovana i sagrađena od strane grčkih arhitekat, inženjera, izvođača i proizvođača materijala. Tokom tog perioda desila se"eksplozija" u upotrebi čelika u Grčkoj, budući da su skoro sve nove konstrukcije bile projektovane na bazi ovog odličnog materijala. U ovom radu neki od ovih objekata su ukratko opisani.*
Ključne reči*: Olimpijske igre,čelične konstrukcije,Evrokodovi.*

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1. INTRODUCTION

The Olympic Games 2004 in Greece belong now to the past and we are in the middle of the distance to reach the year 2008, when the Beijing (China) will become the center of the world during the next Olympic Games. Looking back and according to the final results, we could allege that Greeks have succeeded in that huge, complicated and very difficult universal event. Regarding the new Olympic structures through the whole country that were necessary to be built, some of them (more than ten) were erected in the most important area for the Olympic games, namely the Olympic Athletic Center of Athens (OACA), where the opening ceremony was taken place. Responsible for the aesthetic arrangement and upgrade of this area, was the Spanish architect-engineer Santiago Calatrava, who was also the designer in some of the structures included in this area (e.g. Roof of the Olympic Stadium, Velodrome, Agora and Nations' Wall). In the following, a brief description of some Olympic structures included in OACA area is presented.

The author of this paper, as a technical consultant of the Greek Ministry of Culture and Development, he was responsible together with his team, for the control of the static analyses of the Calatrava's structures (i.e. application of codes, actions, combination of actions, analysis and design etc).

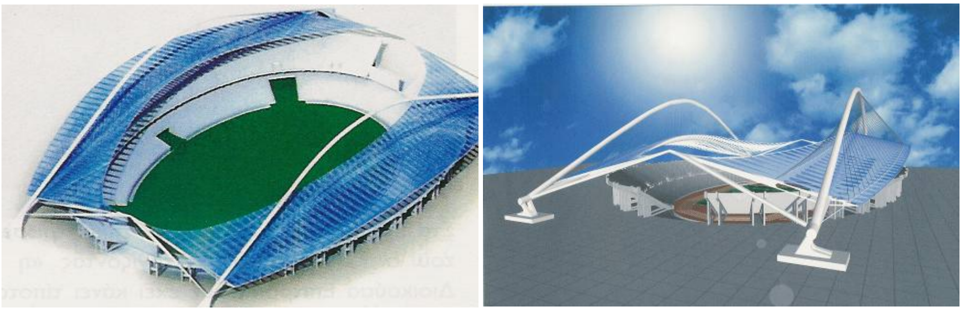
2. ROOF OF THE OLYMPIC STADIUM

The roof of the **Olympic Stadium** (architect S. Calatrava) was constructed entirely from steel, with cladding from polycarbonic sheets. The total steel weight of the superstructure is 18000 tons and the structure has four points of support. The space structure is composed by two pairs of arches which are connected at their support points and are lying on vertical plans. A series of transverse beams keeping the cladding system are supported on the lower arches, while prestressed vertical and inclined cables connected to the upper arches, suspend the lower arches and the transverse beams. The longitudinal distance between the supports of the arches is 305 m while the height of the upper arches is 70 m. The diameters of the arches are 3,20 m (upper) and 3,60 m (lower), with a thickness between 60 and 100 mm. The foundation in each support is realized through a group of piles (about 40, with diameter 1,50 m and height 25-30 m). Because of the parallel work in the interior of the stadium, the two subparts of the roof (i.e. each pair of arches with the cladding) were erected in a distance from the stadium (about 70 m) and then they moved to their final position by sliding with a speed of 3 to 5 m/hour.

Figure 1. The area of OACA in Athens

Figure 2. The opening ceremony

The codes used for the analysis of the roof were the Eurocodes (1, 3, 7 and 8) together with the national codes regarding earthquake. The loading taken into account was combination of the self weight, snow, wind, temperature, earthquake, failure of cables etc. Wind tunnel tests were performed in order to get more realistic information about the aerodynamic response of the structure. In Figures 1 to 16 some details of this roof are presented.



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3. VELODROME

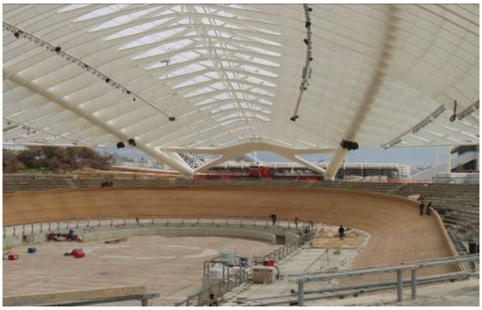
The roof of the **Velodrome** (architect S. Calatrava) was constructed entirely from steel with a total weight of 4500 tons. The cladding consists of white steel sheets externally and timber internally except for a central narrow strip of the roof which has been covered by blue polycarbonic sheets.

Figure 17. General view of the site of velodrome and the temporary towers in 135 m distance


Figures 18 and 19. The Velodrome during the erection

The roof is supported by two pairs of inclined arches connected by welding at their ends. The longitudinal span between the supports is 145 m, the maximum width of the roof is 106 m and the height is 46 m. The main beams of the roof are placed transversally and supported by the lower arches (tubes with diameter 1,80 m and thickness 32 to 54 mm), while prestressed cables (diameter 50 mm) are also used to suspend the roof from the upper arches (tubes with diameter 1,30 m and thickness 60 mm) and to connect the arches. Due to the parallel works in the wooden area of velodrome, the whole roof was erected in a distance of about 135 m from its final position and then it was moved by sliding (with a speed of 10 m/hour). The same codes, materials and loading conditions as in the Olympic Stadium were also applied. In Figures 17 to 25 the various steps of the erection procedure are presented.


Figures 20 and 21. The roof of Velodrome before and after sliding




Figures 22 and 23. Internal view of the velodrome and the sliding equipment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4. AGORA

The **AGORA** is a multi-use complex and consists of fixed parabolic arches (I cross section steel beams) with two different heights, i.e. 19 m and 22 m, in a distance between them 5 m. The span of the arches is 26 m and the whole length of the curved AGORA is approximately 450 m. The arches are connected by inclined bars (RHS and angles) in order to provide partial shading in the covered space and also to secure the stability of the structure, see Figures 26 to 31.


Figures 26 and 27. The AGORA during the erection


Figures 28 and 29. The AGORA during the erection


Figures 30 and 31. The AGORA

5. NATIONS' WALL

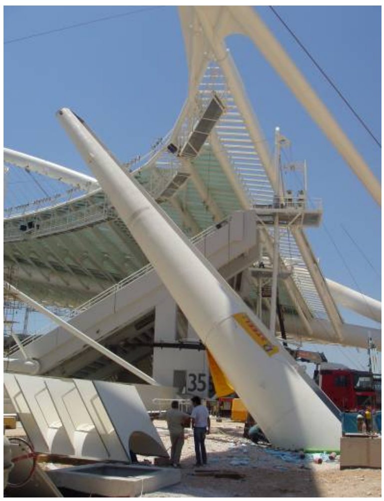
The **Nations' wall** is a 260 m structure consisted of eleven inclined tapered columns with a height of 15 m and a horizontal boxed shape continuous beam supported at their top (1,75x1,00 m). On this beam 960 vertical hollow section bars with a length of 20 m are connected through a hinge. By utilizing a mechanism lying on the upper part of the beam these bars can be rotated in a sinusoidal form, see Figures 32 to 37. The foundation consists of separated pile caps each supported on four piles.


Figures 32 and 33. The Nations' Wall during the erection


Figures 34 and 35. The Nations' Wall, the mechanism and an overview


Figures 36 and 37. The Nations' Wall

In the following figures most the rest Olympic structures are presented without comments due to the lack of space.

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Figures 40 and 41. The Torch

 

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Figures 44 and 45. The Entrances of OACA


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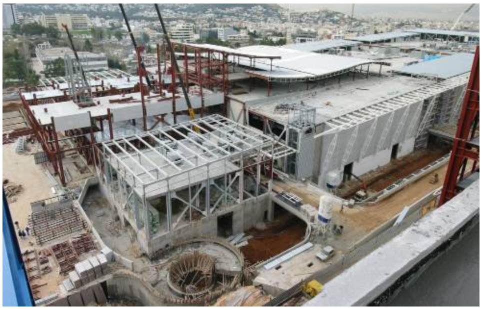
Figures 48 and 49. The Main Press Center (MPC)


Figures 50 and 51. The International Broadcasting Center (IBC) during erection

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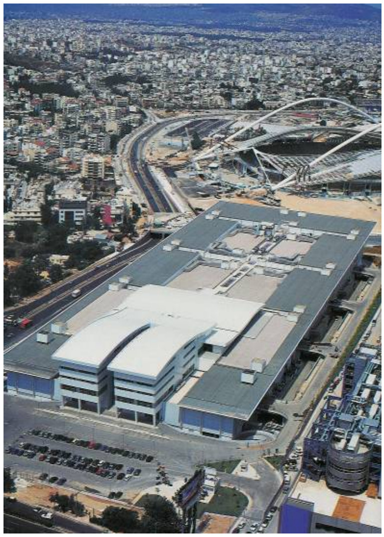

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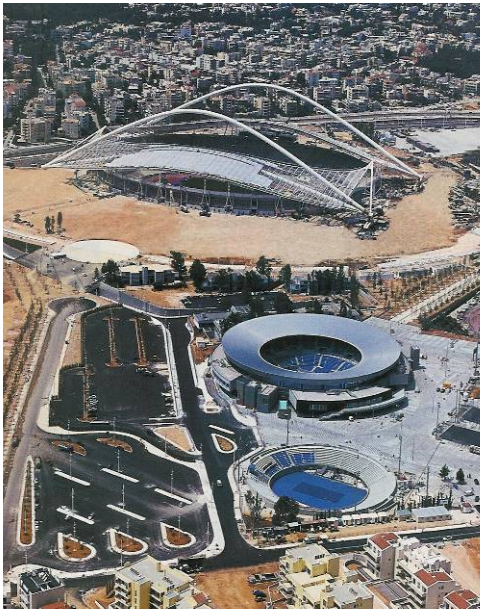

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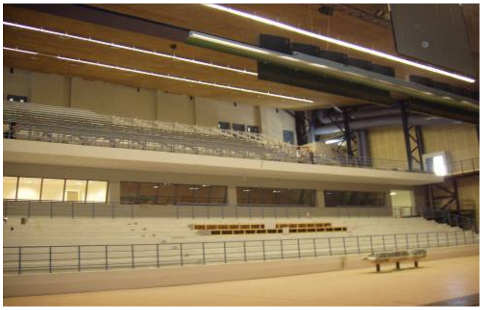

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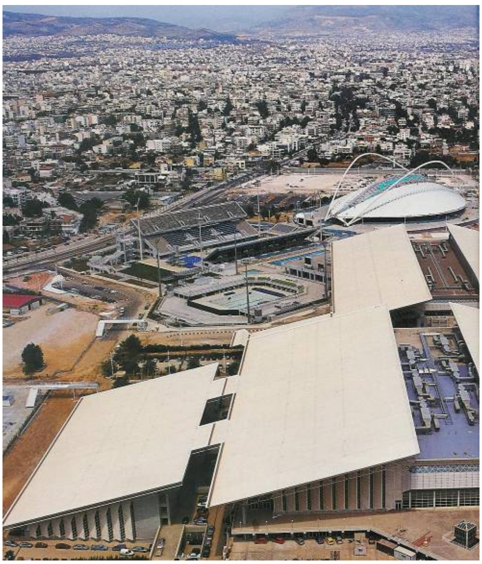
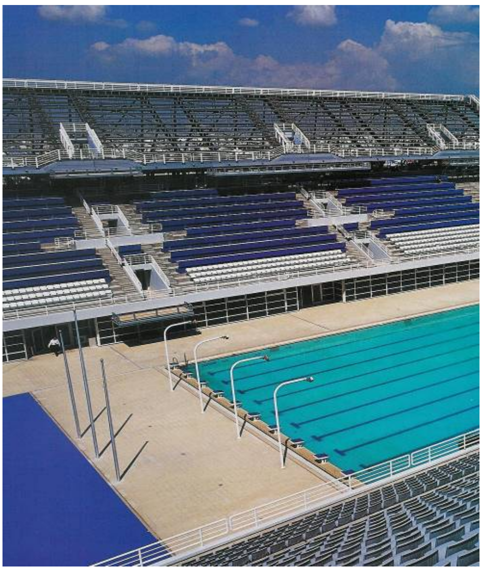

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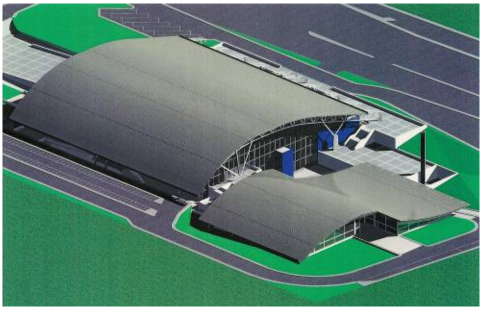


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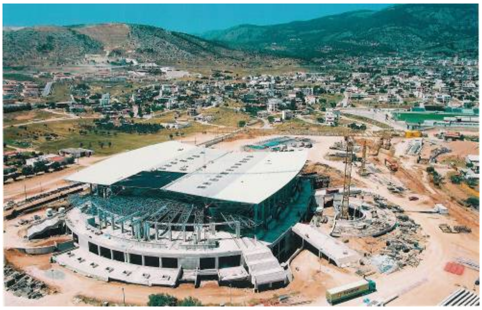


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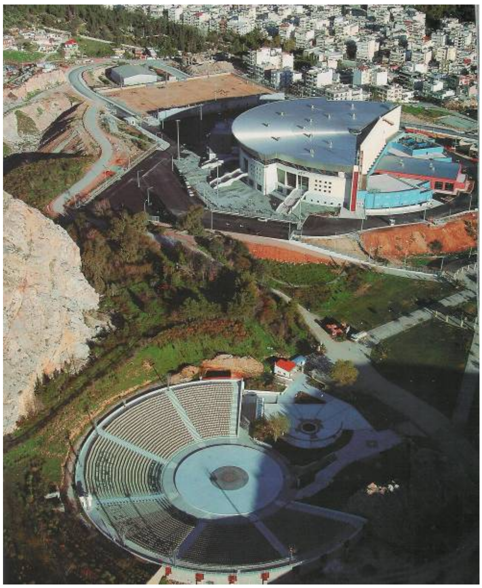

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 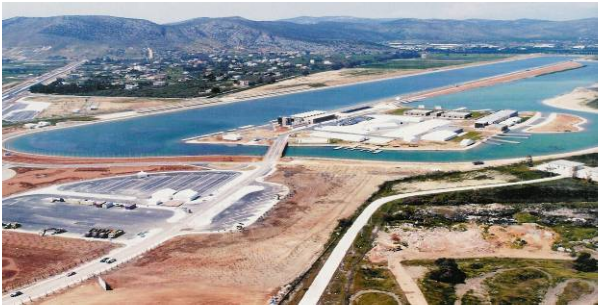

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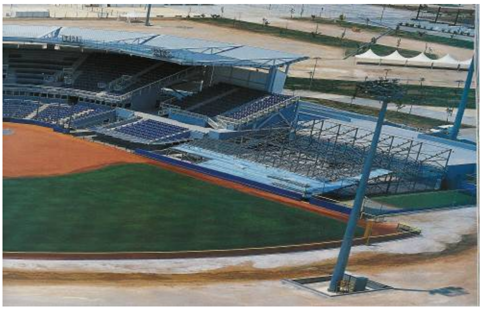 

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TENTH NATIONAL AND
FOURTH INTERNATIONAL
SCIENTIFIC MEETING

iNDiS 2006

NOVI SAD, NOVEMBER 22-24,2006.

Nicolae Taranu[1]
Gabriel Oprisan[2]
Ioana Entuc[3]
Dorina Isopescu[4]

THE USE OF FIBER REINFORCED POLYMERIC COMPOSITES IN STRUCTURAL REHABILITATION OF CIVIL ENGINEERING STRUCTURES

Summary: *The use of fibre reinforced polymer (FRP) composites in structural rehabilitation of civil engineering structures made of traditional materials has now become an almost current practice following the intensive research and development programs carried out in many countries all over the world. Modern repair, strengthening and seismic retrofit techniques increasingly utilize externally bonded polymeric composite products because they offer convenient properties in terms of mechanical strengths, lightness, tailored properties, corrosion resistance, ease of application and other advantages. However a suitable utilization of composite products in structural rehabilitation works requires a detailed analysis of all aspects involved. Strengthening solutions for reinforced concrete elements, masonry and timber members are analysed in the paper underlying the advantages and disadvantages of FRP systems in case of each traditional material involved.*
Key words: *FRP strengthening, plate bonding, fabrics, wrapping, composite jackets*

UPOTREBA POLIMERNIH KOMPOZITA ARMIRANIH VLAKNIMA ZA KONSTRUKCIJSKU SANACIJU GRAĐEVINSKIH OBJEKATA

Rezime: *Upotreba polimernih kompozita armiranih vlaknima (FRP) za konstrukcijsku sanaciju građevinskih objekata izvedenih od tradicionalnih materijala, je postala svakodnevna praksa koja je praćena intenzivnim istraživanjem i razvojnim programima u mnogim zemljama širom sveta. Savremene sanacije, pojačavanje i tehnike povećanja seizmičke otpornosti u velikoj meri koriste spoljno zalepljene polimer-kompozitne proizvode zato što oni obezbeđuju pogodna svojstva, kao što su mehaničke čvrstoće, malu težinu, mogućnost krojenja, korozionu otpornost, laku aplikaciju i druge prednosti. Međutim, odgovarajuća primena kompozitnih proizvoda u radovima konstrukcijske rehabilitacije zahteva detaljnu analizu svih relevantnih aspekata. Rešenja za pojačavanje armiranobetonskih elemenata, zidanih i elemenata od drveta su analizirana u ovom radu, pri čemu su naglašene prednosti i nedostaci FRP systema u slučajevima svakog tradicionalnog materijala posebno.*
Ključne reči: *FRP pojačavanje, athezija ploče, tkanine, omotač, kompozitni omotači*

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1. INTRODUCTION

Over the last three decades structural strengthening of concrete structures has become an important issue due to ageing of infrastructure and the need for upgrading to fulfil more stringent design requirements. Also the seismic retrofit has become more important mainly in seismic active areas. The use of fibre reinforced polymer (FRP) composites in strengthening solutions has become a viable alternative to some of the existing traditional methods due to some advantages such their features in terms of strength, lightness, corrosion resistance and ease of application. Such techniques are also most attractive for their fast execution and low labour costs. FRP composite products for structural strengthening are available in the form of prefabricated strips, precured shapes or uncured sheets applied through wet lay-up procedure.

Prefabricated plates are typically 0.5-1.5 mm thick and 50-200 mm wide, and they are made of unidirectional fibres (glass, carbon, aramid) in a thermosetting matrix (epoxy, polyester, vinylester). Uncured sheets typically have a nominal thickness of less than 1 mm, are made of fibres (unidirectional or bidirectional) preimpregnated or in situ impregnated with resins. Bonding is achieved with epoxy adhesives when prefabricated composite elements are utilised and with impregnating resins in the latter case. Composites were first applied as confining reinforcement of reinforced concrete (RC) columns [1], and as flexural strengthening materials for RC bridge girders [2]. Since the first applications the developments have been tremendous and the range of applications has expanded to timber, masonry and metallic materials. The number of applications involving FRP composites as strengthening materials for RC elements and structures has expanded from a few, about 15 years ago to more than ten thousand nowadays.

2. STRENGTHENING SOLUTIONS OF RC MEMBERS

2.1. Traditional methods

Strengthening solutions of RC members can range from repair of damaged members so that their original load-carrying capacity is restored, to adding elements to increase their strength. All solutions are project-specific to a certain application but some general approaches are commonly utilised. The most traditional techniques for strengthening the RC structures are as follows [3]:

- Increase the reinforced concrete cross-section
- Add prestressing to relieve the dead load
- Use plate bonding to enhance tensile reinforcement of the RC elements
- Add confining elements to improve behaviour of the concrete in the compression members
- Shear strengthening by installing external straps

2.2. FRP composite based solutions

Strengthening of old and/or deteriorated reinforced concrete (RC) members is often required due to the following causes [4, 5]:

- The inadequacy of longitudinal reinforcement in beams and columns, leading to flexural failure. In such cases the bending capacity of concrete elements can be increased through the use of externally bonded FRP plates, strips or fabrics. Alternatively near-surface mounted strips or rods with the fibre direction parallel to the member axis can be utilised.
- The inadequacy of transverse reinforcement, which may have as effect brittle shear failure in structural members like columns, beams, shear walls and beam-column joints. The shear capacity of concrete members can be enhanced by providing externally bonded FRP with the fibres oriented in the transverse direction to the member axis direction, in the case of columns and beams, or in the direction of both the column and the beam direction in the case of beam-column joints.
- Poor detailing in the regions of flexural plastic hinges where the flexural cracking may be followed by cover concrete spalling, failure of transverse steel reinforcement, and buckling of longitudinal steel reinforcement or compressive crushing of concrete. This mode of failure is usually accompanied by large inelastic flexural deformation. By adding confinement in the form of FRP jackets with fibres placed along the column perimeter, the spalling of cover concrete is prevented and the buckling of the longitudinal steel bars is restrained. In this way more ductile responses can be developed and larger inelastic deformations can be sustained.
- Poor detailing in lap splices. This mode occurs in columns in which the longitudinal steel reinforcement is lap spliced in the maximum bending moment regions near the column ends. Debonding may occur once vertical cracks develop in the cover concrete and progresses with cover spalling. By increasing the lap confinement with fibres along the column perimeter the flexural strength degradation can be prevented or limited.

The use of FRP reinforcement cannot modify the stiffness characteristics of existing RC elements; hence the FRP strengthening technique is not applicable if the structural intervention is aiming at increasing stiffness rather than strength or ductility [5].

2.2.1. Flexural strengthening of beams

The need for methods of repair and strengthening of RC beams and girders has been imposed by: degradation due to corrosion of steel reinforcement, cracking of concrete due to excessive carbonation, freeze-thaw action, spalling of concrete cover, effects of alkali-silica reactions and changing in loading patterns [6]. In case of bridges the need for increasing their load carrying capacities requires the adoption of a cost-effective technology that will not distress the traffic significantly. In buildings the materials deterioration and changing needs for building occupancy imposes, in many cases, the strengthening of existing beams. One of the conventional methods for external strengthening implies the addition of adhesive-bonded steel plates on the tension side of the RC beams. The use of epoxy-bonded steel plates is very frequent in Europe and the United States but it suffers from a number of disadvantages:

Steel plates are heavy and difficult to transport, handle and install; the length of individual steel plates is restricted to 8-10m to enable handling and even at these lengths it may be difficult to erect them due to pre-existing service facilities; durability and corrosion effects remain uncertain; contaminants on structural members prior to bonding; surface preparation including the priming systems; steel plate thickness at least 5 mm to prevent distortion during blasting operation; complex profiles are difficult to be shaped with steel plates; expensive false work is required to maintain steel plates in position during bonding.

Composites fabricated either through wet processes on-site or prefabricated in plates, Figure1, and then adhesively bonded to the concrete surface provide an efficient means of strengthening, that can be carried out with no or little disruption in use. The efficacy of the method depends mainly on the appropriate selection of the composite material and on the efficiency and integrity of the bond between the composite and the concrete surface.

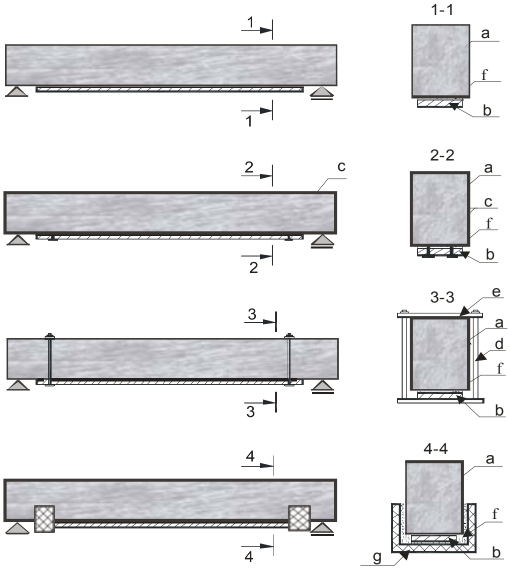

Figure 1. Strengthening of RC beams with FRP soffit plates a- concrete; b-FRP plate; c-anchor bolts; d, e - elements of the metallic jig; f - adhesive layer)

2.2.2. Shear strengthening of beams

When a RC beam is deficient in shear, or when its shear capacity is less than the flexural capacity after flexural strengthening, the shear strengthening of the respective beam has to be considered. It has been realized that the FRP bonded to the soffit of a RC beam does not modify significantly the shear behaviour from that of the unstrengthened beams [7, 8], Therefore, the influence of FRP strips bonded to the soffit for flexural strengthening may be ignored in predicting the shear strength of the beam. Various bonding schemes of FRP strips have been utilized to improve the shear capacity of reinforced concrete beams. The shear effect of FRP external reinforcement is maximized when the fibre direction coincides to that of maximum principal tensile stress. For the most common case of structural members subjected to transverse loads the maximum principal stress trajectories in the shear-critical zones form an angle with the member axis which may be taken about 45o. However, sometimes it is more practical to attach the external FRP reinforcement with the principal fibre direction, perpendicular to the axis direction, Figure 2, [9]. Because FRPs are strong in the direction of fibres only their orientation is recommended to control the shear cracks best. Shear forces in a beam may be reversed under reversed cyclic loading and fibres may be thus arranged at two different directions to satisfy the requirement of shear strengthening in both directions.

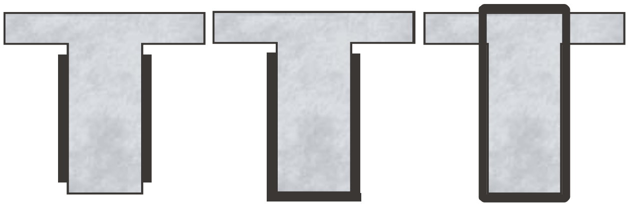

Figure 2. Shear strengthening schemes with FRP composites a-FRP bonded to the web sides only; b-U jacketing; c-complete wrapping

2.2.3. Strengthening of RC plates

When the RC plates are simply supported the one-way plates are strengthened by bonding FRP strips to the soffit along the required direction, Figure 3. For two-way plates strengthening must be applied for both directions, by bonding FRP strips in both directions, Figure 4.

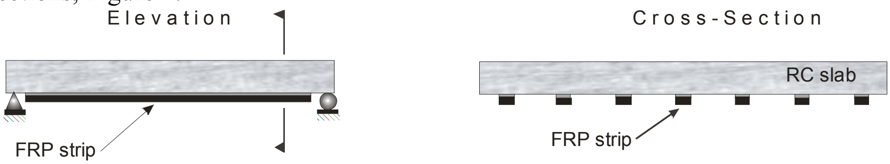

Figure 3. FRP strengthening of one-way simply supported plate: a- elevation; b- cross section

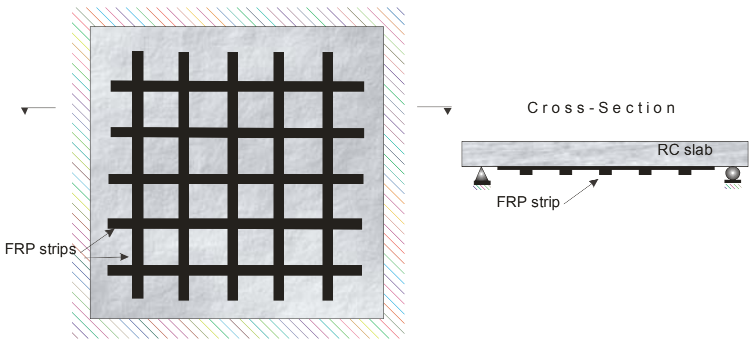

Figure 4. FRP strengthening of a two-way slab:a-slab soffit; b- cross section

The possible collapse mechanism of a two-way slab suggests that the strengthening of such a plate can be concentrated in the central region, Figure 4, and the FRP strips can be terminated far away from the edges [4]. The load capacity of such strengthened plates can be predicted by a yield line analysis, as the part of the slab without bonded FRP strips has enough ductility for the formation of yield lines.

2.2.4. Strengthening of RC columns

Conventional strengthening measures for RC columns range from the external confinement of the core by heavily reinforced external concrete sections to the use of steel cables wound helically around the existing column at close spacing that are then covered by concrete and the use of steel jackets welded together in the field confining the existing columns [10]. Some of these methods are effective but they have some disadvantages: they are time consuming and labour intensive; can cause significant interruption of the structure functioning due to access and space requirements for heavy equipment; rely on field welding, the quality of which is often questionable; susceptible to degradation due to corrosion; introduce changes in column stiffness, influencing the seismic force levels. The strengthening of existing RC columns using steel or FRP jacketing is based on a well established fact that lateral confinement of concrete can substantially enhance its axial compressive strength and ductility [11]. The most common form of FRP column strengthening involves the external wrapping of FRP straps. The use of FRP composites provides a means for confinement without the increase in stiffness (when only hoop reinforcing fibres are utilised), enables rapid fabrication of cost effective and durable jackets, with little or no traffic disruption in most cases. In FRP-confined concrete subjected to axial compression, the FRP jackets are loaded mainly in hoop tension while the concrete is subjected to tri-axial compression, so that both materials are used to their best advantages. As a result of the confinement, both the strength and the ultimate strain of concrete can be enhanced, while the tensile strength of FRP can be effectively utilized. Instead of the brittle behaviour exhibited by both materials, FRP-confined concrete possesses an enhanced ductility.

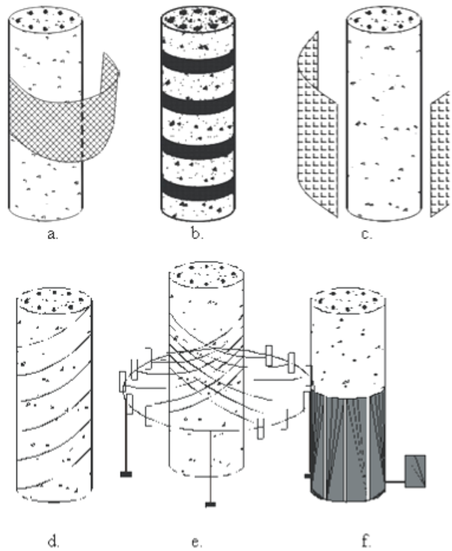

Figure 5. Methods of FRP strengthening for RC columns: a. wrapping of fabric; b. partially wrapping with strips; c. prefabricated jackets d. spiral rings; e. automated winding; f. resin infusion.

For FRP wrapped, axially loaded columns the design philosophy relies on the wrap to carry tensile forces around the perimeter of the column as a result of lateral expansion of the underlying column when loaded axially in compression. Constraining the lateral expansion of the column confines the concrete and, consequently increases its axial compressive capacity. It should be underlined that passive confinement of this type requires significant lateral expansion of the concrete before the FRP wrap is loaded and confinement is initiated. In case of columns rectangular or square in cross section the confinement is effective at the column corners only with negligible resistance to lateral expansion being provided along the flat column sides. A number of different methods (based on form of jacketing material or fabrication process) have been tested at large or full-scale many of which are now used commercially all over the world. A suitable classification of FRP composite jackets is given in, Figure 5 [12, 13].

2.2.5. Strengthening of unreinforced masonry walls with composite materials

Unreinforced masonry (URM) is considered one of the oldest construction materials being until the end of XIX century, the basic material for: foundations, walls, columns, volts, staircases, floor joists, roofs, retaining walls, drainage channels, barrages, etc. Construction with URM elements posses a series of advantages such as: fire resistance, thermal and acoustic insulations between interior and outside spaces, humidity resistance. However the URM elements have some significant inconveniences such as: large self weight (heaviness causes cracks in the other elements of the structures), reduced mechanical strengths in comparison with other traditional materials (steel and concrete), low tenacity, great manual labour consumptions, and vulnerability to earthquakes. Various factors cause deteriorations which must be overcome by strengthening solutions.

Some of the most efficient strengthening procedures using different FRP reinforcements [14] are presented in fig 6.

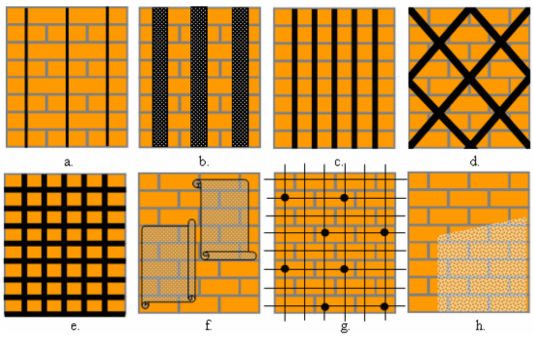

Figure 6. Strengthening procedures using FRP for URM: a. Composite bars embedded near surface in joints; b. Composite sheets bonded to wall surface; c .FRP plates; d. cross arrangements of FRP plates; e. grid disposition of FRP; f. carbon or glass fabric moisten in epoxy resin; g.FRP mesh; h. spray up technique.

An innovative strengthening system to provide both direct tension and in-plane shear transfer from the wall to the floor slab and a strengthening system using glass or carbon fabrics bonded to perpendicular surfaces with a thickness and size designed for the required loading, fig.7, was tested at the University of California in Irvine [15].

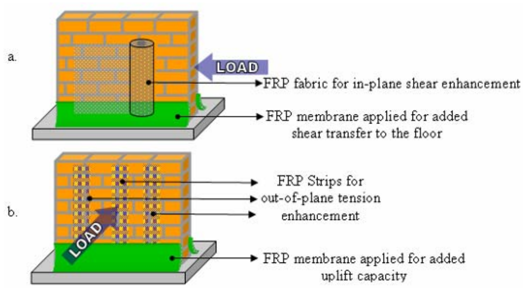

Figure 7. Strengthening system using FRP for URM brick walls: a.In-plane shear strengthening; b. Out-of-plane tension strips (face loaded, single side application)

2.2.6. Strengthening of timber members using fibre reinforced polymer composites

The reinforcement of structural wood products has become in the last decades an efficient method of improving structural capabilities of load carrying members made of this material. Some important steps in earlier stages of research were focussed on using metallic reinforcement, including steel bars, prestressed stranded cables, and bonded steel and aluminium plates. A disadvantage of the metallic reinforcement was the poor compatibility between the wood and the reinforcing materials. In comparison with metallic reinforcement, fibre reinforced polymers (FRP) composites are compatible with structural wood products leading to efficient hybrid members.

Strengthening of timber beams for flexure is similar to strengthening concrete beams in flexure, fig.8. The strengthening procedure utilised impregnated fabrics (made of carbon or glass fibres) with epoxy resins, sheet or plate adhesively bonded to the bottom of the wood beams. Because the FRP are strong in tension and the tension zone of the wood beam is on the bottom side, the combination FRP and wood beams make an efficient composite material in itself. By placing FRP plates or fabrics on the bottom of the wood beams some defects (like knots, cracks, and decay zones) can be outrun, making the wood beam stronger.

When using (FRP) to strengthening wood members (column, beams, panels, connections, etc) the following advantages are identified [16]: increasing in strength and/or stiffness, the variability in mechanical properties is reduced, allow using lower-grade and/or fast-growing species in construction products, reducing the size and weight of the wood structural members, increasing the product ductility, serviceability, and fatigue performance, enhancement of the product durability and dimensional stability.

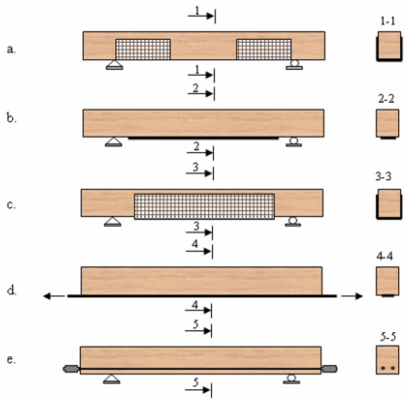

Figure8 FRP strengthening solutions for wood beams: a. shear strength improvement using bonded CFRP/epoxy or GFRP/epoxy; b. resistance moment improvement using bonded CFRP plates at the bottom side of the beam; c .increasing of shear strength and moment with "U" jacketing; d. prestressed CFRP plate; e. CFRP strands or rods [13].

The steps strengthening of poles with FRP, especially for poles affected under the ground line are shown in fig. 9:

- excavation, bearing of the pole, cracks identifications, brushing, artificial drying of the surfaces;
- resin injection in the cracks of the pole;
- surface priming with the epoxy resin or the adhesive;
- confinement of the pole with fabrics.

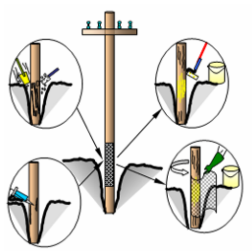

Figure 9 Methods of strengthening applied at the ground line of the pole

FRP strengthening provide an extension of exploitation duration of the wood and assure the ability to assemble, move, rebuild or strengthen (partially or totally) the timber structures. The compatibility of the wood and FRP characteristics is certain and a convenient combination of these materials can be designed. Some repair and strengthening schemes for timber columns are suggested in fig. 10. These types of strengthening using FRP materials could be applied on portions of wood column or on the entire length of the element, when the size of the damaged area requires such an intervention.

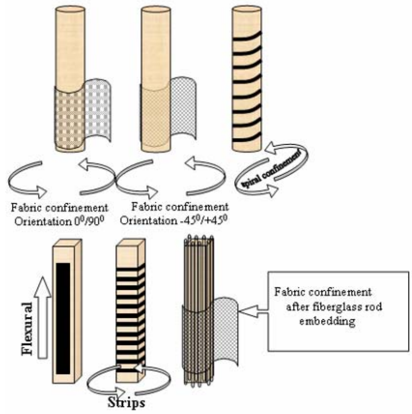

Figure 10 FRP strengthening technique applied to wood columns

3. ADVANTAGES OF FRP COMPOSITE STRENGTHENING

- FRP composite have higher ultimate strength and lower density than steel, although the strength to density ratio much higher than steel plate can not be generally fully utilised.
- The lower weight of FRP materials makes handling and installation significantly easier than in case of steel plates. Composite plates applied to the soffit of bridge girders do not require heave lifting equipment. When FRP plates are applied pressure is exerted to their outer surface to remove adhesive in excess and entrapped air. They can practically be left unsupported. In general there is no need to use bolts for FRP plate fixing and this avoids the risk of damaging the existing steel reinforcing bars.
- FRP composite sheets are available in long lengths (compared to steel plates generally limited to 6m) and their installation is much simpler: laps and joints are not required; the material can accommodate some irregularities; the thin FRP plates and sheets can follow a slightly curved shape without prebending; overlapping required when strengthening plates in two directions is not a problem because the composite products are thin.
- The energy required to produce FRP materials is less than for traditional materials fact that leads to sustainable solutions with minimum impact on the environment.
- The combination of all these advantages leads to simpler and quicker strengthening processes than when steel products are utilised. This is especially important for bridges because of the high costs of circulation lanes closures.

4. DISADVANTAGES OF FRP COMPOSITE STRENGTHENING

- The most important disadvantage of FRP externally strengthened structures seems to be the risk of accidental damage, vandalism or fire occurrence. However strengthening using FRP plates affected by the composite products damage can only reduce the overall factor of safety and it is unlikely to lead to collapse.

- New unfamiliar failure mechanisms are possible particularly in FRP plate bonding and specialist expertise should be provided [13].
- Workmanship skill and quality are critical to the success of applying an FRP composite strengthening solutions. Therefore certification schemes for workers and supervisors are needed to be developed prior to application of these procedures especially at important works.
- It is difficult to control the quality of the adhesive layer or the presence of the entrapped air than can affect the bond between FRP plate and the concrete surface.
- Experience on the long-term properties of FRP strengthening schemes is limited, and this can be a disadvantage for structural members requiring a very long design life.
- The relatively high initial cost of the FRP materials and products used in the strengthening schemes is a perceived disadvantage but the comparisons should be made on the complete strengthening procedure and life-cycle assessment.
- Many potential clients may claim the lack of experience of most operators in the construction market but this can be overcome by choosing qualified designers and contractors.

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NOVI SAD, NOVEMBER 22-24,2006.

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GENERAL CONCEPTS REGARDING THE REHABILITATION OF OLD STEEL HIGHWAY BRIDGES

Summary:
In the western part of Romania, on the highway network still are in operation, after two World Wars and other events, some old highway bridges erected on the beginning of the last century. These structures are "witnesses of the past" representing technical monuments with emblematic character. The duty of the administration is to maintain these structures in service. The paper presents an overview on the evaluation methodology of the remaining service life on the base of classical and new principles.
Some case studies are presented: bridge in Mihaileni, Bocsig, Lugoj, Savarsin, and especially the bridge in Arad, a symbol for the town.

Key words: *bridges, highway, evaluation, rehabilitation, service life*

OPŠTI PRISTUPI REHABILITACIJI STARIH ČELIČNIH MOSTOVA NA AUTO PUTEVIMA

Rezime*:*
U istočnom delu Rumunije na mreži autoputeva još uvek su u upotrebi stari mostovi građeni početkom prošlog veka. Ovi objekti su "svedoci prošlosti" reprezentujući to vreme kao spomenici - sa karakterom amblema.Dužnost administracije je da održava ovakve objekte u toku eksploatacije. U radu je dat pregled metodologije procene preostalog veka trajanja objekta na osnovi klasičnih i novih principa.
Prikazani su neki primeri (studije slučajeva)kao što su: most u MIhaileni, Boscigu, Lugoju, Savarsinu i naročito most u Aradu, koji je simbol grada.

Ključne reči*: mostovi,autoputevi, procena-vrednovanjem rahabilitacija, upotrebi vek*

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1. INTRODUCTION

Romania has a highway network of 153 057 km on which there are placed 3192 bridges (Romanian Highway Administration Report 2003). From the total number only 83 (≈ 3%) are steel bridges. The usual range of these bridges is situated in the field of medium spans and was dominated in the past by concrete bridges. The lack of motorways must be also to be emphasized (only 113 km). At the same time 57 % of the total number of bridges is designed for the Loading class I (Lorries A 13, S 60). For them the National Administration of Roads proposes (in a long term program) their rehabilitation and upgrading to Loading class E (Lorries A 30, V 80).

They are a reduced number of old steel highway bridges, generally older than 80 – 90 years, mainly in the western part of Romania, which are witnesses of the past. Even if these structures are in operation the technical condition of these bridges is not satisfactory. Some aspects can be noticed:

- Insufficient clearance (the width → B = 5,0 – 6,0 m) and therefore vertical members of the main truss girders damaged by the impact with vehicles;
- The maintenance of these structures is not continue, the owners have changed in the history (different administrations), the corrosion is important;
- The complete lack of documentation is general;
- Insufficient knowledge of material qualities;
- All these structures are riveted;
- Tendency of the administration to replace these structures.

In comparison with the existing railway bridges, the situation is different. First of all by the railways the number of steel bridges is dominating, the maintenance is better, documentation is generally available.

Consequently the rehabilitation of old existing highway bridges is not current and some recommendations, according to the experience obtained by the rehabilitation of some representative structures can be done. It should be mentioned that all these structures have an historical and artistical value.

2. PRESENT VERIFICATION METHODOLOGY

The rehabilitation of bridges is a complex matter. The Romanian Highway Administration adopted a qualitative verification methodology based on the appreciation by the expert of the technical condition of the structures (AND 522-2002). Some quality indexes are defined; finally the technical condition of the structure is given by

$$I_{ST} = \sum_{i=1}^{i=5} C_i + \sum_{i=1}^{i=5} F_i \tag{1}$$

The quality index C is a sum of five aspects:

C_1 → is referring to the main girder
C_2 → is referring to the deck elements
C_3 → is referring to the infrastructure and bearings
C_4 → is referring to the bed river
C_5 → is referring to the quality of the deck surface.

For the functional requirements also five aspects are relevant

F_1 → expresses the traffic condition on the bridge
F_2 → expresses the loading class of the highway
F_3 → takes into consideration the year of construction and structure type
F_4 → is referring to the quality of fabrication, erection and operation conditions
F_5 → is referring to the maintenance of the structure.

For every index marks (from 1 – 10) are given. Finally the technical condition of the structure results from the total sum and can be ranged in one of the following categories:

- Very good technical condition;
- Good technical condition;
- Satisfactory technical condition;
- Unsatisfactory technical condition;
- The present technical condition can not assure the safety of the structure.

On these conclusions the administration can take a decision regarding the time planning of maintenance works. This method is suitable for concrete bridges and relatively new (20 -30 years old) steel bridges. For older steel bridges it can have only an informative character. A more refined methodology must be adopted. In this direction the experience obtained by the verification of steel railway bridges can be used.

Following the examination of the existing documentation a simple analysis of the structure is recommended. This can lead to some immediate restrictions in circulation. In situ tests are possible and not as expensive as railway bridges tests. The results can be used for the calibration of the structure. Material tests from secondary elements - or if it is possible from main elements are useful.

The analysis of technical conditions of these bridges can also contain non destructive tests. They are possible only after the removing of the deck and cleaning of the structure. In this phase a detailed and carefully inspection of the structure by the expert is compulsory.

Taking into account the year of construction, the following assumptions can be made:

- approx. 1900 – 1920, mild steel with a low carbon content;
- after 1920 mild steel with the qualities of St 37.

Still existing wrought iron bridges were generally replaced immediately after the Second World War.

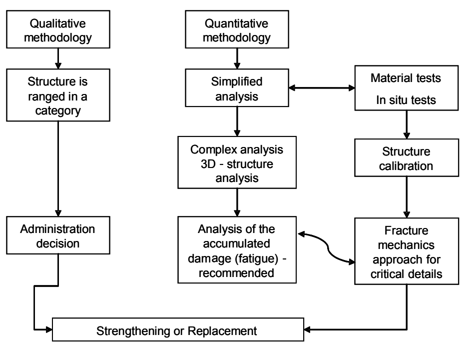

Figure 1. Methodology for the verification of old existing highway bridges.

Charpy tests made with samples from existing bridges are relevant; due to the large dispersions of values, for useful conclusions a large number of Charpy tests are necessary. Generally, steel fabricated in this time (providing from the same steel plants, like Resita or Gyor) has the same qualities as the steel used in railway bridges:

- % C is situated in the range 0,09 – 0,16%;
- the yielding stress is 230 N/mm^2;
- the 27J transition temperature is around 0 - +5°C based on Charpy V Notch tests.

A general scheme of the verification methodology is presented in figure 1.

3. RECOMMENDATIONS FOR THE IMPROVING OF THE PRESENT METHODOLOGY

Space analysis of the structure is useful for skew bridges, bridges in curvature (rarely by old structures) or for continuous truss girders. For bridges in alignment, the results are not too different from a traditional calculus. However, a computer aided analysis is in present, easy to be added to the file of the structure.

A more difficult problem is the fatigue assessment of the structure. Even if in the usual standard for highway bridges (STAS 1844-75) this verification is not foreseen, the damage accumulation methodology applied for railway bridges can be adopted. In this direction, for the Wöhler curve the assumptions made by the Swiss Railways for existing bridges – Figure 2 can be adopted.

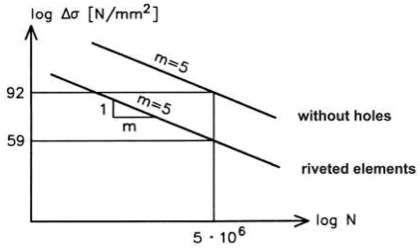

Figure 2. Wöhler curves adopted by the Swiss Regulation

Difficult is evaluation of the stress history. Information about the circulation in different periods are necessary. For example, in the case of the bridge in Savarsin (§ 4) a stone quarry has functioned for three years, about 20 trucks (with 30 tones) crossed the bridge daily. With the stress history and the appropriate Wöhler curve, the accumulated damage can be evaluated.

$$D = \sum \frac{n_i}{N_i} \leq 1 \tag{2}$$

For bridges with usual spans situated on the National Highway Network, taking into account the circulation in the past, fatigue problems were not decisive. It is worth to mention that the Swiss standard SIA 161 offers a very informative diagram in this sense. For important structures with large spans and critical details, a next step, the Fracture Mechanics approach is recommended.

The presence of cracks in structural elements modifies essentially their fracture behaviour. Fracture, assimilated in this case as crack dimensions growth process under external loadings, will be strongly influenced by the material toughness. The authors proposed a complementary method based on the fracture mechanics basic concept

$$J_I \leq J_{Ic} \tag{3}$$

In practice two situations can be distinguished:

- **D < 0,8** the probability to detect cracks is very low. The inspection intervals (generally between 3 – 6 years) can be established on criteria independent of fatigue. Nevertheless, a special attention must be paid to critical details.
- **D ≥ 0,8** cracks are probable and possible. An in situ inspection and the analysis of critical details are strongly necessary. Also a fracture mechanics approach is recommended.

Generally, establishing the maintenance program, the determination of inspection intervals, the inspection priorities of structural elements and finally the calculation with high accuracy of the remaining service life of existing steel bridges takes into account the following main data:

- type of structure and exploitation conditions (traffic events);
- information about structural steel (mechanical properties – yield strength, tensile strength, hardness, transition curve ductile – brittle fracture and transition temperatures, chemical composition, metallographic analysis);
- determination of critical members and details;
- crack detection and inspection techniques for evaluation of the initial crack size – a_0 and crack configuration;
- recording of the stress spectrum for the critical members under the actual traffic loads;
- evaluation of the critical crack size – a_{crit} based on failure assessment diagrams;
- fracture mechanics parameter – J_{crit}, δ_{crit}, (fracture toughness);
- simulation of the fatigue crack growth;
- temperature, environment conditions.

4. RECOMMENDATIONS FOR THE IMPROVING OF THE PRESENT METHODOLOGY

The bridge in **Mihăileni** is situated on the National Highway DN 74 (km 17 + 600). The structure was erected in 1895. There is no documentation about the bridge. The structure is composed by five stringers I 36, eleven cross girders and a parabolic main truss girder. The width of the bridge 5,60 m, does not correspond to requirements of a National Highway. The deck made by Zorres elements, filled with ballast is heavy, supporting the asphalt surface. The stresses computed in the structure for the present loads (A 30, V 80) exceeded the allowable values by 15 – 20 %. Considering the insufficient clearance (Figure 3), the structure was finally demolished with the recommendation to be reused on secondary road with reduced traffic. An approximate fatigue calculus performed by the damage accumulation method revealed that the structure – due to the relatively reduced traffic in the past – did not reach the limits of the design life.

The bridge in **Bocsig** over the Crisul Alb River is situated on the local highway DJ 792 A (km 1+192m). The bridge was erected in 1902 and designed to a reduced loading class (A 10, S 40). The structure has generally the same structural form and components as the precedent bridge. The main girder is a parabolic truss with a span L =

16 x 3,82 = 61,12 m. The upper and lower chord are single span truss members, the diagonals and vertical elements are slender, Due to the position in a curve, the end vertical post is damaged by the impact with the vehicles. The width of the bridge is 5, 00 m with two foot path of 0,50 m (Figure 4). The maintenance of the bridge is unsatisfactory, but generally with strengthening measures, the bridge can be maintained in circulation.

Figure 3. The bridge in Mihăileni; frontal view

Figure 4. The bridge in Bocsig; general view ***Figure 5. The bridge in Săvârşin;***

The bridge in **Săvârşin** over the Mureş River on the local higway DJ 707 A (km 1 + 271 m) is a remarkable structure with four spans (Figure 5) erected in 1897.

In Table 1 the cross section of the main girders elements is presented. The technical condition of the bridge is unsatisfactory, the elements are corroded and some verticals and diagonals are damaged by the impact with the vehicles. The existent floor beams, stringers and cross girders are simple supported elements. The deck consists on Zorres elements filled with ballast, supporting an asphalt surface. In present the structure has a special importance being the only crossing of the river in a large area. It can be also mentioned that in Savarsin is the present summer residence of the former King of Romania Mihai I.

Taking into account the importance of the structure, its historical value the decision of strengthening of the structure was taken:

- for the stringers the flanges were consolidated by supplementary plates (Figure 6);
- the cross girders were transformed in switch girders (Figure 6);
- for the lower chord of the main girder a supplementary tie member was chosen (Figure 6);

- for the upper chord the direct strengthening with two angles, improving also the local stability was chosen (Figure 7);
- diagonals and vertical members have to be first of all straighten, and strengthen by additional plates;
- the old floor system was replaced by a composite deck.

All these operations are difficult and suppose a high technical level of all in situ works.

Element	Cross - section
Upper chord (final panels)	8x360; L90x90x9; 355x13 / 2Pb 8x360; L90x90x9; 355x13
Lower chord (panels 1,2,9,10/ 3,8 / 4,5,6,7)	360x13; L90x90x9 / 13x360; L90x90x9; Pb 9x240 / 13x360; L90x90x9; 2Pb 9x240
Final diagonals	2Pb 9x270; L70x70x8 / L70x70x8
Diagonals (panels 3,4,7,8 / 5,6)	L80x80x8 / L90x90x9
Final posts	2Pb 330x8; L70x70x7
Current posts	L70x70x7 / L60x60x8 / L60x60x6

Table 1. Cross section of the main girder elements.

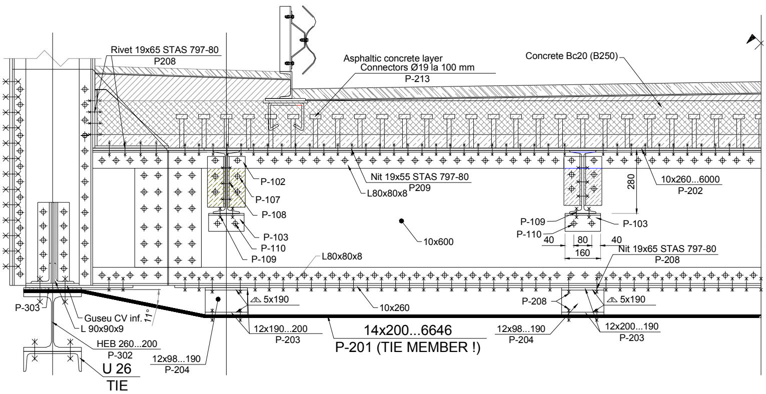

Figure 6. Strengthening of the structure (cross section)

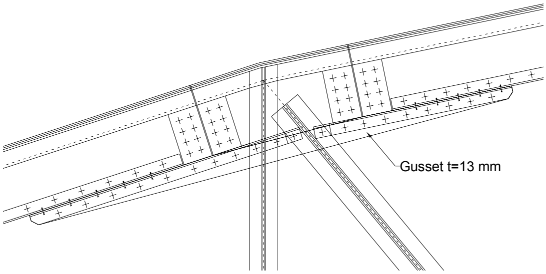

Figure 7. Upper chord strengthening

The bridge in **Lugoj** is situated in the middle of the old town and is a symbol for the people and it is know as the "Iron Bridge". The year of construction is probably 1905 (no documents in this direction). The structure with two spans L = 2 x 33,68 m is generally in a good technical condition. In 1982 general maintenance works were performed; the classical deck with ZORRES elements eas replaced by a concrete deck in interaction with the steel structure trough rigid connectors (Figure 8).
Nevertheless a evaluation of the remaining fatigue life is necessary, the bridge was for many years the principal crossing over the river Timiş for the National Highway DN6 (Timişoara – Bucharest) and consequently, the traffic in the past was important.

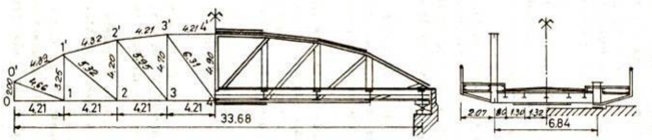

Figure 8. Strengthening of the structure 1982

Figure 9. The bridge in Lugoj – general view

5. CONCLUSION

The paper gives an overview of the assessment methodology for old steel highway bridges. Every case must be separately considered.

Nevertheless the rehabilitation of such representatives structures is one of the main task of the bridge engineers.

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TENTH NATIONAL AND
FOURTH INTERNATIONAL
SCIENTIFIC MEETING

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NOVI SAD, NOVEMBER 22-24,2006.

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LARGE SCALE MODEL PRE-EXPERIMENTAL ANALYSES OF MUSTAFA PASHA MOSQUE

Summary: Large scale model of Mustafa Pasha Mosque is a subject of experimental and numerical analyses. *Numerical analyses are to help in prediction of the large scale model response to shaking table tests and to put some bounds and estimates for large scale model behavior in order to prevent and avoid any unexpected premature heavy damages or even collapse of the test structure. Such numerical assessments of structural behavior would help in specifying locations for measurements of characteristic response values during the test of the* large scale model as well as in deciding about the choice of input time histories and its intensities in the experiment. It is expected that *shaking table tests would globally confirm the pre-experimental numerical analysis results in such a way that* large scale model *damage propagation would be kept under control. All analyses were executed for unified material mechanical characteristics. Linear modal analyses, response spectrum analyses and time history analyses were performed for different values of elasticity modules and different damping ratios. Material non-linear push over analyses were performed for different material models and for variable input material mechanical characteristics in order to estimate crack initiations, crack propagation and base shear force close to collapse.*
Key words**: **Stone-brick Masonry, Time-history, Nonlinear Material Models, Push-over Analysis.

PRE-EKSPERIMENTALNE ANALIZE MODELA U VELIKOJ RAZMERI MUSTAFA PAŠINE DJAMIJE

Rezime: *Predmet razmatranja je eksperimentalna i numerička analiza Mustafa Pašine džamije. Numeričke analize su radjene da bi se predskazao odgovor modela prilikom testiranja na vibroplatformi i kako bi se procenile granice u kojima se kreće, a da se pri tome izbegnu neočekivana prevremena velika oštećenja, pa čak i rušenje ispitivanog modela. Numeričke procene ponašanja konstrukcije će pomoći u određivanju mernih mesta za akviziciju podataka za vreme eksperimentalnog testa modela kao i u izboru vremenskih istorija i intenziteta ubrzanja. Očekuje se da će eksperimenti na vibroplatformi globalno potvrditi rezultate pre-eksperimentalnih analiza tako da oštećenja modela budu pod kontrolom. Sve analize su uradjene za materijal čije su mehaničke osobine unificirane. Uradjene su: modalne linearne analize, analize spektralnog ponašanja i analize vremenskih istorija za različite vrednosti modula elastičnosti i prigušenja. Materijalne nelinearne push-over analize su uradjene za različite materijalne modele i za različite ulazne mehaničke karakteristike materijala sa ciljem da se proceni početak pojave pukotina, njihovo propagiranje kao i sile smicanja u osnovi modela neposredno pre rušenja.*
***Ključne reči**: Zidarija u kamenu i cigli, vremenska istorija, nelinearni materijalni modeli, push-over analiza.*

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INTRODUCTION

16 scientific institutions from 12 countries (Italy, Belgium, Portugal, Greece, Slovenia, Romania, R.Macedonia, Turkey, Israel, Egypt, Morocco, Algeria) are involved in the European Project: **"Earthquake Protection of Historical Buildings by Reversible Mixed Technologies -PROHITECH"** in the frame of **FP6.** Without entering into explanation of complexity of the project, just one piece of the activities that should be done in the scope of it is a very detailed analysis of a chosen, existing historical buildings (belonging to cultural heritage of particular country which is characteristic and typical to the Mediterranean basin). One of those chosen historical buildings that should be subject of experimental and numerical analyses is the Mustafa Pasha Mosque in Skopje, R. Macedonia. Reference to the existing mosque in what follows will be as *prototype structure*. The model of the prototype (physical or numerical), which is in geometric scale 1/6 with the prototype will be addressed as *large scale model*.

Three main groups of numerical and experimental activities are underlined that should be accomplished for the mosque structure. The objective of first group numerical analysis activities is to help in prediction of the large scale model response to shaking table tests. These so called pre-experimental analyses are to put some bounds and estimates for large scale model behavior in order to prevent and avoid any unexpected, premature heavy damages or even collapse of the test structure. Such numerical assessments of structural behavior would help in specifying locations for measurements of characteristic response values during shaking table test of the large scale model as well as in deciding about the choice of input time histories and its intensities in the experiment. One should expect that shaking table tests would globally confirm the pre-experimental numerical analysis results in such a way that large scale model damage propagation would be kept under control.

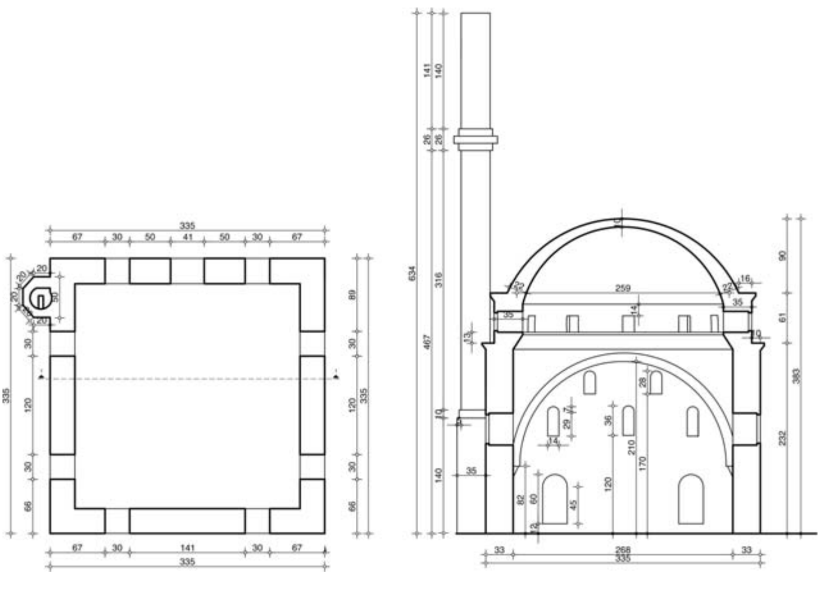
Second group of numerical analysis activities would follow after shaking table tests are done. Experimental results should help in recalibration of input values for the numerical analysis model to obtain better identification with the tested large scale model. A good correlation of experimental and analytical results would create an application pattern for real structures. Next step should be to strengthen the large scale model by a chosen reversible technique and to accomplish the shaking table test of the large scale model by applying additional ground motion time histories.

Third group of numerical analysis activities are directed toward prototype structure. Whatever has been done for the large scale model should be repeated as a case study for the prototype.

Partial results of first group of numerical analysis activities are presented in this paper. The large scale model of the mosque with minaret, ready for experimental testing on shaking table, was built in masonry of stone, clay brick and lime mortar. Some of the masonry unit's mechanical characteristics were obtained experimentally. Complementary compression tests and shear test on masonry wall panel specimens were effectuated. All numerical analyses were executed for unified material mechanical characteristics. Linear modal analyses, response spectrum analyses and time history analyses were performed for different values of elasticity modules and different damping ratios using program SAP2000. Material non-linear push over analyses were performed using program LUSAS for different material models (modified von Misses, Mohr-Coulomb, Drucker-Prager and crack and crush concrete model) and for variable input material mechanical characteristics in order to estimate crack initiations, crack propagation and base shear force close to collapse.

LARGE SCALE MODEL GEOMETRY

The layout and the elevation of the large scale model are shown in Fig.1. The physical model is a 1/6 scale replica of the prototype structure of the mosque. Walls of the body of the mosque (33cm tick) are built in three layers thru its thickness. The outside layers, 10cm tick, are made of masonry in stone, brick and lime mortar and the inside (middle) layer, 13cm tick as an infill of mortar. The one layered pandentifs are made of masonry in stone, brick and lime mortar as well as is the tambour. The masonry dome of the mosque with variable thickness is made of bricks in lime mortar. The lower part of the minaret is embedded in the wall of the mosque. Its outer diameter is 52cm and the inner diameter is 32cm. The outer diameter of the upper part of the minaret is 42cm and the inner diameter is 22cm. Both parts of the minaret are stone masonries. All entities of the mosque are shown in Fig.2. The finite element mesh of all large scale model entities is covered with quadrilateral shell elements. Structure weight without RC pedestal was taken to be 217kN.



6.1.

Figure 1. Layout and the elevation of the large scale model

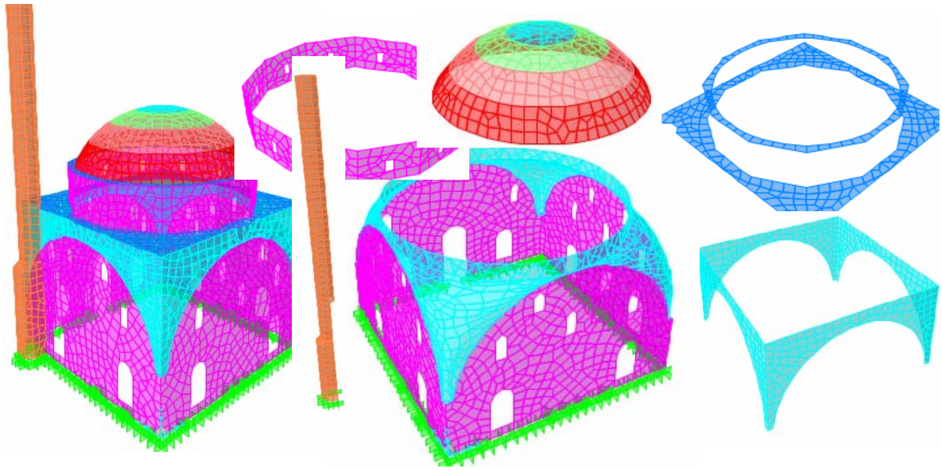


Figure 2. Entities of the mosque

MATERIAL CHARACTERISTICS

To preserve similarity of the large scale model with the prototype structure the similitude requirements have to be satisfied. Details for proposed material mechanical characteristics that satisfy these requirements are given in ref. [1] and are transferred in **Table 1**. Many parameters (input values) are involved in controlling dynamic structural behavior. Some of them are easily measurable with high reliability and certainty but some are difficult to quantify due to their scarcity even if experimental data exists. Good examples of the later are the mechanical properties of compound masonry. Details for mechanical characteristics of stone masonry units used in building the model are given in ref. [2]. Mechanical characteristics of clay brick masonry units and the lime mortar used in building the model are given in ref. [3]. Three additional masonry wall compression tests and one masonry wall shear test are completed, Fig.3. Tested masonry walls force-displacement relations are presented in Fig.4. The masonry units in those tested masonry walls are with the same mechanical properties as mentioned above. All masonry units or masonry walls' mechanical characteristics available from tests are summarized in **Table 2**.

Scaling parameter	Scaling factor	Units	Prototype	Model	Adopted scaling factor
length	l_r	m	20/20	3.3/3.3	1/6
time	l_r	sec	60	10	1/6
frequency	l_r^{-1}	Hz	3.0	18	6
gravity acceleration	neglected				
input acceleration	l_r^{-1}	g	0.1-0.2g	0.6-1.2g	6
mass density	1	kN/m^3	19.0	19.0	1
strain	1	μstr			1
normal compress. stress	1	MPa	0.28	0.06	
modulus of elasticity	1	MPa	680	680	
compressive strength	1	MPa	27	27	
shear strength	1	MPa			
• stone			0.15	0.15	
• brick			4.7	4.7	
• mortar			0.1	0.1	
displacement	l_r	mm			1/6
force	l_r^2	kN			1/36

Table 1. Comparison of principal properties of prototype and model following " Gravity forces neglected" similitude low for Mustafa Pasha mosque - Skopje

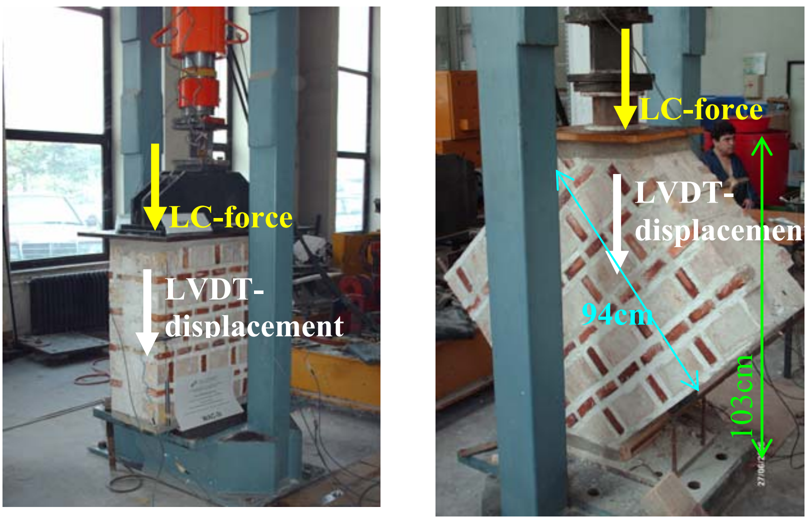


Fig. 3. Compression test ant shear test of the masonry wall

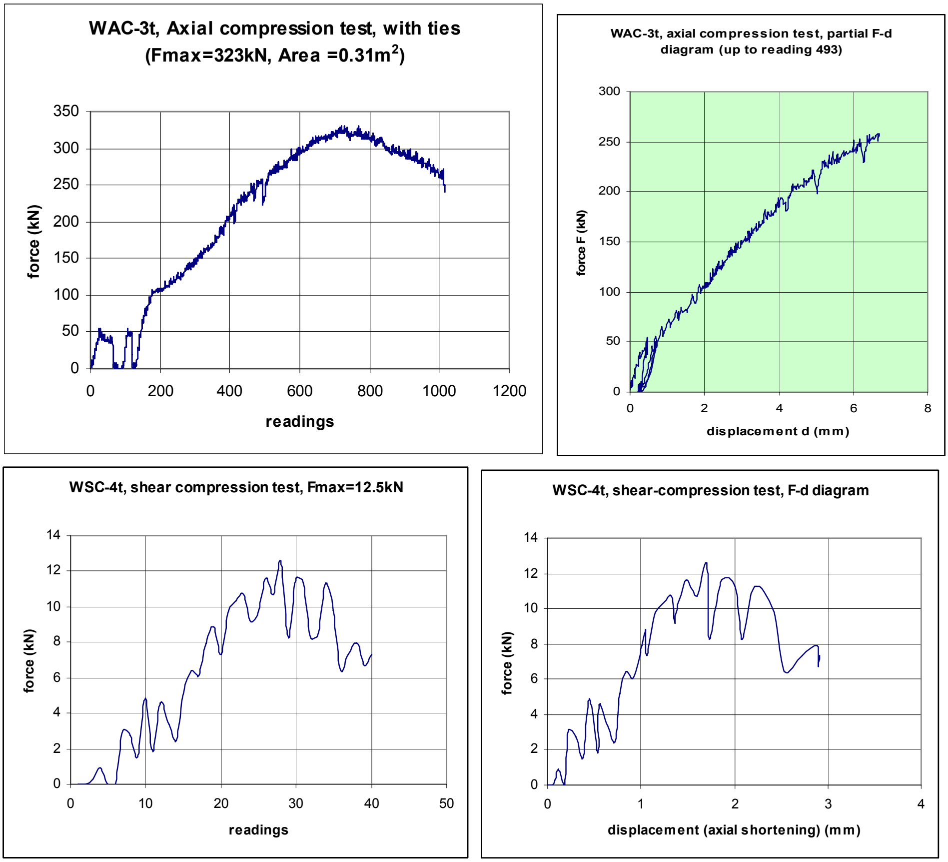


Fig. 4. Masonry walls force-displacement relations

		Elasticity modulus-initial E_0 [MPa]	Elasticity modulus-secant E_s [MPa]	compressive strength [MPa]	Rupture strength [MPa]	Tension strength [MPa]	Bond strength [MPa]
Masonry units	Stone	6700-9000	6700-9000	53.6		5.94	
	Large Brick			9.7			
	Small Brick			19.4			
	Mortar			0.5 (0.6)	0.4		
Masonry wall		290	100	1.04			

Table 2. Summary of masonry units and masonry walls mechanical characteristics available from tests

All empty spaces for mechanical characteristic in **Table 2**, necessary for numerical analyses, have to be completed indirectly from literature sources, empirical formulas or additional numerical analyses on masonry wall test specimens. Masonry wall compression strength, as a basic mechanical characteristic that was obtained from experiments, was checked using most appropriate proposed formula by EC 6 [4],

$$f_k = K \cdot f_b^{0.65} \cdot f_m^{0.25} \text{ [N/mm}^2\text{]}$$

f_k – masonry compressive strength
f_b – masonry unit compressive strength
f_m – mortar compressive strength

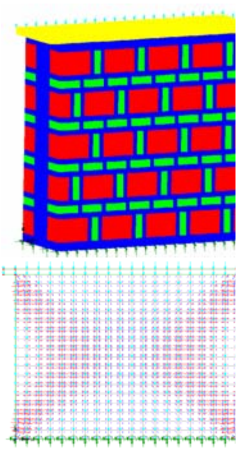
Masonry units compressive strengths:

Small bricks: f_b = 9.7 MPa
Larger bricks: f_b = 19.4 MPa
Stone: f_b = 53.6 MPa
Mortar: f_m = 0.5(0.6) MPa

structure	**7. K**	f_b	f_m	f_k
mosque	0.4	9.7	0.5	1.4730
mosque	0.4	9.7	0.6	1.5417
mosque	0.4	19.4	0.6	2.4192
mosque/minaret	0.4	53.6	0.6	4.6833
mosque	0.6	9.7	0.6	2.3126
mosque	0.6	19.4	0.6	3.6288
mosque/minaret	0.6	53.6	0.6	7.0249

Table 3. Estimated bounds for masonry compressive strength [MPa]

It is evident that the lowest masonry design compressive strength f_k (already reduced by a safety factor) in **Table 3**, obtained from the empirical formula, is higher then what was measured from masonry compressive tests (1.04MPa). In the course of numerical push-over analyses for different material models with unified material mechanical characteristics some additional estimates are made in order to define necessary stress and strain parameters relevant for particular nonlinear material model. The tensile or flexural strength of masonry is controlled by the weakest link of the system which is in most instances the tensile bond strength of the masonry unit – joint interface. Because of the lack of strong bonding between joint material and masonry units the tensile strength of masonry is much smaller than its compressive strength, and may in some instances approach zero. Many expressions and estimates are suggested in the literature [5,6,7] to connect the compressive strength of the masonry unit with the compressive strength of the masonry and the modulus of elasticity of the masonry. Similar expressions exist to correlate joint material (mortar) compressive strength with its modulus of elasticity, its Poisson's ratio and compressive strength of the masonry. Data obtained from the masonry wall compression tests and from the masonry wall shear test experiments were used in numerical simulations of these experiments. Trial and error procedure was used for various combinations of input mechanical properties for the stone, brick and mortar units in a materially nonlinear 3D micro solid elements discretization of the experimental test panels, ref. [8]. The scope of the analyses was to approach numerically the experimentally obtained force – displacement curve. When satisfactory closeness of the experimental to the analytical curve was achieved a new series of numerical experiments were performed to get equivalent unified mechanical characteristics for the masonry. Summary of these sets of numerical analyses in order to calibrate the numerical model with experimental data are shown in Fig.5 and Fig.6.



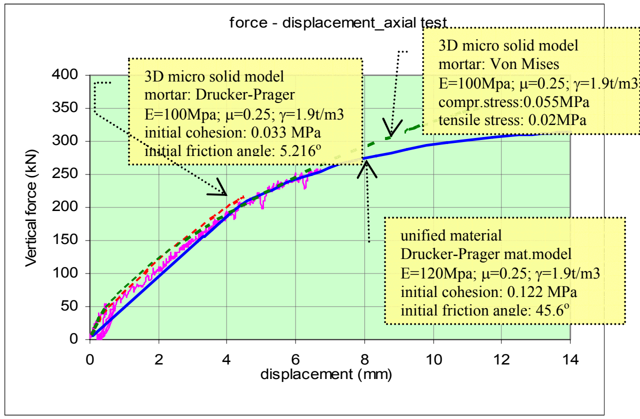
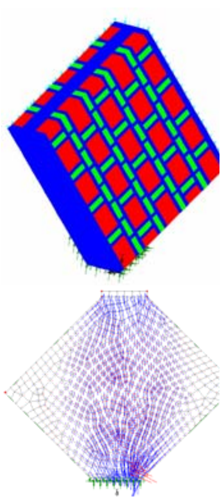


Figure 5. Calibration of the numerical model with experimental data, compression test



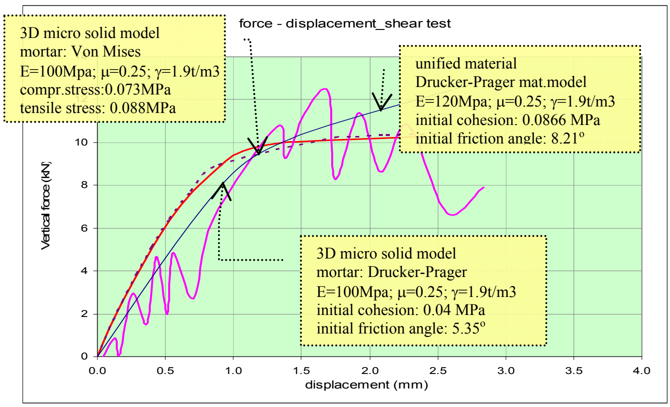


Figure 6. Calibration of the numerical model with experimental data, shear test

As was expected, the crucial material mechanical characteristic was the tensile bond strength of the masonry unit (stone or brick) – joint interface with mortar.

MODULUS OF ELASTICITY

In linear elastic analysis, the only material characteristics that are needed are the elasticity modulus E and Poisson ratio ν. In materially nonlinear analyses despite elasticity modulus at list uniaxial compression strength and uniaxial tension strength additionally are needed to fully define the yield criterion function. In what follows, outside the previous section on material characteristics, complementary considerations on the value of the elasticity modulus E for the homogenized mosque and minaret masonry are given.

Values for elasticity modules or more correctly deformation modules are very important in all numerical analyses. For defined geometry of the large scale model, defined choice of finite elements and mesh refinement one can influence structural stiffness and structural dynamic characteristics in linear elastic time history numerical analyses only thru the values of elasticity modulus, Poisson ratio and structural damping. It is clear that the elasticity modulus is the most speculative characteristic in a way that the linear structural response to a particular ground motion could be manipulated. One can shift structure response spectrum values in different ranges of ground motion response spectra by playing with the value of E.

Numerical calibration of elasticity modulus for the body of the mosque and for the minaret was done from small amplitude vibrations test of the large scale model. Additional estimates for material mechanical characteristics mentioned in previous section would emerge in attempt to accomplish a fair fit for the fundamental frequencies obtained by applying small amplitude free vibration test experimentally and numerically. Comparison of small amplitude vibration dynamic characteristics for the large scale model with small amplitude vibration dynamic characteristics for the prototype are given in **Table 4**.

Structure	1^{st} frequency [Hz]	2^{nd} frequency [Hz]	3^{rd} frequency [Hz]
Prototype	1.04	3.0	3.2
Large.sc.model	7.0	18.6	18.8

7.1.1.1. Table 4. Frequencies from small amplitude vibration tests

One could create a picture about the fulfillment of similarity requirements between the prototype structure and the test model by comparing the experimental values of ratio ($T1_{prototype}$ / $T1_{l.sc.model}$) as well as the ratio [$(T_1/ T_2)_{prototype}/(T_1/ T_2)_{l.sc.model}$].

$T1_{prototype} / T1_{l.sc.model} = 6.731$ (close to 6)

$T2_{prototype} / T2_{l.sc.model} = 6.2$ (close to 6)

$T3_{prototype} / T3_{l.sc.model} = 5.875$ (close to 6)

$(T_1/ T_2)_{prototype} = 2.885$, $(T_1/ T_2)_{l.sc.model} = 2.657$

$(T_1/ T_2)_{prototype}/(T_1/ T_2)_{l.sc.model} = 1.086$ (close to 1)

The above dynamic characteristics, experimentally obtained, have a very good correlation with the scale factor for the geometry of the large scale model (1/6) which is taken as a default to compress the time scale for the ground acceleration records. Obtained results as well as similitude theory justify the equivalence of geometric scale with the time scale ($\lambda_l = \lambda_t = 1/6$). On the other hand the numerical calibration of elasticity modules due to unconditional trust to lowest large scale model small amplitude vibration test frequencies requires rather high values for the elasticity modulus for the minaret $E_{minaret}$=9000MPa and for the body of the mosque E_{mosque}=1000MPa. Due to discrepancy of these numerical values with the experimentally obtained ones in **Table 2.**, in the time history and push-over analyses that proceed, different combinations of values for material mechanical properties are considered in order to bound the response of the large scale model.

DYNAMIC ANALYSIS

No commercial computer program exists that could perform nonlinear time history analysis for structures modeled with 3D elements. This is mainly to the fact that the unloading branch and all consecutive yielding episodes in structural behavior are not satisfactorily defined when 3D elements are applied. Obviously, the most interesting type of analysis concerning this project, such as the structural behavior due to strong motion records is, should be bounded in the frame of linear analysis capability options. Under such circumstances the engineering judgment and experience in analyzing the obtained results from linear analysis runs are very important in assessment of any component of structural nonlinear behavior extrapolated from the consecutive linear ones. This fact puts serious restrains on all analysis results. To some degree this disadvantage could be smoothen by good estimates of possible bounds for induced forces, stresses and displacements offered by available linear analysis options.

Linear modal analyses, response spectrum analyses and time history analyses are performed for different values of elasticity modules and different damping ratios using program SAP2000. All structural elements of the mosque and the minaret are tick shell quadrilaterals. The number of elements is 4188 and the total number of nodes is 4002.

In order to avoid any premature damage of the test model or even its collapse, it is important to put some predefined bounds on amplitudes of input acceleration records. In that sense an estimate for upper bound of acceleration intensity that could be produced by the shaking table platform was made. The capacity of the shaking table is to produce acceleration of 1.2g. However, for this particular large scale test model (approximate mass of 24t to 30t) the range for maximum acceleration amplitudes that could be achieved would be 0.6g to 0.8g. For example, in the time history analyses we are dealing with, applying El Centro acceleration record Array #9, 180 (USGS Station 117), Imperial Valley 5/19/40 0439 (source PEER Strong Motion Database Record) with pick ground acceleration PGA=0.313g, the upper value for the amplification factor for acceleration amplitudes is 2.56 instead of 6.

Table 5. presents modal analysis results for two sets of elasticity modules. Emosque=1000MPa and Eminaret=9000MPa are used in order to get close to the large scale model ambient vibration test fundamental frequences. Emosque=120MPa and Eminaret=2000MPa are an estimate for high level of damage.

A constant modal damping is used in response spectrum analyses. An equivalent of 5% of critical damping for small amplitude vibrations would probably be real. For moderate amplitude vibrations and higher ground acceleration inputs 10% damping was used and for strong motion acceleration inputs, for what was expected heavy structural damage, a value of 20% of critical damping was used. One could argue that such high modal damping values are inappropriate, especially, because inherently, additional structural damping is included thru the changing modulus of elasticity. The answer to such arguments is not straightforward but in supporting the opposite it has to be underlined that in assessing the bounds of structural dynamic nonlinear behavior no much more could be done by employing linear analysis.

	$E_{minaret}$=9000MPa E_{mosque}=1000MPa		$E_{minaret}$=2000MPa E_{mosque}=120MPa	
Mode	Period	Frequency	Period	Frequency
	[sec]	[cyc/sec]	[sec]	[cyc/sec]
1	0.147103	6.798	0.326716	3.0608
2	0.13897	7.1958	0.309463	3.2314
3	0.053645	18.641	0.151538	6.599
4	0.051611	19.376	0.144451	6.9228
5	0.034942	28.619	0.099128	10.088
6	0.031874	31.373	0.090154	11.092
7	0.029154	34.301	0.078754	12.698
8	0.025098	39.844	0.071447	13.996
9	0.024277	41.191	0.069275	14.435
10	0.023417	42.704	0.066441	15.051
11	0.02254	44.366	0.059028	16.941
12	0.02189	45.682	0.057517	17.386

Table 5. Modal analyses.

The EUROCODE 8 elastic response spectrums for 5%, 10% and 20% damping for amplitude of acceleration a=1m/sec2 are shown in Fig. 7. The period (T) is compressed by 6 as is the time scale. Spectral values for base shear forces in X, Y and Z directions for amplitude of acceleration a=8m/sec2 are given in **Table 6**. for the above specified percentages of damping (RS_d5, RS_d10 and RS_d20) . This amplitude of acceleration corresponds to the estimated potential of the shaking table to produce output acceleration of approximately 0.8g. The shaking table test is designed to be realized in X direction.

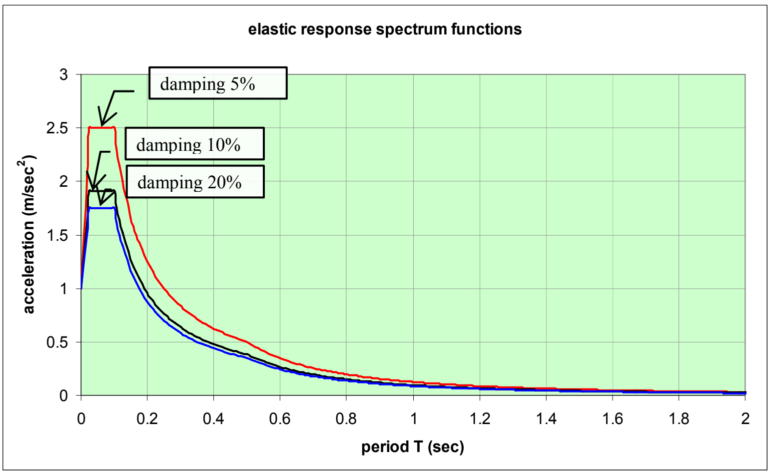


Figure 7. EUROCODE 8 elastic response spectrums

Extreme base shear forces from time history analyses ECTH_t6_ a=8m/s^2_d5, ECTH_t6_ a=8m/s^2_d10 and ECTH_t6_ a=8m/s^2_d20 in **Table 6**. are for 5%, 10% and 20% damping. The original El Centro acceleration record is magnified by factor 2.56 (in order to produce a shaking table output acceleration of approximately 0.8g) and the time scale is compressed by 6, Fig.8.

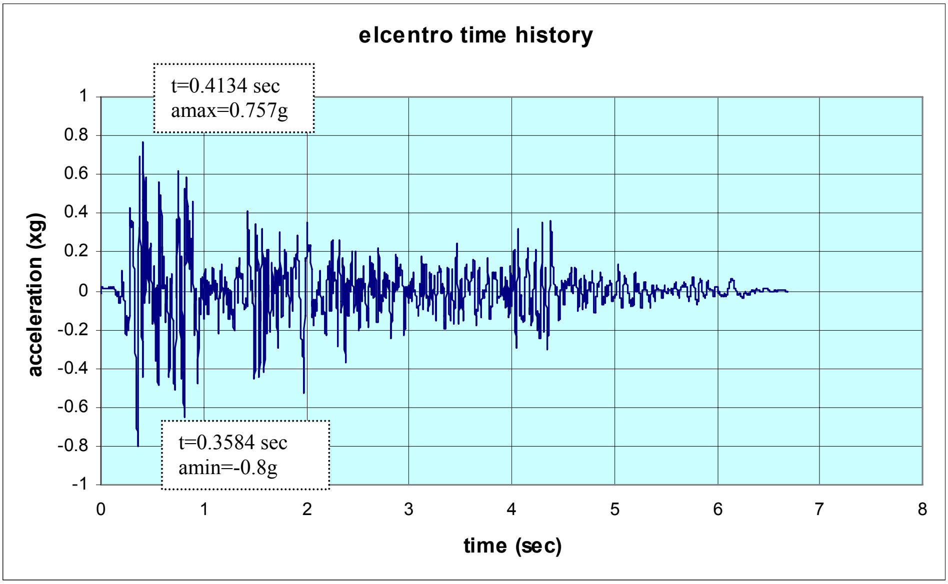


Figure 8. El Centro acceleration record magnified by factor 2.56, time scale compressed by 6

LinRespSpec/ LinModHist		$E_{minaret}$=9000MPa E_{mosque}=1000MPa			$E_{minaret}$=2000MPa E_{mosque}=120MPa		
		GlobalFX [KN]	GlobalFY [KN]	GlobalFZ [KN]	GlobalFX [KN]	GlobalFY [KN]	GlobalFZ [KN]
RS_d5	Max	297.03	73.94	12.84	199.61	60.11	27.50
RS_d10	Max	232.30	29.84	10.08	157.77	25.31	22.85
RS_d20	Max	215.26	14.00	8.78	147.29	12.88	21.25
ECTH_a=8m/s^2_d5	Max	260.88	52.25	11.71	178.05	58.05	24.97
ECTH_a=8m/s^2_d5	Min	-230.85	-53.49	-11.02	-187.78	-58.28	-26.11
ECTH_a=8m/s^2_d10	Max	193.31	20.97	9.64	136.48	23.83	22.25
ECTH_a=8m/s^2_d10	Min	-179.20	-21.32	-8.94	-120.38	-23.40	-21.64
ECTH_a=8m/s^2_d20	Max	143.26	9.14	6.88	96.27	8.02	17.15
ECTH_a=8m/s^2_d20	Min	-141.68	-11.22	-6.22	-79.97	-7.52	-13.86

Table 6. Base reactions

The influence of modules of elasticity as well as the damping ratios is evident from **Table 6**. For example the base reaction in direction X for 5% damping and for higher modules of elasticity is 2 to 3 times higher then the corresponding value for 20% damping and for lower modules of elasticity.

Componential stresses S11, S22 and S12 for few characteristic elements are given in **Table 7**. The location of these elements is shown in Fig.9. Local element axes 1 and 2 are in the plane of the element. Local axis 3 is orthogonal to element plane.

Elem. No.	Comp. Stress [MPa]		Eminaret=9000MPa, Emosque=1000MPa				Eminaret=2000MPa, Emosque=120MPa			
			Dead	5%	10%	20%	Dead	5%	10%	20%
3040	S11	min	-0.005	-0.061	-0.039	-0.028	-0.003	-0.079	-0.034	-0.014
		max		0.060	0.042	0.027		0.081	0.041	0.020
	S22	min	-0.070	-4.398	-3.057	-2.007	-0.062	-1.877	-1.506	-1.103
		max		4.104	2.726	2.133		2.798	1.892	1.074
	S12	min	0.005	-0.086	-0.061	-0.038	0.006	-0.063	-0.043	-0.029
		max		0.073	0.057	0.043		0.075	0.046	0.026
43	S11	min	-0.026	-0.078	-0.058	-0.042	-0.026	-0.063	-0.038	-0.024
		max		0.090	0.058	0.039		0.058	0.037	0.021
	S22	min	-0.133	-0.399	0.296	-0.217	-0.130	-0.323	-0.196	-0.124
		max		0.458	0.297	0.199		0.295	0.189	0.110
	S12	min	0.000	-0.013	-0.009	-0.007	0.000	-0.010	-0.007	-0.005
		max		0.012	0.010	0.007		0.012	0.007	0.005
108	S11	min	-0.031	-0.228	-0.172	-0.130	-0.030	-0.163	-0.118	-0.084
		max		0.221	0.162	0.124		0.163	0.107	0.072
	S22	min	-0.147	-1.057	-0.795	-0.599	-0.123	-0.746	-0.547	-0.389
		max		1.011	0.744	0.576		0.746	0.490	0.329
	S12	min	-0.004	-0.076	-0.056	-0.043	-0.004	-0.053	-0.040	-0.028
		max		0.069	0.055	0.041		0.059	0.037	0.025
89	S11	min	-0.031	-0.091	-0.068	-0.045	-0.032	-0.095	-0.054	-0.033
		max		0.100	0.067	0.051		0.083	0.047	0.032
	S22	min	-0.147	-0.402	-0.285	-0.185	-0.152	-0.425	-0.237	-0.139
		max		0.433	0.287	0.214		0.380	0.208	0.133
	S12	min	0.000	-0.016	-0.014	-0.010	0.000	-0.014	-0.010	-0.007
		max		0.018	0.013	0.009		0.013	0.009	0.007
76	S11	min	-0.031	-0.197	-0.154	-0.114	-0.032	-0.170	-0.108	-0.073
		max		0.212	0.158	0.122		0.150	0.112	0.079
	S22	min	-0.152	-0.980	-0.763	-0.565	-0.157	-0.845	-0.534	-0.360
		max		1.055	0.783	0.606		0.742	0.559	0.390
	S12	min	0.000	-0.012	-0.010	-0.008	0.000	-0.012	-0.008	-0.005
		max		0.014	0.009	0.007		0.010	0.007	0.005

Table 7. Element component stresses due to 2.56 x original El Centro (PGA ≅ 0.8g, time scale compressed by 6)

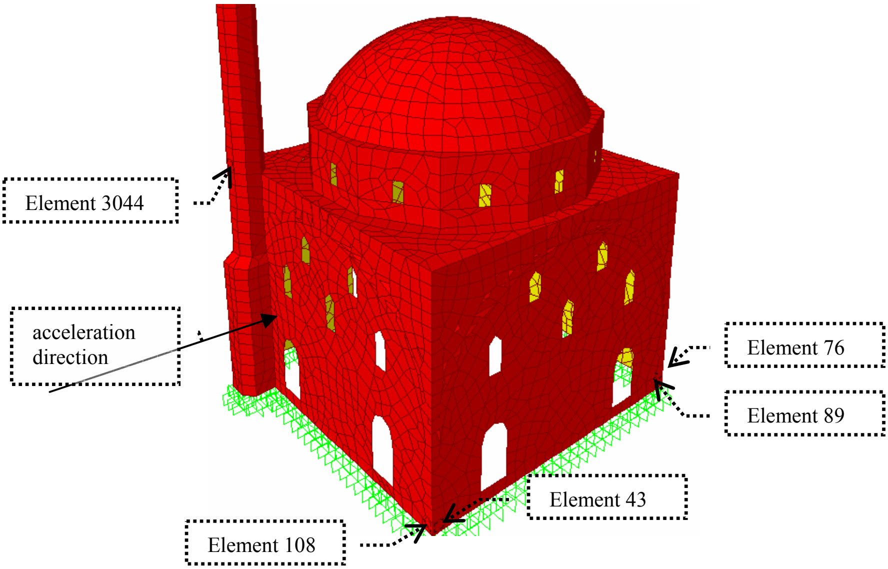


Figure 9. Element locations

A question arises about usefulness of these linear dynamic analyses results in assessing structural behavior in nonlinear range. Initial cracks are easily detected if all material strength characteristics are known. It is a simple matter of scaling the intensity of the used earthquake acceleration record such that the maximum tension stress in any element in its time history superimposed with its associated stress due to structure self weight is equal to the material strength in tension. For example, the minaret as a weakest part of the structure, meaning that first cracks would appear in horizontal mortar layer at the section close to the extrusion of the minaret thru the body of the mosque (most probably element 3040), would sustain approximately PGA = 0.03g if a critical bond stress at the interface of the mortar and the stone was taken as 0.15MPa. In a similar way, if the influence of the minaret on the behavior of the mosque was taken as negligible, initial cracks of the mosque are expected for approximately PGA=0.1g to 0.15g.

No additional comments of the dynamic analyses results will be given here due to limited space in this paper. More detailed explanations and comments are given in ref. [8].

In a more profound parametric study, dynamic analyses for a series of different earthquake time histories should be performed, or more likely for some that are authentic to the region where the structure is located in order to get better estimates for response bounds.

PUSH-OVER ANALYSIS

All push-over analyses are obtained using LUSAS with tick shell quadrilateral elements for different material models: modified von Misses[1], Mohr-Coulomb, Drucker-Prager and cracking/crushing concrete. The number of elements is 3491 and the total number of nodes is 2748. For each loading step with monotonically increasing horizontal

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[1] Model for ductile material is used for a crude but very stable estimate of behavior.

force in global X direction load increments were distributed proportional to the structural elements' masses, independent on its Z coordinates. As in the previous section, when the program SAP2000 was used, a rather coarse element mesh is also applied for push-over analyses. Insisting on a finer element mesh won't be appropriate inasmuch as other input parameters in these analyses are more unreliable or let say more scarcely distributed and prone to engineering judgment. Different input material mechanical characteristics are used for the mosque and for the minaret. In some computer runs the minaret is taken to behave as linear elastic in some as nonlinear. Some are without minaret. Exhaustive parametric push-over analysis could be found in ref. [8]. Plenty of tabular, graphical and visual output results could be analyzed in different ways giving priority to some presentations over the others. Very few among them are presented in this paper showing *lateral force-top of dome horizontal displacement in X direction* (P-Δ) as well as some escorting pictures showing cracking/yielding developments for chosen levels of horizontal force intensities up to assessed large scale model bearing capacity. P-Δ relations in Fig.10 are obtained by implementing Drucker-Prager material model for two pairs of calibrated unified input material properties that satisfy the wall panel compression test in the first run and that satisfy the wall panel shear test in the second run. Fig.11 presents two characteristic stages in crack/yields developments. The first is close to cracks initiation in the mosque and the second is close to collapse of the model. Both cases are for unified material strengths calibrated from the wall panel compression test. These results correspond to collapse estimates for the large scale model in the range of PGA=0.9g to 1.4g.

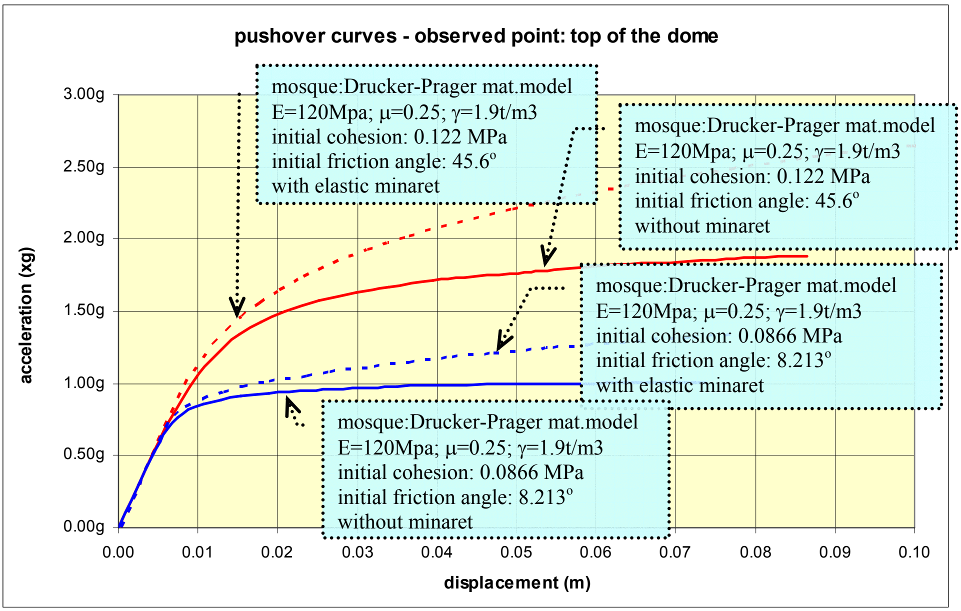
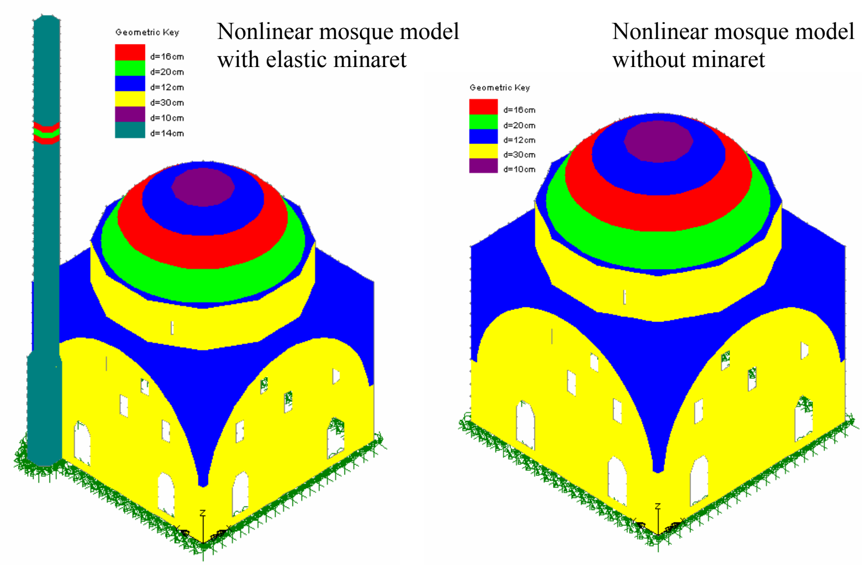
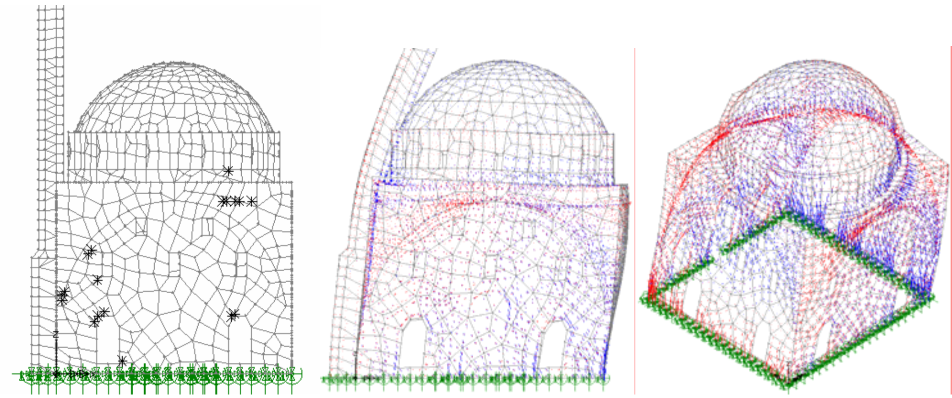


Figure 10. P-Δ relations are obtained by implementing Drucker-Prager material model for two pairs of calibrated unified input material properties that satisfy the wall panel compression test in the first run and that satisfy the wall panel shear test in the second run.



-initial yield points and principal strain vectors



-yield points and principal strain vectors at load factor close to collapse

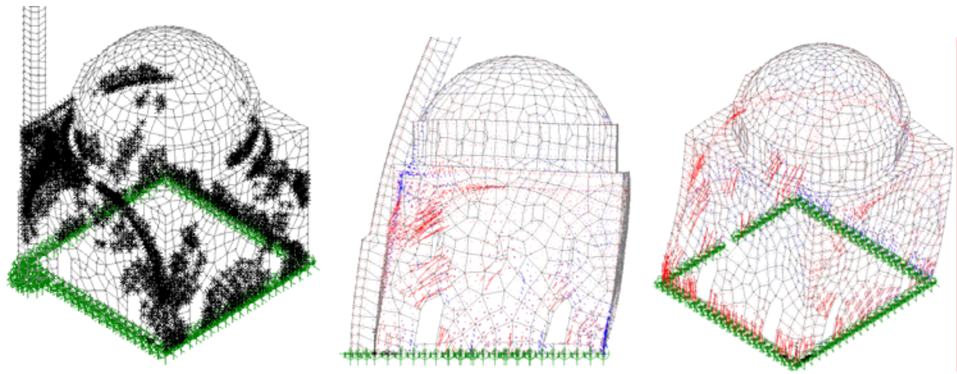


Figure 11. Two characteristic stages in cracks/yields development

FINAL REMARKS

The objective of these numerical analyses is to help in prediction of the large scale model response of Mustafa Pasha Mosque to shaking table tests. These pre-experimental analyses are to put some bounds and estimates for large scale model behavior in order to prevent and avoid any unexpected, premature heavy damages or even collapse of the test structure. Such numerical assessments of structural behavior would help in specifying locations for measurements of characteristic response values during shaking table test of the large scale model as well as in deciding about the choice of input time histories and its intensities in the experiment. It is expected that shaking table tests would globally confirm the pre-experimental numerical analysis results in such a way that large scale model damage propagation would be kept under control.

Many parameters are involved in controlling nonlinear dynamic structural behavior by the instruments of numerical analysis. Some of them are easily measurable with high reliability and certainty but some are difficult to quantify due to their scarcity even if experimental data exists. In the macro analysis approach, advocated in this study, additional difficulties arise in calibrating values for unified material characteristics that would represent the structure as a whole. This is actually a benchmark study whose validity has to be confirmed by the large scale model shaking table test that should be effectuate in the months to follow.

ACKNOWLEDGMENTS

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PAPERS

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ARCHITECTURAL HERITAGE OF SCHOOL BUILDINGS IN VOJVODINA IN THE CONTEXT OF NEW ARCHITECTURAL TENDENCIES

Summary: *There are more then 450 elementary and secondary school institutions in Vojvodina. Their architecture varies from late XVIII century house with strong Austro-Hungarian influence up to modern and post-modern buildings from the second half of the XX century. The central question that shall be discussed in this paper is the role of the existing school buildings in Vojvodina, in the context of new architectural trends in this field, which are, according to EU White Paper on Education and Training 1996, influenced by: the impact of the information society, the impact of internationalisation and the impact of scientific and technological knowledge. Different architectural typologies of school buildings, when analyzed according to these design factors, are expected to render different solutions. The purpose of this paper is not to propose the model, but to plot out the route to these new educational institutions by identifying the options available in education, training and new architectural concepts.*

Key words: *school buildings, school design, school architecture, EU White Paper on Education and Training, Vojvodina*

ARHITEKTONSKO NASLEĐE ŠKOLSKIH ZGRADA U VOJVODINI U KONTEKSTU NOVIH ARHITEKTONSKIH TENDENCIJA

Rezime: *Danas, 2006. godine, u Vojvodini postoji preko 450 školskih institucija osnovnog i srednjeg obrazovanja. Arhitektura ovih objekata (ukoliko su namenski građene školske zgrade) je veoma neujednačena, i u radu je predložena periodizacija na osnovu istraženih objekata od XVIII do kraja XX veka. Osnovno pitanje koje se razmatra jeste upotrebna i arhitektonska vrednost i mogućnost rekonstrukcije ili revitalizacije ovih objekata u kontekstu novih zahteva projektovanja školskih zgrada, definisanih u dokumentu EU White Paper on Education and Training 1996 kao: uticaj informatičkog društva, uticaj internacionalizacije i uticaj naučnog i tehnološkog razvoja. Osnovni cilj rada nije usmeren ka predlaganju konkretnih rešenja, već ka identifikaciji mogućih intervencija koje bi odgovorile na nove zahteve.*

Ključne reči: *školske zgrade, arhitektura školskih zgrada, arhitektonsko projektovanje školskih zgrada, EU White Paper on Education and Training, Vojvodina*

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1. INTRODUCTION

In the year 1996 the European Commission published its conclusions on education in the form of a document *White Paper on Education and Training: Teaching and Learning, Towards the Learning Society*, which made a major influence on principles of designing school buildings in the EU Member States [17]. This document emphasized the need for new Building Codes on this subject (on which ground all new school buildings will be founded and built), and, at the same time, forced all EU Member States to evaluate their existing school building according to these new recommendations. Finally, *White Paper* presented new concept of educational system, based on a term of *"lifelong learning"*, which established a new era in school system organization and, consequently, in school building architecture. It can be expected that this new educational philosophy will soon be introduced to the educational institutions of Republic of Serbia. The reason for this expectation can be found in a strong economic base on which the principle of *"lifelong learning"* has been defined, that sets the needs of today's global market as a primal influence.

The internationalisation of trade, the global context of technology and, above all, the arrival of the information society, have boosted the possibilities of access to information and knowledge for people, but at the same time have as a consequence changed work organization and the skills learned. This trend has increased uncertainty for all and for some has led to intolerable situations of exclusion. Education and training have now emerged as the latest means for tackling the employment problem.

One more reason to conduct research on educational architecture is the disproportion in scale of investment in school construction on international and local level. While in USA, on average, construction on two new schools begins every day [16], in Vojvodina only one school has been completed in the last year (since the beginning of the year 2005),[1] while many others are subjected to reconstructions and revitalizations, often regardless of the statement that ``bad school houses are silent killers of teaching and student learning"[15].

2. NEW INFLUENCES ON SCHOOL DESIGN

As this century draws to its close, we can see that the causes of change in society have been diverse and have affected education and training systems in different ways. Demographic trends have increased life expectancy radically changing at the same time the age structure of the population, thereby increasing the need for lifelong learning. The substantial rise in the number of working women has altered the traditional place of the family in relation to school and the upbringing of children. Expansion of technical innovation in all areas has generated new knowledge requirements. Consumption patterns and lifestyles have changed. In addition, people have been alerted to environmental problems and the use of natural resources and this has affected both education and training systems and industrial activities. **Three major, profound and wide ranging factors of upheaval** have emerged, however, which have transformed the context of

[1] According to the official data gained from Secretary for Education and Culture of AP Vojvodina

economic activity and the way our societies function in a radical and lasting manner, namely: the onset of the information society; the impact of the scientific and technological world; and the internationalisation of the economy [17]. These events are contributing towards the development of the learning society. They bring risks, but also opportunities which must be seized.

The construction of this society will depend on the ability to respond in **two important ways** to the implications of these events. The first response focuses on the need for **a broad knowledge base** and the second is designed to build up **abilities for employment and economic life**. Establishing the learning society will also depend on how **those involved and the institutions** in education and training pursue the developments already under way in the EU Member States [17]. *White Paper* considers that European society is in a transitional phase towards a new form of society beyond current short term forecasts.

During the 1940s and 1950s the notion of progressive education seemed to fit neatly with the utopian spirit of modern movement architecture with its emphasis on prefabrication and flexibility. Large areas of glazing often went hand in hand with inadequate levels of heating and ventilation to create extremes of temperature and environmental discomfort. Linked to the use of untested, lightweight factory technology newly introduced to the construction industry, problems within schools built at the time soon emerged [10]. Today, the quest to build environmentally sound new schools and to find ways of upgrading existing educational buildings to provide satisfactory level of comfort is one of the most important aspects of the school designer's remit. One of the major concerns lies in the need to improve existing buildings. How do architects rectify difficult conditions, without resort to total demolition of what are robust and serviceable structures? Often piecemeal strategies are adopted for upgrading those parts of the building fabric which can be readily replaced during the school vacations, for example: new high performance flat roof membranes; double glazed windows; window and door panels made with environmentally sound high performance reconstituted timber sections and the replacement of old uneconomical heating system.

2.1. Does the school design affect educational outcome?

It is often assumed that the quality of educational facilities makes no difference on bottom-line academic achievement. Many researchers and educational proponents now assert that school facilities are important to education. There are a number of excellent empirical studies of the explicit relationship between facility characteristics and educational outcomes [3, 8]. Looked at empirically, there is now considerable evidence that certain design characteristics like school size, classroom size, location, and the provision of secluded study spaces all make substantial differences in learning outcomes, and, in particular, that school size and classroom size make a difference in academic achievement.

Between the early 1960s and 1980, 344 articles were published pertaining to the effects of school size on academic achievement and other achievement-related variables [7]. Prior to the 1960s, many educators and policy makers believed that increasing the size of schools was an important reform idea. This belief led in part to comprehensive schools (large campuses from primary to pre-college education) in the UK and regional schools in the USA. In Vojvodina, usually only kindergarten and primary school were combined as one urban structure. Larger schools were more cost-effective and believed to be more educationally efficient. In the now-classic *Big School, Small School*, Barker and Gump [1] conducted a study of a sample of very big (over 2000 students) and very small (100-150 students) high schools in Kansas. They concluded, however, that small schools offered students greater opportunities to participate in extracurricular activities and to exercise leadership roles. In particular, participation in school activities, student satisfaction, number of classes taken, community employment, and participation in social organizations were all superior in small schools relative to large schools. A review of some of the subsequent studies appeared in the 1980 Journal of Youth and Adolescence [7]. Small schools (those on the order of 500 students) also have lower incidence of crime levels and less serious student. All the above findings relate to design variables other than achievement outcomes (lower incidence of crime levels, less student misconduct, greater participation in extra-curricular activities, etc.).

Other, more recent studies, however, have looked directly at the question of the impact of school size on academic performance. In a report written while at the U.S. Department of Education's Office of Educational Research and improvement, Fowler [6] argued that the issue of school size effects at the elementary school level, based upon “the number of students and the general agreement of the findings” is conclusive.

Many studies over the past ten years have looked at classroom size and classroom density and their impacts on educational outcomes. The results, in short, are that high density conditions have been found to lead to increased aggression, decreased social interaction, and non-involvement, all mediating variables. In classrooms with fewer students, teachers can have more interactions with each student, can provide a rich and vastly differing array of interactions, can establish learning centres, student learning teams, peer tutors and other instructional strategies, all of which improves the quality of interactions with each student. These effects may in turn lead to increased educational performance, though we know of no study testing this relationship empirically and directly. Teacher attitudes also improve as class size is reduced from 30 to 20. Students in small classes participate more than those in large classes [11].

Location of the school building is also proved to be of the greatest importance. Exposure to traffic noise at elementary schools has been associated with deficits in mental concentration, making more errors on difficult tasks, and greater likelihood of giving up on tasks before the time allocated has expired. Furthermore, children don't get used to noise, which influences their blood pressure [2]. While blood pressure, concentration, and task persistence are neither academic achievement nor prosocial outcomes, they are important mediators of educational outcomes [9]. The appropriate location of new schools and their proper design should be able to alleviate these noise-related problems.

How these principles are to be introduces in the school building heritage of Vojvodina is the main question of this paper.

3. SCHOOL BUILDING HERITAGE OF VOJVODINA

Since the future of many school buildings in Vojvodina lies in their reparation, reconstruction and upgrading, which is similar to many European countries [5], it is very important to develop a precise knowledge of architectural heritage of school buildings in this Province and how it can be upgraded to meet educational needs of modern society, especially in the sense that " the penalty society pays for forgetting the past is to lose a common heritage of bearings and reference points"[17].

The survey that has been conducted on Vojvodina's school building heritage was in accordance with basic principles of, by far, the most comprehensive historical investigation of school architecture which has been published by Seaborne [12] and Seaborne and Lowe [13]. These studies examined English school buildings from the educational as well as the architectural point of view. ``Even those schools with little or no architectural merit are often important from the educational and sociological points of view" [12]. As school curriculum changes and educational reform creeps incrementally through schools, school organization, administration, and physical structure also change, albeit at a seemingly slower pace. ``Similarly, the ideas of educational reformers, if they are really to take effect, must sooner or later be expressed in organizational and architectural terms. Changes in the curriculum and methods of teaching are also likely to be reflected in the layout of buildings and the arrangement of classes" [13]. In this sense, school buildings in Vojvodina can be divided to periods of development according to the following historical events:

- period before intensive migration of people from Serbia to Vojvodina (**before 1690**), when schools were belonging exclusively to the monasteries (clerical schools);
- **from 1690. to the beginning of the XVIII century**-schools are not perceived as an autonomous architectural typology, they are found in the individual houses of teachers;
- **from the beginning of the XVIII century until 1880s**-in this period typology of elementary (trivial) school has been formed, with one or two classrooms and teachers dwelling;
- **from 1880s to year 1914.**-period of intensive large-scale building of secondary schools with major Austro-Hungarian influence (schools are beginning to have public-building-character)
- **period between two World Wars**-architecture of schools is divergent, academic styles are still present (modern style buildings are beginning to occur in Vojvodina);
- **from 1945. until 1980s**-schools are being built in the periphery of cities in late modern style, usually with the character of administrative building;
- **1980s**-new building code for school buildings is being introduced in Vojvodina, schools gain more human scale and architecture;
- **1990s until year 2006.**-schools are mainly being reconstructed and revitalized.

4. ARCHITECTURAL HERITAGE OF SCHOOL BUILDINGS IN VOJVODINA IN THE CONTEXT OF NEW ARCHITECTURAL TENDENCIES

On the basis of previously stated classification, solutions for existing buildings that can be routed are:

1. Rural schools that have been built until the beginning of the II World War and made from mud and chaff (straw) and unfired bricks have been influenced with humidity and water, and very often ask for immediate reconstruction, as a key-stone of architectural and educational heritage. Since they are equipped with only few classrooms, their upgrading to larger institutions can be made only by principles set in pavilion school typology with closed corridors. Small classrooms, natural surroundings and location should be preserved as their main quality;
2. Secondary school buildings, built between 1880s and 1930s in historical styles, present today very important urban landmarks in Vojvodina cities and can meet many of contemporary educational needs, except the lack of social spaces inside the schools and spaces that can provide access to new informational technologies. Their location usually lacks in greenery and space, and their classrooms are often large;
3. Modern schools from 1950s, 1960s and 1970s usually lack in thermal- and hydro isolation and their materialization and interior decoration is often very simple. If revitalized, they can be transformed into institutions of much greater educational potentials, since their physical capacities are often very generous;
4. Finally, location of post-modern schools from 1980s, and different architectural approach that has been conducted, ask also for a different treatment of open and public spaces and better integration with urban or rural surroundings (in this sense, they are very similar with late modern schools).

Picture 1. Possibilities of redesigning schools: two modern secondary schools in Vojvodina today (left:Ruma, middle:Bačka Palanka) and example of redesign from Netherlands (right: De Stern en Merkelbach school, Amsterdam)

In all cases that have been briefly analyzed, introduction of basic principles of sustainable architecture is necessary, in all means possible. The importance of this approach to revitalization of schools is underlined with educational requests, since "environmental education can be considered to be a part of its effective implementation"[14].

5. CONCLUSION

Should money be spent on rehabilitating turn-of-the-century buildings and on deferred maintenance? Or should money be spent on new facilities? If school buildings are renovated, or new ones built, can they be more responsive to new ideas in education? Can they aid improvement of instruction and the improvement of academic performance?

There are two different ways to approach this issue: the development of patterns and design guidelines based either on translation of empirical research or on extrapolations from educational reform ideas in combination with the practical experience of educators. The first way is to "translate" the empirical research literature on the effects of school buildings on educational performance into research-based design guidelines, patterns, or design principles, and then work to implement those design guidelines in new and renovated school building projects. This is an inductive, inferential, inherently creative process.

The second, and still scientifically acceptable way if it is done with humility and caution, is to extrapolate from educational reform ideas or the experience of reflective educators in order to give these ideas architectural form. What is meant by this is to take an educational idea—like the notion of site-based management—and ask what characteristics, if any, of buildings might assist in achieving this idea. In the absence of empirical evidence, we cannot say such inductive architectural principles will for sure improve performance, only that they might.

The value of architecture for its own sake is rarely linked to positive educational outcomes, although many authors have proved this statement to be wrong. It is more readily identified with financial profligacy. The values that bring this about are clearly entrenched within the social framework and may take generations to transform. Schools in Vojvodina appear to be profoundly introspective institutions, and it is clear that until the school becomes more user-friendly and gains value in the minds of the majority, poor funding ad poor quality will go hand in hand.

Outside the classroom that features which encourage good social interaction between differentially aged children can be very positive. Inhibiting the sense of freedom for reasons such as health and safety, fears of bullying and difficulties of control, are the greatest limitations to the well-balanced school enviroment [5]. School students, like any community of people, learn as much from each other outside the classroom as they do from their lessons [5]. This notion of community was central to the original thinking of the early educators, such as John Dewey [4].

Good school architecture can significantly enhance the experience of education. In practical terms this does not simply mean the provision of minimal comfort conditions, which is often the case in Vojvodina. It requires architects to go further. Providing pleasant social areas, encouraging meaningful social interaction between students and their teachers, integrating schools in the community and developing mutually supportive structures, introducing new information technologies, profounding the sense of spatial awareness and sustainable development have become crucial principles of school design.

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TENTH NATIONAL AND
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SUSTAINABLE DEVELOPMENT IN THE DOMAIN OF CONSTRUCTIONS OF THE STEEL STRUCTURES BY A CORRECT MANAGEMENT OF THE CORROSION OF METALS

Summary: *The sustainable development concept studies the an-tropic influence upon the causes and consequences of the decay of the environment and the long term directions of researching directions, considering: social equity, environmental protection and economic efficiency. Corrosion decay cause economic losses and shortens the service life of constructions. The maintenance costs are high and if the protection of the surfaces is not insured, the pollution of the environment increases.*

Modern industry manufactures high strength steels with high anti-corrosion protection levels. Costs increasing, scientific research in this domain will adopt certain criteria of anti corrosion protection, dictating upon the system of protection itself. The Romanian design codes specify standard steel brands elaborated based on the corrosion classes. Protection methods and preventive strategies are the result of corrosion management and control starting with laboratory tests and simulations.

New materials and technologies used for the construction of steel structures and the evolution of the environmental conditions towards ever increasing diversity as well as aggressiveness need complex testing. Qualified personnel and upgraded testing equipments and modern data analyses are necessary, but the scientific research must develop first of all.

Key words: *sustainable development, corrosion protection, management of corrosion, corrosion classes and factors.*

ODRŽIVI RAZVOJ U DOMENU GRAĐENJA ČELIČNIH KONSTRUKCIJA UZ ADEKVATNO UPRAVLJANJE KOROZIJOM METALA

Rezime: *Koncept održivog razvoja značajno utiče i prouzrokuje konsekvence u smislu propadanja životne sredine. U tom smislu razmatraju se društveni i ekonomski aspekti zaštite životne sredine. Korozija prouzrokuje velike ekonomske gubitke kroz skraćenje eksploatacionog veka objekata. Koštanje održavanja je otežano jer zaštita površina je nedovoljna jer je životna sredina sve zagađenija.*

Ključne reči: *održivi razvoj, zaštita od korozije, upravljanje korozijom, klase korozije i uticajni faktori.*

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1. INTRODUCTION

Modern concepts of quality and performance in the construction field comprises the sustainability criteria as a basic necessity. Whenever the exploitation conditions are severe, along with initial demand of superior technical characteristics of the material the manitenance unaltered during all the service life of the construction is imposed.

Promotion of a sustainable development asks extensive knowledge of the environmental conditions and of all the factors that lead to its decay or, on to the improvement of its status. The total energy consumption is ever greater and the demands of water and other irreplaceable resources are continuously increasing. Studying the human influence upon the causes of the environment decay and the consequences are the first steps of the future society considering the relationship between social equity, environmental protection and economical efficiency, figure 1.

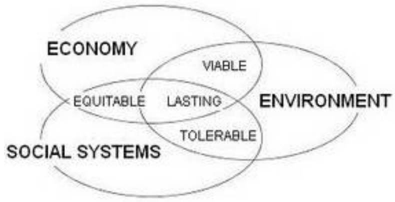


Figure 1 Relationship between the poles of the sustainable development of the human society [7]

An essential objective consists in extending the knowledge of the bounds between the human society and the environment in order to master the techniques of predicting the an-tropic impact and it is reached by:

- Developing the research in the domain of Earth's sciences to harmonize the evolution of the society with natural tendencies of the planetary changes;
- Developing and customizing new techniques of analysis and prediction for an accurate evaluation of non adequate human interventions that alter the natural systems;
- Integrating the physical, economical and social sciences in the domain of social behavior that affects the environment.

The prediction capacity of the response of different ecosystems (terrestrial, marine, biologic etc.) to various perturbations allows the design of restoration measures and also a general monitoring.

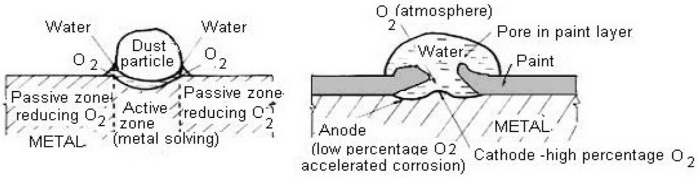
2. DISTRUCTIVE EFFECT OF CORROSION OF METALS

Corrosion consists in the attack of an aggressive environment upon a metallic material. The distructive action of corrosion is mainly the cause of severe ecconomic consequeces materialized in physical losses, reduced strength capacities and diminish of the security in exploitation and of course, the service life. Maintenance becomes more and more expensive and secondary effects occur, like impurification of the products, pollution etc.

The protection measures against corrosion vary, their choice being almost always the result of a technical, economical and even ecological compromise. The identification of this phenomenon becomes an ever more serious problem of the modern society, the annual total losses being estimated at hundred of billions dollars.

As the major part of metallic structures (almost 80%) is placed in atmospheric conditions, a wide field of investigations is developed in the domain of atmospheric corrosion. The most aggressive agents are the *air* (oxygen, carbon dioxide, nitrogen oxides and sulphur dioxides etc.) containing also solid particles (soot, dust etc., fig. 2.a) and *water* from rainfall and condense (fig. 2 b).

Almost always the atmospheric corrosion follows the pattern of an electrochemical mechanism, very thin water layers having the role of electrolyte; the lack of humidity prevents this phenomenon from taking place. In this respect several archeological researches put into evidence the entirely preservation in dry atmosphere of iron artifacts dating from 1500-2000 years ago.



a. **b.**

c.

Figure 2. Scheme of the atmospheric corrosion phenomenon: a. – unpainted surfaces, b. – paint surfaces, c. –atmospheric corrosion on a metallic surface

The corrosion process is fed by several other elements whose presence in the atmosphere is generally inevitable: gases, metallic oxides resulted from corrosion and different impurities deposited on the surface of the metal like dust, salts, acids etc. These impurities facilitate the start of aeration or *differential concentration electrochemical corrosion* for which these impurities are micro-cathodes and the metallic surface is the anode, destroying itself.

Relative humidity of the air influences substantially the value of the losses due to corrosion. The loss of metal for steel exposed to impure atmospheric conditions is put into evidence in the graph in figure 3 (the pollution factor is SO_2 with a proportion of 0.01%).

The surface of the metal with the impurities gathered there becomes a hygroscopic mixture absorbing the moisture from atmosphere and keeping it at higher levels than the external relative humidity. Temperature variations speed up the corrosion in the presence of solid pellicles that may crack. If an increase of temperature is followed by drying of the metallic surface, the electrochemical process is slowed down.

3. FACTORS THAT INFLUENCE THE CORROSION PHENOMENON

The influence of chemical, physical and mechanical properties depend in each case on the way the metal is chosen and used. Structural steel is classified in specific categories according the exploitation conditions. In table 1 this aspect is put into evidence with the specification of the standard regulations that govern this classification.

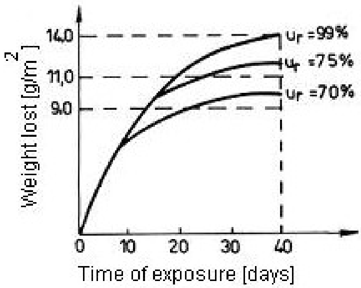


Figure 3. Influence of relative humidity of the air upon the losses due to corrosion

An accurate equivalence between the products specified in EUROCODES and the Romanian standards is almost impossible considering the diversity of steel grades and qualities, e.g. the correspondence between the Romanian standard STAS 500/2,3 and the European standard validated on Romanian market, SR EN 10025+A1.

In the present the trend points to steel brands with high mechanical strengths and superior resistence to corrosion in the domain of constructions. An important energy consumption is implied in the production of these steels with superior resistance to corrosion and so to speak, these materials situate themselves at high levels of energy. The corrosion phenomena reduce the metals at lower levels of internal energetic equilibrium, in their initial status of oxides, clorides, sulphurous components etc.

	STAS 500/1,2,3	**SR EN**	
Types of structural steel	General steel grades, carbon steel and low alloy steels	SR EN 10025	Hot rolled sections made of basic steel and quality steel
	Steel with increased strength for atmospheric corrosion	SR EN 10155	Steel with higher strength for atmospheric corrosion conditions
	Steel with fine granulation for welded structures	SR EN 10113	Hot rolled sections made of weldable structural with fine granulation
	Steel with resistance to brittle fracture at low temperatures		
	Steel with very high yield limits	SR EN 10149	Hot rolled plates made of steel with high yield limits for cold forming

Table. 1. Structural steels

Professor Staehle [6] classifies the influence of specific corrosion factors upon the whole phenomenon in the following:

- *Material factor* (figure 4) is a result of several compromises: for ex., the technical evaluation of an alloy is the compromise between the resistance to corrosion and other mechanical properties like strength and weld-ability, finally a compromise between the technical competence and economical premises.

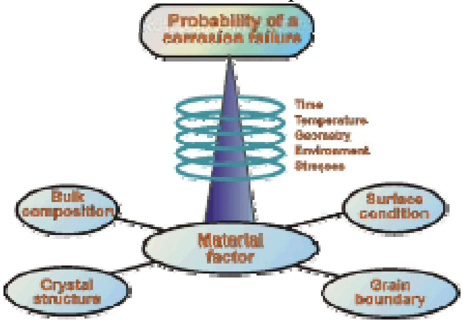


Figure 4. The material factor controlling the probability of a corrosion failure [6, 6]

- *The environmental factor* with multiple influences (see figure 5). An accurate and thorough definition of the contribution of the corrosion phenomenon must include the description of the local environment in contact with the metallic surface, mostly the humidity and drying alternation, the impurity deposits and modifications of the flow pattern, all these considerable influencing the chemical dynamics on the metallic surface
- *The stress factor* influences by value (mean, maximum and minimum), constant stress value, stress concentrators, bi-axial stress of I, II and III type, cyclic frequency, wave shapes. The source of these stresses is the whole ensemble of actions on structures, residual stresses and other types of actions (thermal dynamic action, vibrations, pressure, rotations etc.).

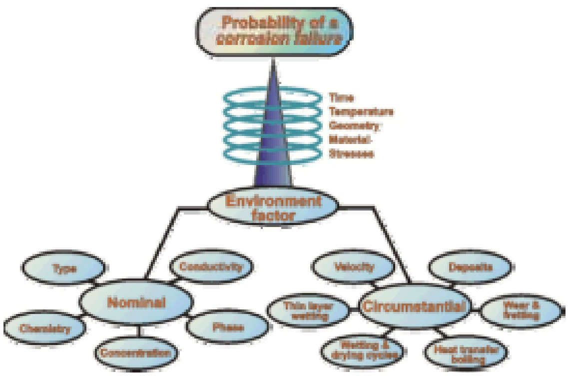


Fig. 5. The environment factor controlling the probability of a corrosion failure [2, 3]

- *Geometry factor* is present due to the discontinuities in the material structure intensifying the stresses due to the generation of galvanic potentials, chemical crevices and geometrical restraints that determine corrosion concentration areas.
- *Temperature factor* with a negative influence due to the increase of electrochemical reactions speed under the growth of temperature, almost doubling it for every 10˚C growth. Corrosion under stress of mild steel due to the presence of nitrates is not probable at 20˚C, but it will certainly happen at

100˚C. In practice, the thermal transfer appears at the interface between the metal and the solution.

- *The time factor* will increase the complexity of the chemical alterations between the crystals and on the surface of the deposits, the chemical strength and the thermal transfer, finally putting in evidence the external faults (corrosion and erosion).
- The *human factor* due to errors that influence negatively the corrosion process. Based on different studies, the following human errors were put into evidence: operational, from the design, of maintenance, of fabrication, of installation of equipment and devices and of surveying.

4. CRITERIA OF CHOOSING ANTICOROSION PROTECTION SOLUTIONS

The development of the industry in the structural steel production field finds itself in a continuous “competition” with the evolution of the aggression parameters of the corrosive environment. New materials and technologies appear with superior characteristics and high resistance to corrosion but in the same time, new more complex and more aggressive environments are identified.

As a result of this competition between the protection against corrosion and the corrosion phenomenon itself, the simple and “easy to make” solutions are not efficient anymore, more specific and sophisticated measured imposing themselves.

Because the corrosion phenomena are produced by various environments anticorrosion, protection methods and technologies will vary also, some generating independent domains of study.

In the present, prior after the determination of the importance category of the construction will be the very determination of the corrosive class of the environment where the construction is placed and exploited (table 2), with all the necessary specifications (difference between the internal and external corrosive conditions, between the industrial environment and other types of surrounding conditions) as well as the specific corrosion sensitivity of different elements or of the whole construction.

The factors that influence the durability of the protection systems are:

- Structural concept,
- Quality of the protection materials,
- Ititial staus of the protected surface,
- Efficiency of preparing the surface for protection,
- Limit conditions,
- Quality of the treatment and the exposing conditions after the treatment.

The principal criteria regarding the choice of the protection solutions against corrosion are the following:

- Correlation between the characteristics of the materials implied and the corrosivity class taken into account;
- Insuring the anticorrosion protection in a certain class of corrosivity for a wider variety of the nature of the environment;
- The durability of the systems and products used for the anticorrosion protection;
- Simplification of the framework of verifications for the quality of execution and maintenance of these systems and products;

- Ratio efficiency/complexity;
- Accessability;
- Ratio betwen the execution in workshop and at the building site during the mounting stage, mostly from the point of view of insuring the quality;
- Ratio efficiency/total costs;
- Ratio initial costs/ maintenance costs.

Corrosion classes according to SR ISO 9223 and SR EN ISO 12944-2	Corrosion speed [µm/year] according to SR ISO 9223 and SR EN ISO 12944-2	Aggresivity class (STAS 10128)[1]	Corrosion speed [µm/year] (STAS 10128)
C1 weak	≤1.3	**1m**	<10
C2 mean	1.3...25	**2m**	11...100
C3 high	25...50	**3m**	110...500
C4 very high-industrial environment	50...80	**4m**	>500
C5-I,M very high-marine	80...200		-

Tab. 2 Equivalence between the corrosion classes and aggresivity classes of the steel grades

The durability of the the anticorrosion protection layers on the steel surfaces are classified in three classes:

- **L** – limited durability: 2...5 years,
- **M** – medium durability: 5...15 years,
- **R** – high durability, over 15 years.

The protection method will consider the maximum efficiency based on a technical-ecconomical study considering the real conditions of the steel structure and the implication of the costs for protection upon the total costs of the construction. In the case of industrial activity, the implication of these extra costs on the estimated values of the finite products are also studied.

5. MANANGEMENT METHODS OF THE CORROSION OF STEEL STRUCTURES

All the criteria mentioned in the above chapter emphasize the complexity of the corrosion phenomenon. Its multiple interfering factors lead to very specific manifestations, imposible to be generalized so the solutions adopted for the anticorrosion protection must be the result of a systematic best organized approach.

Current studies put in evidence the fact that technical progress finds and offers multiple methods of decelerate or even stopping the process of corrosion; it also gives efficient ways of management of this phenomenon. Still, a good management of the corrosion may be conducted only if preventive strategies are applied, these implying:

- Be more aware of the high costs level of the corrosion events and the potential economies resulting from a corrosion management,
- Change the false idea that the corrosion is implacable,

- Change the politics, the regulations and the standards also the procedures of management for increasing the economies,
- Improvement of the education and training of the personnel;
- Use of practical methods and experience accumulated in the design domain,
- Use of the science progress for improving the methods of evaluating, prediction and execution.

6. CONCLUSIONS

A modern understanding of the corrosion phenomenon puts into light the wide dimensions of the decay of materials in almost every domain and sustained efforts are done for the continuous development of new techniques and procedures for the control of the corrosion phenomenon.

The adoption of the solutions for the anticorrosion protection is the result of an organized, systematic approach imposed by the concept of management of corrosion.

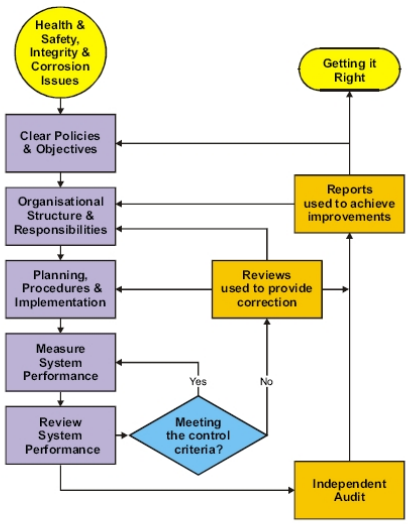


Fig. 5. *Flow chart of the management of corrosion [6]*

A good management of corrosion may be developed by appealing to the adequate prevention strategies and planning and putting in practice this management must insure the elimination of decay or at least their reducing at a significant degree.

The solutions for anticorrosion protection become specific for a certain situation, their adoption and the technology applied needing high trained personnel with a vast experience in the corrosion domain.

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DEGRADATION OF A REINFORCED CONCRETE SILO UNDER SALTS ACTION

Summary: *The paper analyses the degradations that occurred to a silo for re-crystallized salt, due to corrosive action of salt. The silo, built in 1970 has a frame reinforced concrete structure on four levels, with floors of reinforced concrete. Due to some specific causes (corrosive action of salt, reduced depth of concrete cover, unprotected structural and non-structural elements, delay of maintenance and repair operations), the cover concrete and the steel reinforcement were destroyed, making dangerous the service security of the silos. Analyzing the degradations and having in view the rehabilitation of the structure to ensure security exploitation the authors proposed some measures: rehabilitation of foundations, coatings of columns and walls, injections of cracks, rehabilitation of floors, and application of protection layers.*
Key words: *reinforced concrete, salt, corosion, silo.*

DEGRADACIJA ARMIRANOBETONSKOG SILOSA USLED DEJSTVA SOLI

Rezime: *U radu su analizirana oštećenja koja su se pojavila na silosu za re-kristalisanu so, usled korozionog dejstva soli. Silos, koji je izgrađen 1970 godine, ima ramovsku armiranobetonsku konstrukciju na četiri etaže sa sa međuspratnim tavanicama od armiranog betona. Usled određenih razloga (koroziono dejstvo soli, smanjena debljina zaštitnog sloja betona, nezaštićeni konstruktivni i nekonstruktivni elementi, odlaganje održavanja i sanacionih radova), zaštitni sloj betona i armatura su bili uništeni, predstavljajući opasnost za bezbednu eksploataciju silosa. Analizirajući oštećenja i imajući u vidu sanaciju konstrukcije kojom bi se osigurala bezbedna eksploatacija autori su predložili neke mere: sanaciju temelja, zaštitne slojeve za stubove i zidove, injektiranje pukotina, sanaciju međuspratnih konstrukcija i primenu zaštitnih slojeva.*
Ključne reči: *armirani beton, so, korozija, silos.*

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1. INTRODUCTION

One of the most important problems for the construction patrimony is that of rehabilitation of buildings, that are damaged by different causes like: seismic actions, aggressive agents or that are not corresponding as functionality and destination.

The complexity of aspects that must be considered when a damaged building is consolidated can be increased by other facts such as: missing of technical documents of the structure, the difficulties connected to the investigations for establishing the physic-chemical characteristics of component materials, of foundation soil or the type and degradation state of infrastructure. During the analyze process for establishing the best solutions of rehabilitation must chose the right method of investigation and also to give the most secure and economical solution of rehabilitation.

The paper presents the aspects of a study of rehabilitation done for a silo damaged by salt action.

2. EXPERTISING METHOD

According to Romanian standards concerning the quality in construction [1] all works of repair and rehabilitation of structures must be effectuated on the base of an expertise done by technical experts.

The investigation methods were established by experts having in view that the building is situated in aggressive environment and there were:

- qualitative evaluation method
- evaluation method by simplified computation of resistant capacity
- evaluation method by non-destructive tests

The criteria for evaluation of seismic protection level for the silo are given by:

I. - execution period for the silo: 1968-1970;
II. - number of levels: 5 levels
III. - structural system: walls of reinforced concrete that support on frames with columns and transversal beams; floors at +3,05 m, +16,15m and +19,30 m with longitudinal beams that support on transversal beams; the walls of masonry on slabs.
IV. - the class of importance of the building: class III;
V. - seismic zone for computation: E.

Having in view the upper criteria the following mthods of investigation were adopted:

(1) qualitative evaluation - E1 (§11.3 from P100-92);
(2) analitical evaluation by computation - E2a (§11.4.1…11.4.9 from P100-92);

According to norms P 100-92, the qualitative evaluation was done by examining the construction in place (the structure and the non-structural elements).the degradations are presented in the next chapter.

By analitical evaluation of the silo there were determined the maximum stresses (bending moments – M, shear force – T and axial force – N) for the columns and beams, considering the structure as new, in the conditions of determining the loads and the stresses according to the new standad. These stresses correspond to conventional seismic load (S_{nec}) defined by the provision P100-92.The ensuring degree to seismic action „R" is defined by :

$$R = \frac{S_{cap}}{S_{nec}} \quad (1) \qquad (1)$$

S_{cap} – capable conventional seismic load of the construction;

S_{nec} – design conventional seismic load determined considering the construction as a new construction.

The computation was for fundamental group of loads, with maximu gravitational loads and special group of loads with long term gravitational loads and seismic action (according to STAS 10107/0-90).

3. STRUCTURAL DESCRIPTION

The salt silo from Salt Factory is an industrial building, rectangular in plan (26.97x5.75 m), is developed on 4 levels (groundfloor + fist floor + silo + technological floor).

Figure 1. General view

The resistant structure of silo is realized of reinforced concrete frames on two directions. The slabs are of reinforced concrete. The silo cells have structural walls of reinforced concrete; there is not internal partition among the cells. To the techological floors there are masonry walls. The roof is of reinforced concrete floor with two slopes.

The materials quality was determined by non-destructive tests with Schmidt hammer.

4. STATE OF DEGRADATION

One The degradations were produced because of specific causes, such as:

- agressivity of environment due to technological process, pickling vapours or combination of pluvial water and salt;
- the deterioration of protection layer of structural elements or a too small cover of reinforcement.

As general phenomenon it can observe that the concrete cover is destroyed and the steel is partialy or totaly damaged by corrosion.

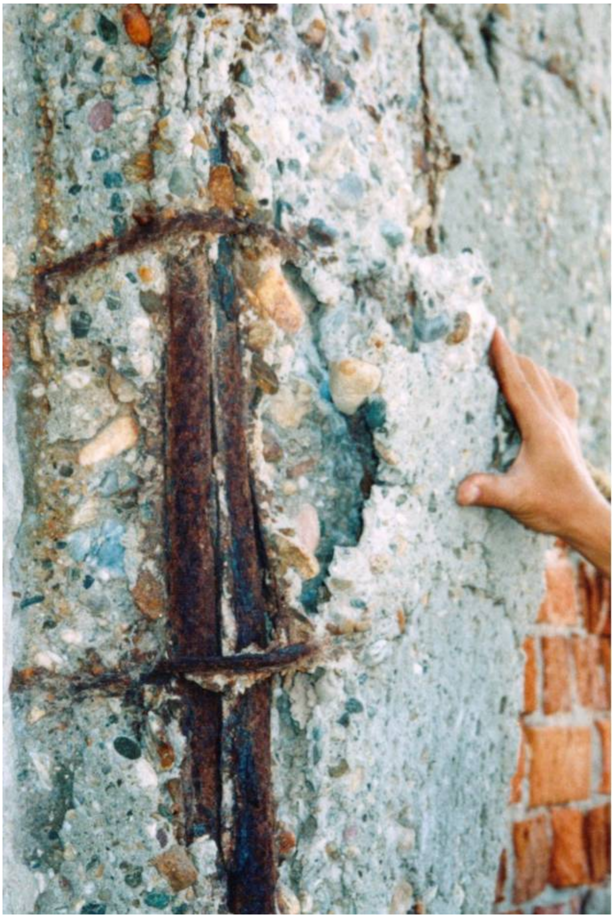

Figure 2. Degradated column

To the columns there are major degradations of columns, that can result in the collaps. Also there are places where the bond between concrte and steel is destroyed.

Figure 3. Cracks on transversal girders

Figure 4. Degradation of the walls silo

Figure 5. Degradation of the floors

In beams there are cracks with big openings, that allow the access to the aggresssive agents.

The silo walls present a advanced state of degradation on exterior side, with steel corrosion, concrete cover is also completely destroyed.

The degradations are due to the aggressive action of salt vapours because of dissociation of salt and presence of clor ions, that diffuse in the concrete mass and react with steel, resulting its corrosion.the reaction is with volume increasing, so expansion of volume will result in the cracking of concrete cover.The steel corrosion is favourized by the porosity of concrete that has increased in time because the reaction between th calcium hydoxide and clor from the salt, resulting the calcium cloride, that is soluble and is levigated by meteirical water.

From visual analyse of floors it resulted an advanced state of degradation: conrete cover is destroyed and the corrosion of steel.

5. THE RESULTS OF ANALITHICAL METHOD (E2a)

After establishing the loads the statically computation was done on a 3D model. The analysis was by the method of finite elements, considering two directions of seismic action.

After the determination of bending moments on both directions it results that the structural columns will resist to specific loads because the great number of plastic articulations that transform the structure in mechanism. Also in computation there was not considered the degradation of concrete section.

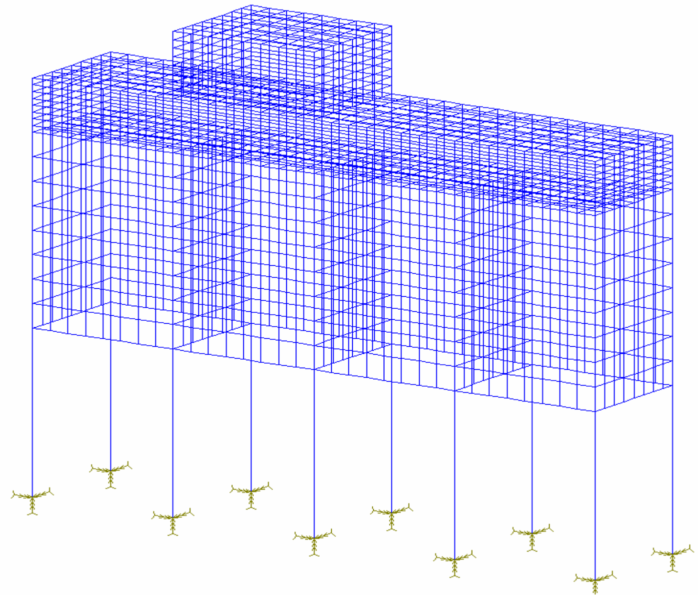

Figure 5. Degradation of the floors

6. CONSOLIDATION SOLUTIONS

The technical rehabilitation of the silo for ensuring a protection level according to Norm P100-92 must consider must be realized because the ensuring degree does not accomplish the necessirties given by actual norms.The consolidation solutions that are reccommended for structures of reinforced concrete frames with reinforced concrete floors must consider the chemical aggressivness in service.The Norm P100-92 considers that in expertize there are at least two solutions:

a) minimal solution, that avoids the collaps; it must improve the seismic protection by coatings with reinforced concrete to frames and diafragms, that must be capable to take over and transmit the horizontal forces; rehabilitation of foundations; also the cracks in beams or floors must be injected and the plaster must be replaced in destroyed zoned; aplication of protection layers.

b) maximal solution, capable to guarantee the new requirements; in this case there are two situations that refer to the partial or total demolation of silo. In the partial solution of demolation the walls of silo that are replaced with wood walls of small height. The total demolation of the silo can be considered if it is justified from economical reasons.

7. CONCLUSIONS

Having in view the nature of degradations and causes that produced them, the silo does not present security in service and there are necessary important intervention measures for consolidation. The rehabilitations solution recommended in the expertise must be established in the designing project in accordance with designer and beneficiary and must be approved by expert.

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TENTH NATIONAL AND
FOURTH INTERNATIONAL
SCIENTIFIC MEETING

iNDiS 2006

NOVI SAD, NOVEMBER 22-24,2006.

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THERMAL REHABILITATION OF BUILDINGS FROM THE SANITARY FIELD CASE STUDY

Summary: *The buildings from the public health system in Romania mean a fix patrimony of an important diversity as structure, operation conditions and age. Most of them, with an age more than 50 years, show a deficit from an energetic point of view, being characterized by high energy consumption for meeting the sanitary, hygienic and comfort requirements. This means not only high operation expenses, but also it enables the development of some pathogenic factors that can affect the health of the employed people and patients. Now, it is developed a rehabilitation program of an important number of such buildings that has in view the achievement of the quality criteria of the inside environment with low energy consumption. The paper presents the results of a study concerning the thermal rehabilitation of a hospital built in the 70's. It has in view: the analysis and diagnosis of the present state, the assessment of the energy consumption and of the quality criteria fulfilment of the inside environment, the strategies of thermal rehabilitation, the impact of the rehabilitation strategies one environment quality and energy consumption. The choice of a certain strategy has also in view, beside the quality environment and energy efficiency, the economic criterion.*

Key words: *energy consumption, thermal rehabilitation, indoor environment quality.*

THERMALNA REHABILITACIJA SANITARNE ZGRADE - STUDIJA SLUČAJA

Summary: *Zgrada javog zdravstvenog sistema u Romunji osrednje vrednosti ali različitih značaja objekata stanjA i uslova eksploatacije koji su zastareli. Većina od njih je starija od 50 gorina, pokazuju deficit sa energetske tačke gledišta, karakterišu ih veliki utrošci energije da bi ispunile sanitarne, higijenske i uslove komfora. Ovaj način nema samo manu velike cene koštanja rada, već takođe može dovesti do patogenih parametara sa negativnim uticajem na zdravlje zaposlednih ljudi i posetilaca. Napravljen je program rrehabilitacije zna;ajnog broja ovih zgrada koji uzima u obzir dostignu'a kriterijuma kvaliteta okru/ja-sredine sa malim utroškom energije. U radu su izneti rezultati proučavanja koja se odnose na termičku rehabilitaciju bolnice građene 70-tigh godina prošlog veka. Obuhvaćeni su: analisa i dijagnoa postojećeg stanja, procena utroška energije i kriterijumi kvaliteta za ispunjenje uslova spoljne sredine, strategija termičke rehabilitacije, kao i uticaj strategije rehabilitacije, kvaliteta sredine i utroška energije. Izbor odgovarajuće strategije je takođe uzeta u obzir, kvaltet sredine i energetske efikasnosti, kao i ekonomski kriterijumi.*

Ključne reči: *utrošak energije, termička rehabilitacija, kvalitet spoljne sredine.*

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1. INTRODUCTION

Recent studies show that buildings are responsible for 45% of the CO_2 emissions in the air and that they are determined by the energy consumption resulted from the fossil fuels burning. The buildings dedicated to health caring, in general, and especially the hospitals buildings have an important role in this context, in accordance with the information offered by the Agency Commercial Building energy Consumption Survey

The same source of information noticed that, even though that at the most monitorized public buildings, it was recorded a decreasing of the energy consumption with 11%, in hospital buildings is recorded an increasing with 15% in 2004 than 1999.

In Romania, the buildings for the public health sector show a deficit from the energy efficiency point of view. There are situations when it is not possible to ensure the most propitious comfort and hygienic conditions even with high energy consumptions. In the spirit of the European direction concerning the energy conservation, it was started an ample rehabilitation program that has in view:

- the increase of the envelope thermal insulation:
- the improvement of the heating/air-conditioning equipments;
- the improvement of the lightening systems;
- the solar gains use;
- in this program it were included also 2 buildings sections of the County Hospital from Bacău, that were the subject of an energetic audit with the aim of an analysis of the actual situation and the elaboration of a rehabilitation strategy.

2. THE GENERAL PRESENTATION OF THE BUILDING

The studied building was built during the 1972-1974, taking part of a hospital ensemble with many pavilions. The building has 2 sections, perpendicular on each other, with ground floor and 5 floors, and ground floor and 6 floors, separated by a settling joint The side façades have the main body with a Northern and Southern orientation and are made of concrete diaphragms of 15cm thickness and brickwork veneering of 13cm thickness. The closing on the longitudinal façades, toward East and West to the main body, respective North and South to the connection body, is achieved by unloading façade panels, already finished, with 30cm thickness, made of two concrete layers of 7cm and 16cm light cellular concrete.

The roof of terrace type has a thermal insulation of light cellular concrete of 12cm thickness.

The windows are made of wood carpentry, with two coupled frames and glazing of 3mm thickness. The access doors are metallic, of steel profiles.

The heating equipment is of bitubular type, with hot water 95ºC/75ºC with forced circulation and inferior distribution.

3. THE PRELIMINARY BUILDING INVESTIGATION; THE QUALITATIVE DETECTION OF THE THERMAL IRREGULARITIES IN THE BUILDING ENVELOPE USING THE IRTHERMOGRAPHY METHOD

The preliminary investigation of the building consists in the technical documentation analysis on which is based the building, in the visual analysis, emphasizing the characteristic elements, and in the qualitative detection of the thermal irregularities by IR thermography.

The visual building analysis do not emphasized fissures, degradations or areas strongly affected by moisture, the envelope elements being in a good state.

The thermographic analysis was made with the following aims:

- to estimate the concordance between the information from the existent documentation and the building present state having in view the homogeneousness of the thermal insulation materials, determined also by the changing with the time passing of the materials thermal characteristics;
- to assess and localize the thermal losses through the thermal insulation and by untight areas of the walls, roofs, doors and windows.

The thermographic analysis of the surgical pavilion was made during the day of 18th March 2006, with a closed sky and exterior temperature of +4.5ºC. Inside the hospital wards, the interior air temperature had values between 22ºC and 24ºCand in the halls approximately 22ºC.

The examination of the IR images gives information concerning the temperature profile on the façades areas determined by the lack of homogeneousness of the thermal insulation and emphasizes the following aspects:

- the linear thermal bridges determined by the floors presence, the crossing between the diaphragms on two directions and the unfavourable effects of the concrete consoles that are supporting the façades elements (brise-soleil) (figure 1)
- the influence of the finishing state on the temperatures field and the heat loss at the socle level (figure 2)
- the positive effect on the thermal protection level get by the wooden carpentry with ordinary glazing replacement with PVC frames with thermopane glazing.

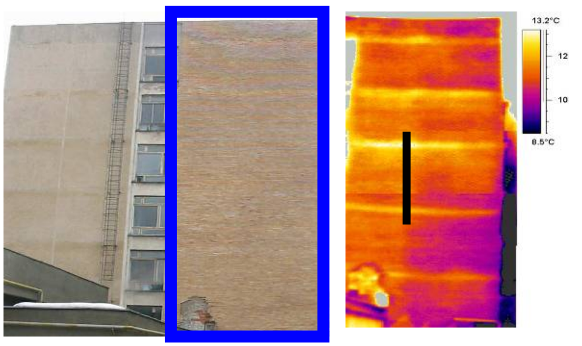

Figure 1. IR image, Northern façade, on the right. There are emphasized the thermal bridges created by the floors. On these surfaces the temperatures are with approximately 2ºC higher than the rest of the façade

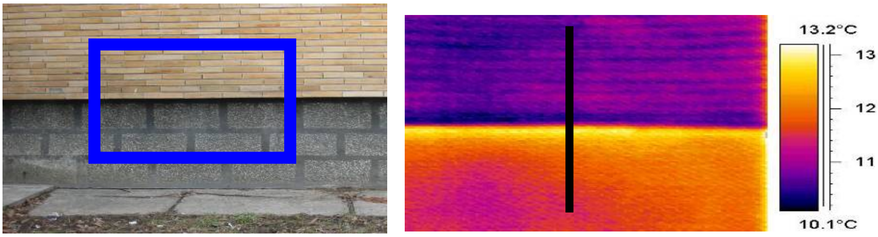

Figure 2. IR image, Northern façade; detail concerning the socle and the unheated basement influence

The IR images analysis do not emphasize important deficiencies concerning the thermal insulation, the presence and the route of the thermal bridges being taken into account in the energetic and thermal expertise, after the design study and the visual examination.

4. ESTIMATION OF THE BUILDING ENERGETIC PERFORMANCES

4.1. The assessment of the thermal characteristic of the envelope elements

There were analyzed the outline building elements that compose the envelope, such as:
- exterior walls made of prefabricated big panels that form the longitudinal façades;
- the concrete diaphragms plated with brickwork, forming the lateral façades;
- the terrace roof;
- the floor over the unheated basement;
- the exterior carpentry.

The closing elements structure was considered to be that from the design, because the IR analysis confirmed the concordance with the present situation. The materials characteristics and the thermal conductivity value have been multiplied by a coefficient grater than one that has in view some changes appeared in time because of the settlement, moisture and some defaults during the site works.

The correction linear coefficient has been estimated on the base of the surface thermal field of the physical and geometrical homogeneousness areas: corners, intersections, joints between panels.

4.2. The normal yearly consumption for spaces heating and hot water

The calculus method used for the estimation of the yearly thermal energy necessary for heating and hot water is based on the unsteady thermal regime assumption through the opaque construction elements and takes into account the effect of the gains generated by the human activity and solar radiation on the resulted interior temperature imposed by the thermal comfort standards.

The yearly consumption value depends on the envelope aria, the adjusted thermal resistance on the whole building, the ventilation rate and the adjusted number of days-degrees for heating.

5. THE BUILDING ENERGETIC EFFICIENCY

The solutions for the building energy efficiency had in view some investments in building and interventions on the heating equipments and had in view the decreasing of heat losses by transmission, decreasing of air infiltration, the increasing of installation output, the insurance of the indoor environment quality from the comfort and best air composition point of view.

There have been suggested more alternatives for the efficiency solutions concerning the envelope elements that after a performance analysis at the element level, they were combined in the frame of some strategies or efficiency alternatives. The alternatives were analyzed at the building level, the option for the best alternative was made on a multicriterial analyze

5.1. Solutions and efficiency strategies

The energy efficiency measures at the building level consist in:
a. Interventions at the flat roof level
a.1. the application of an additional thermal insulation layer over the water proofing at the roof level, changing it in an inverse flat roof;
a.2. an additional layer of thermal insulation on the existent one and the restoration of the water proofing;
b. Interventions at the floor level over the basement
b.1. the application of a thermal insulation layer of mineral wool at the inferior side of the floor over the basement;
b.2. the application of polyurethane foam at the inferior side of the floor over the basement
c. Interventions on the opaque parts of the exterior walls
c.1. the application on the exterior side of a thermal system made of a polystyrene layer covered with thin plaster with organic binding material;
c.2. the application on the interior side of a thermal insulation layer of the same thickness covered with plaster cardboards;
c.3. the changing of the existent façade in a glazed one of double-peau type.
d. Interventions on the glazed areas of the exterior walls;
d.1. the replacement of the wooden carpentry with PVC frames and thermopane glazing, with ventilation devices and Venetian blinds for the overheating avoiding during summer time
e. Interventions on the heating and domestic hot water equipments
e.1 the replacement of the taps with double adjustment with thermostats.
On the performance analysis base at the element level there were proposed 3 alternatives (strategies) of modernization:
Alternative I – a.1; b.2; c.2; d.1; e.1
Alternative II – a.1; b.2; c.1; d.1; e.1
Alternative III – a.1; b.1; c.3; d.1; e.1

5.2. The selection of the best alternative of energy efficiency

Taking into account the specific theoretic approaches of the systems engineering and decisions theory there was elaborated a method of general selection for the best thermal rehabilitation of the existent buildings that implies the following steps:

a. The analyze of the objectives and restrictions

The main objective of the thermal efficiency increasing for buildings means the decreasing of the energy consumption resulted from fossil fuels combustion necessary for operation. In the same time, it must to take into account also the other requirements fulfilment, directly related to the energy consumption, that refer to the insurance of a healthy and comfortable indoor environment with low investment costs, the time of investment repayment, architectural appearance, the technological implications.

In the decision theory spirit, if the decreasing of the energy consumption necessary for the building operation is an objective, the others become restrictions. Not only for the objective, but also for the restrictions, it must exist a relevant performance indicators set for the effect estimation for each strategy (measures combination) on the behaviour of the building system.

b. The delimitation of the optional space

The optional space is made of the possible combinations ensemble between the efficiency measures, respectively the alternatives and options ensemble. The delimitation of it is made on the practical or even subjective criteria. In the studied case, the optional space is made of the 3 alternatives previously presented.

c. The estimation of the performances for each possible option (alternative)

The building performances estimation from the points of view of energy consumption necessary for the building operation and the requirements fulfilment concerning the indoor environment quality may be achieved by assessment methods of each performance indicator separately using different calculus instruments or by numerical simulation of the whole building behaviour, by means of complex programs, such as TRNSYS, ESP-r, Energy Plus, etc. All these allow a realistic assessment of the performances having in view the mutual interconditionings and they are recommended for a correct decision concerning the best efficiency strategy.

For the presented case study, it was estimated each performance indicator taken separately. The performance indicators values are put in the extended performance matrix which is the base of the future selection (table 1)

The alternative	Effective values/ The solution efficiency related to the (c_j) criterion			
	The energy savings ΔE(MWh/year)	Comfort index PMV(-)	The time period necessary to recover the investment (years)	The architectural appearance
I	798889	-0,75	9,3	unchanged
	0,75	0,5	1,00	0,75
II	863407	+0,5	11	changed
	1,00	0,75	0,75	0,5
III	658210	-0,25	16	improved
	0,5	1,00	0,5	1,00
Importance coefficients λ_j	30%	30%	20%20%	20%

Table 1. The performance analysis of the selected thermal rehabilitation alternatives

d. The best alternative selection

An important step in the selection procedure is the estimation of all data from the performance matrix and the selection of those that responds in a better way to the proposed objective with the restrictions respecting. For this aim it is adopted a scale of estimation that allows the option comparisons with different performances, each estimated performance indicator being compared with the nominal value, if it exists. In the present study, depending on the performance indicators value, they were placed on a values scale between 0.5 and 1.00, having the ratings of satisfactory, well, very well.

The last step of the selection process consists in the definition of the use function by according some values for the balancing coefficients. Here may intervene certain aspects apparent subjective, that appear in an explicit way and may be negotiated. For example, in some situations, the criterion concerning the investment costs may have a higher weight than that concerning the indoor environment quality and in this way, a certain strategy is selected in which the respective indicator may answer at a satisfactory level, in the detriment of other that may answer at a "well" level, but is more expensive.

The selection is made on the estimation matrix, whose terms represent the value of the use function for each option, taken apart (table 2).

The alternative	The use function $U_{ij} = \lambda_{ij} \cdot c_{ij}$				
	Energy savings	Comfort index	Time for investment recovery	Architectural appearance	100%
I	22,5	15	20	15	72,5
II	30,0	22,5	15	10	77,5
III	15	30	10	20	75

Table 2 The selection of the best solution on the base of use function

In accordance with the estimation matrix, the second alternative presents the best rating, and as result it was proposed for application.

6. CONCLUSIONS

The energetic efficiency increasing of buildings dedicated to health care represents an important problem not only for Romania, but also for the other countries from communist camp.

For the adoption of the correct strategy for the energy efficiency increasing it is necessary a realistic estimation of the existent situation, but also a multicriterial approach. In this context, the IR analyze associated with a calculus methodology in dynamic regime, offers the basic data for the solutions elaboration for the efficiency increasing. The estimation methodology for the best solution integrates criteria concerning the energy savings and the indoor environment quality with those concerning the economic and architectural aspects.

Acknowledgements

The paper presents the results of some researchers made in the frame Grant CNCSIS "The insurance of the indoor environment quality with low energy consumptions – a feature of the sustainable architecture" 2004-2006.

The authors thank to the County Hospital managers from Bacău, which sustained the expertise works.

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TENTH NATIONAL AND
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NOVI SAD, NOVEMBER 22-24,2006.

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FATIGUE ASSESSEMENT OF WELDED HIGHWAY AND RAILWAY STEEL BRIDGES ACCORDING TO EUROCODE 3

Summary: *The paper gives an overview of the fatigue analysis of welded steel bridges according to Eurocode 3. The design philosophy must be changed especially in choosing the constructive details. A case study is also presented: the design and fatigue analysis of a welded highway composite bridge with a span of L=32,4m.*
With the increasing traffic intensity, fatigue becomes the main factor determining the safety of welded steel bridges.
Key words*: Fatigue, welded steel bridges, Eurocodes, fatigue, fatigue load model, stress range, detail category, traffic mix, trains type.*

PROCENA ZAMORA ZAVARENIH DRUMSKIH I ŽELJEZNIČKIH ČELIČNIH MOSTOVA PREMA EVROKODU 3

Rezime: *U radu je prikazan pregled postupaka za analizu zamora zavarenih čeličnih mostova prema Evrokodu 3. Ukazano je dailozofija projektovanja mora biti promenjena, naročito u izboru konstrukcijskih detalja. Prikazan je jedan primer: proračun i analiza zamora zavarenog spregnutog drumskog mosta raspona L=32,4m.*
Sa povećanjem inteziteta saobraćaja zamor postaje najvažniji faktor koji određuje sigurnost zavarenih čeličnih mostova.
Ključne reči: *Zamor, zavareni čelični mostovi, evrokodovi, model opterećenja zamora, područje napona, kategorija detalja, šema opterećenja, tipovi vozova*

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1. INTRODUCTION

With the elaboration of the structural Eurocodes a comprehensive set of standards reflecting the actual state of design rules in Europe was created [1]. Beginning with 2010 the Eurocodes are compulsatory also in Romania.

In the direction of fatigue analysis important changes appeared in comparison with the classical calculus concept. Higher axle loads and increasing traffic intensity had as result that bridges are subjected to greater fatigue loading than in the past.

The paper presents the principal steps in fatigue analysis of highway welded steel bridges.

2. LOADS FOR FATIGUE ANALYSIS ACCORDING TO EUROCODE 3

Traffic running on steel bridges produces a stress spectrum (loading history) which may cause fatigue. The stress spectrum depends a lot of parameters such as: the geometry of the vehicles, the axle loads, the vehicle spacing, the compositions of the traffic and its dynamic effects.

The damage will be produced by vehicles load crossing the bridge. In order to perform the fatigue analysis the real vehicles were systematized in some different load models.

2.1. Load Models for road bridges

Loads due to the traffic, consisting of cars, lorries and special vehicles (e.g. for industrial transport or military) give rise to vertical and horizontal, static and dynamic forces. EN 1991-2 [2] define different load models to be used for design of road bridges with loaded lengths less than 200m. Load models for loaded length greater than 200 m may be defined in the national Annex or for the particular project. In general, the use of Load Model 1 is safe-sided for loaded length over 200m.

The load models defined in this section do not describe actual loads. They have been selected and calibrated so that their effects represent the effects of the actual traffic in the year 2000 in European countries.

Vehicle traffic may differ between bridges depending on its composition (e.g. percentage of lorries), its density (e.g. average number of vehicles per year), its conditions (e.g. jam frequency), the extreme likely weights of vehicles and their axle loads and if relevant, the influence of road signs restricting carrying capacity. These differences should be taken into account through the use of load models suited to the location of a bridge by choice of different adjustment factors.

Characteristic loads are intended for the determination of road traffic effects associated with ultimate limit state verifications and with particular serviceability verifications.

For the ultimate limit state verifications four load models, named **Load Model 1** (**LM1**), **Load Model 2** (**LM2**), **Load Model 3** (**LM3**) and **Load Model 4** (**LM4**), are defined. Load Models 1, 2 and 3, where relevant, should be taken into account for any type of design situation and Load Model 4 only for some transient design situations.

2.1.1. Fatigue Loads models for road bridges

Five fatigue load models of vertical forces are defined. The use of the various Fatigue Load Models, depending on the structure's material, is defined in the EN 1992 to EN 1999.

Fatigue Load Models 1 – has the same configuration of the characteristic Load Model 1 defined for design at ultimate limit states but the values of the axle loads are multiplied by 0.7 and the values of the uniformly distributed loads are multiplied by 0.3.

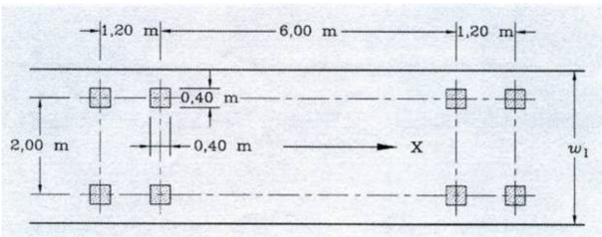
The maximum and minimum stresses ($\sigma_{FLM,max}$ and $\sigma_{FLM,min}$) should be determined from the possible load arrangements of the model on the bridge.

Fatigue Load Models 2 (set of "frequent" lorries) – consist of a set of idealized lorries, called "frequent" lorries, each "frequent lorry" being defined by: the number of axles and the axle spacing, the frequent load of each axle, the wheel contact areas and the transverse distance between wheels.

The maximum and minimum stresses should be determined from the most severe effects of different lorries, separately considered, traveling alone along the appropriate lane.

Fatigue Load Models 3 (single vehicle model) – this model consists of four axles, each of them having two identical wheels. The weight of each axle is equal to 120kN and the contact surface of each wheel is a square of side 0.40m. The geometry is shown in Fig. 1.

The maximum and minimum stresses and the stress ranges of each cycle of stresses fluctuation, i.e. their algebraic difference, resulting from the transit of the model along the bridge should be calculated. Where relevant, two vehicles in the same lane could be taken into account but the distance between the two vehicles is not less than 40 m.



w_l : Lane width
X : Bridge longitudinal axis

Figure 1 — Fatigue Load Model 3 (single vehicle model)

Fatigue Load Models 4 (set of "standard" lorries) – this model consists of sets of standard lorries which together produce effects equivalent to those of typical traffic on European roads. A set of lorries appropriate to the traffic mixes predicted for the route as defined in Table 1 should be taken into account.

The maximum and minimum stresses and the stress ranges of each cycle of stresses fluctuation, i.e. their algebraic difference, resulting from the transit of the model along the bridge should be calculated. Where relevant, two vehicles in the same lane could be taken into account but the distance between the two vehicles is not less than 40 m.

The stress range spectrum and the corresponding number of cycles from each fluctuation in stress during the passage of individual lorries on the bridge should be the Rainflow or the Reservoir method.

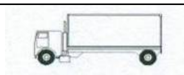
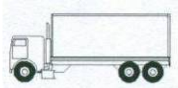
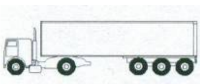
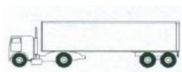
LORRY SILHOUETE	"FREQUENT" lorries		"STANDARD" lorries					Wheel type
					TRAFFIC TYPE			
	Axle spacing (m)	Frequent axle loads (kN)	Axle spacing (m)	Equivalent axle loads (kN)	Long distance	Medium distance	Local traffic	
					Lorry %	Lorry %	Lorry %	
	4.5	90 190	4.5	70 130	20	40	80	A B
	4.20 1.30	80 140 140	4.20 1.30	70 120 120	5	10	5	A B B
	3.20 5.20 1.30 1.30	90 180 120 120 120	3.20 5.20 1.30 1.30	70 150 90 90 90	50	30	5	A B C C C
	3.40 6.00 1.80	90 190 140 140	3.40 6.00 1.80	70 140 90 90	15	15	5	A B C C
	4.80 3.6 4.40 1.30	90 180 120 110 110	4.80 3.60 4.40 1.30	70 130 90 80 80	10	5	5	A B C C C

Table 1 —Set of "frequent" and "standard" lorries and traffic mixes

Fatigue Load Models 5 – this model consists of sets of the direct application of recorded traffic data, supplemented, if relevant, by appropriate statistical and projected extrapolations.
The stress range spectrum is determined like for FLM4.

A traffic category on a bridge should be defined, for fatigue verifications, at least by: the number of slow lanes, the number N_{obs} of heavy vehicles (maximum gross weight more than 100kN), observed or estimated, per year and per slow lane. The traffic category can be evaluated with indicative values for N_{obs} given in Table 2.
Intermediate values of N_{obs} are not excluded, but are unlikely to have very significant influence on the fatigue life time.

Traffic categories		**N_{obs} per year and per slow lane**
1	Roads and motorways with 2 or more lanes per direction with high flow rates of lorries	**2.0×10^6**
2	Roads and motorways with medium flow rates of lorries	**0.5×10^6**
3	Main roads with low flow rates of lorries	**0.125×10^6**
4	Local roads and motorways with low flow rates of lorries	**0.05×10^6**

Table 2 —Indicative number of heavy vehicles expected per year and per slow lane

2.2. Loads Models for railway bridges

The rail traffic actions defined in EN 1991-2:2001 have been selected so that their effects, with dynamic increments taken into account separately, represent the effects of service traffic. This load models can be used to design bridges on the standard track gauge and wide track gauge European mainline network.

Rail traffic actions are defined by means of load models. For the ultimate limit state verifications five load models, named **Load Model 71** (**LM71**) and **Load Model SW/0** (**LMSW/0** for continuous bridges), **Load Model SW/2** (**LMSW/2**), **Load Model HSLM** (**LMHSLM**) and **Load Model "unloaded train"** (**LM"unloaded train"**), are defined. Load Models 71 and SW/0 represent normal rail traffic on mainline railways, Load Model SW/2 represent heavy traffic loads, Load Model HSLM represent the loading from passenger trains at speeds exceeding 200km/h and Load Model "unloaded train" represent the effect of an unloaded train.

2.2.1. *Fatigue Loads models for railway bridges*

For calculating the fatigue life of structures three mixes of rail traffic are considered. A fatigue damage assessment shall be carried out for all structural elements which are subjected to fluctuations of stress. For structures carrying multiple tracks the fatigue loading shall be applied to a maximum of two tracks in the most unfavorable positions.

For normal traffic based on characteristic values of **Load Model 71,** including the dynamic factor **ϕ,** the fatigue assessment shall be carried out taking into account the next three traffic mixes: **"standard traffic", traffic with 250kN-axles"** or **"light traffic mix"** depending on whether the structure carries mixed traffic, predominantly heavy freight traffic or lightweight passenger traffic in accordance with the requirements for each particular project.

Each of the mixes is based on an annual traffic tonnage of 25×10^6tonnes passing over the bridge on each track and on twelve **Train types** for fatigue, named in EN 1991-2:2001, Annex D as **Type1** to **Type 12**. An example of this "Real Trains" is presented in Figure 2 and the main traffic mixes in Table 3.

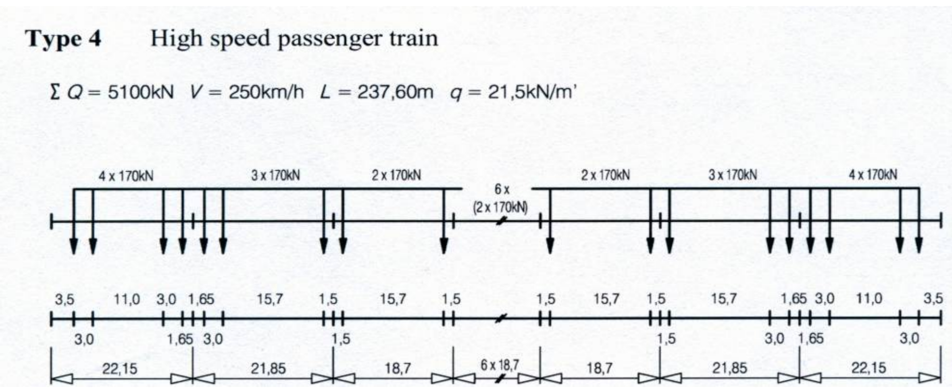


Figure 2 — Some Type trains

Traffic mix:	Train type	Train's descriptions	Number of trains/ day	Mass of train [t]	Traffic volume [10^6t/year]
Standard traffic mix with axles ≤22.5t	**1**	Locomotive-hauled passenger train, 200km/h	12	663	2.90
	2	Locomotive-hauled passenger train, 160km/h	12	530	2.32
	3	High speed passenger train, 250 km/h	5	940	1.72
	4	High speed passenger train, 250 km/h	5	510	0.93
	5	Locomotive-hauled freight train, 80km/h	7	2160	5.52
	6	Locomotive-hauled freight train, 80km/h	12	1431	6.27
	7	Locomotive-hauled freight train, 120km/h	8	1035	3.02
	8	Locomotive-hauled freight train, 100km/h	6	1035	2.27
Total			**67**		**24.95**
Heavy traffic mix with 25t axles	**5**	Locomotive-hauled freight train, 80km/h	6	2160	4.73
	6	Locomotive-hauled freight train, 80km/h	13	1431	6.79
	11	Locomotive-hauled freight train120km/h	16	1135	6.63
	12	Locomotive-hauled freight train 120km/h	16	1135	6.63
Total			**51**		**24.78**
Light traffic mix with axles ≤ 22.5t	**1**	Locomotive-hauled passenger train	10	663	2.4
	2	Locomotive-hauled passenger train	5	530	1.0
	5	Locomotive-hauled freight train, 80km/h	2	2160	1.4
	9	Suburban multiple unit train, 120km/h	190	296	20.5
Total			**207**		**25.3**

Table 3 —Railway Traffic mixes

3. FATIGUE ASSESSMENT ACCORDING TO EUROCODE 3

3.1. Fatigue design method based on stress range

A fatigue damage assessment shall be carried out for all structural elements which are subjected to fluctuations of stress. For the verification against fatigue, EC 3-2 [1], adopted the Palmgren-Miner rule of cumulative damage induced by different loads acting on the structure.

The most difficult matter is the establishment of loading history (stress spectra) which depends on many factors. The fatigue loading from traffic should be obtained for the project specification in conjunction with EC 1-3. The project specification should either define the fatigue loads in such a way that the verification may be carried out in the form of a cumulative damage assessment so that can be conducted as an equivalent stress range. $\Delta\sigma_{E_2}$.

For the simplified fatigue check of road bridges may be applied the **FLM3**, as it was described in 2.1.1, in conjunction with the traffic data specified by the competent authority.

For the simplified check of railway bridges the characteristic values of **LM 71,** as it was described in 2.2.1, should be used, including the dynamic factor ϕ_2 , in the accordance with the requirements for particular project.

For the simplified fatigue loading above specified, the following procedure may be used to determine the design stress range spectrum, unless otherwise specified by the competent authority.

In all critical section of structural element it will be to calculate the maximum stress $\sigma_{P,max}$ and the minimum stress $\sigma_{P,min}$. The reference stress range $\Delta\sigma_P$ for determining the damage effects of the stress range spectrum should be obtained from:

$$\Delta\sigma_P = \left|\Delta\sigma_{P,\max} - \Delta\sigma_{P,\min}\right| \tag{1}$$

and the damage effects of the stress range spectrum may be represented by the **damage equivalent stress range** related to $2x10^6$cycles, $\Delta\sigma_{E2}$, see Figure 3:

$$\Delta\sigma_{E2} = \lambda \times \Phi_2 \times \Delta\sigma_P \tag{2}$$

where: λ - is the damage equivalence factor,

ϕ_2 – is the damage equivalent impact factor.

The damage equivalence factor λ should be obtained from:

$$\lambda = \lambda \times \lambda_2 \times \lambda_3 \times \lambda_4 \le \lambda_{\max} \tag{3}$$

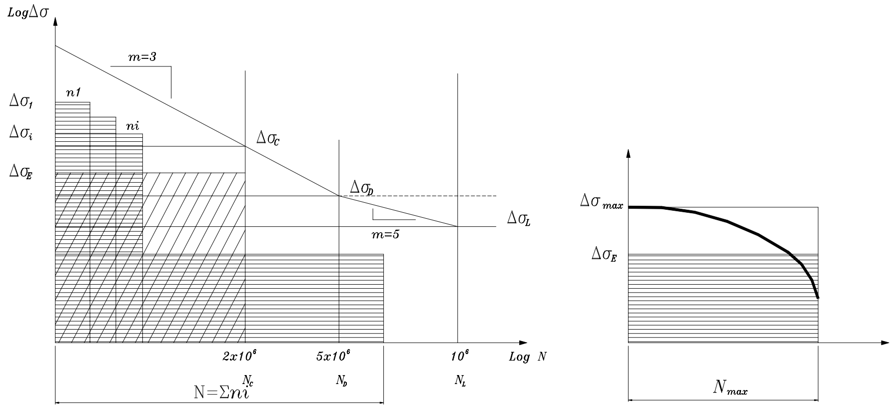


Figure 3 — Wöhler Curve, Damage cumulate and the loading factor in the real life of the element.

where: λ_1 – is a factor for different type of girders that take into account the damaging effect of traffic and depends on the length (span) of the influence line area;

λ_2 – is a factor that takes into account the traffic volume;

λ_3 – is a factor that takes into account the design life of bridge;

λ_4 – is a factor that takes into account the heavy traffic on other lanes;

λ_{max}-is the maximum λ-value taking account of the fatigue limit

All λ_i-values for road or railway bridges can be calculated with the relations defined in ENV 1993-2:2003, paragraphs 9.5.2 and 9.5.3.

The damage equivalent impact factor ϕ_2 may be taken as equal to 1.0 for road bridges and for railway bridges the value of ϕ_2 should be calculated with:

$$\Phi_2 = \frac{1.44}{\sqrt{L_\Phi} - 0.2} + 0.82 \text{, witht: } 1{,}00 \le \phi_2 \le 1.67 \tag{4}$$

The stress range $\Delta\sigma_E$ as a substitute for the design stress range (all $\Delta\sigma_i$) can be also presented [3] in the form of $\Delta\sigma_E = \alpha\Delta\sigma_{max}$ where α is considered as a loading factor in the real life of the element. Using the equation of Woehler curve and the figure 3 result the following equation for α:

$$\alpha = \left[\sum \left(\frac{\Delta\sigma_i}{\Delta\sigma_{max}}\right)^m \cdot \frac{n_i}{N_{max}}\right]^{1/m} \le 1 \tag{5}$$

The influences that can appear are resulted from the successive appearance of the different amplitude couples and can be neglected.

The fatigue assessment shall be made by checking the criteria:

$$\gamma_{Ff}\Delta\sigma_{E2} \le \Delta\sigma_c / \gamma_{Mf}$$

and/or (6)

$$\gamma_{Ff}\Delta\tau_{E2} \le \Delta\tau_c / \gamma_{Mf}$$

where: γ_{Ff} - is the partial factor for fatigue loads (γ_{Ff} =1.00);
γ_{Mf} - is the partial factor for fatigue resistance ((γ_{Mf} =1.00 for redundant structural elements and γ_{Mf} =1.15 for key structural elements);
$\Delta\sigma_c$ and $\Delta\tau_c$ – are the reference value of the fatigue strength at N_c=2 millions cycles of a specified category of structural detail (fatigue strength).

3.2. Assessment method based on recorded traffic (Alternative procedure)

As an alternative to the procedure given above, the fatigue assessment based on recorded traffic can be used. A stress history should be obtained by analysis using recorded representative real traffic data (real lorries), multiplied by a dynamic amplification factor φ_{fat} which take into account the dynamic behavior of the bridge and depends on the expected roughness of the road surface.

The road surface can be classified in terms of the power spectral density (PSD) of the vertical road profile displacement G_d, i.e. of the roughness. G_d is a function of the spatial frequency n, $G_d(n)$, or of the angular spatial frequency of the path Ω, $G_d(\Omega)$, with $\Omega=2\pi n$. The actual power spectral density of the road profile should be smoothed and then fitted in the bi-logarithmic presentation plot by a straight line in an appropriate spatial frequency range.

The fitted PSD can be expressed in a general form as:

$$G_d(n) = G_d(n_0)\left(\frac{n}{n_0}\right)^{-w} \quad (7.a) \qquad \text{or} \qquad G_d(\Omega) = G_d(\Omega_0)\left(\frac{\Omega}{\Omega_0}\right)^{-w} \quad (7.b)$$

where: n_0 – is the reference spatial frequency (0.1 cycle/m),
Ω_0 - is the reference angular spatial frequency (1 rad/m),
w – is the exponent of the fitted PSD.

Often it is convenient to consider velocity PSD, G_v, in terms of change of the vertical ordinate of the road surface per unit distance traveled. Since the relationship between G_v and G_d are:

$$G_v(n) = G_d(n)(2\pi n)^2 \quad (8.a) \qquad \text{and} \qquad G_v(\Omega) = G_d(\Omega)(\Omega)^2 \quad (8.b)$$

When w=2 the two expressions (8) of velocity PSD are constant.
The class limits are graphed versus displacement PSD.
Unless otherwise specified the recorded axle loads should be multiplied by:
φ_{fat} = 1.2 for surface of good roughness (asphalt or concrete layers),
φ_{fat} = 1.4 for surface of medium roughness (old roadway layers, cobblestones, etc.).

The cumulative fatigue damage calculated by use of records should be multiplied by the ratio between the design working life and the duration considered on the histogram. In the absence of detailed information, a factor 2 for the number of lorries and a factor 1.4 for the load levels are recommended.

4. CASE STUDY

In the field of small and medium spans bridges (the majority of structures can be included here) the present trend to use steel welded composite (in connection with the concrete slab) girders must be emphasized. Generally, these steel girders consist of very clear and simple sections, thick web and upper and bottom flanges, composed of a minimum number of plates, usually only one. The progress registered in the last time in buckling calculation allows to decrease the number of stiffeners and to improve their arrangement.

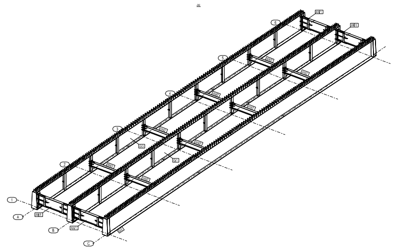
In order to analyze which is the impact of the new standards on the design activity a new composite steel-concrete structure road bridge, was chosen the bridge over Barzava river located on the county road DJ 585 at km. 13+400, Bocsa to Ramna, with a span of L=32.4m., simply supported main girders (Figure 4).

The fatigue assessment was performed for the main girders of the bridge, considering two constructive details: the cross sections in the middle of the span (x=16.2 m.) and on the bottom flange but weld (x=11.0m).

The design of web vertical stiffeners was made as a T-section (see Figure 5), that they avoid web's local buckling and fulfil the constructive requirement:

$$\frac{b}{t} = \frac{100}{20} = 5 \le 12.5\sqrt{\frac{235}{f_y}} = 12.5\sqrt{\frac{235}{355}} = 10.17 \qquad (9)$$

a)



b)

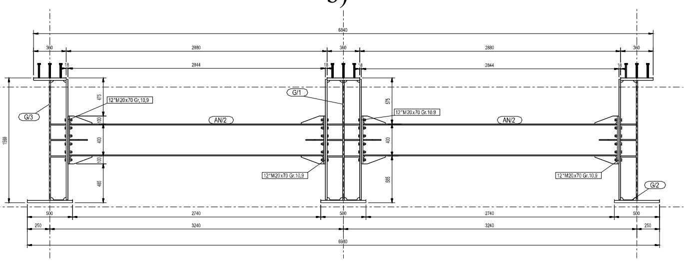


Figure 4 − Steel structure of Bocsa-Ramna road bridge: a)Spatial assembly; b)cross-section

Only the stiffener's flange was welded with continuous longitudinal fillet welds on the main girder bottom flange.

For the welded built-up main girder at bottom flange level it's to notice three types of constructive solution which guide to the next notch cases, according to detail categories described in EC 3:

a) – detail category $\Delta\sigma_C$=80 N/mm^2 (transverse splices in plates, welded from both sides or continuous longitudinal weld that crosses a transverse butt weld**),** for the prolongation cross butt weld of bottom flange's plates (see Figure 5, detail 1);

b) – detail category $\Delta\sigma_C$=100 N/mm^2 (web to flange full penetration weld**),** for the welds between web and flanges (see Figure 5);

c) – detail category $\Delta\sigma_C$=56 N/mm^2 (longitudinal attachment, l=150mm**),** for the fillet weld between web vertical stiffeners and girder bottom flange plate (see Fig.5);

At upper flange level the **detail category $\Delta\sigma_C$=80 N/mm^2** as effect of welded shear connectors on base material must be added.

4.1. Fatigue assessment based on equivalent stress range

Taking into account the load models recommended by Romanian design code [4] such as **special vehicle V80** (4 axles, gross weight of 800 kN), **heavy lorry A30** (3 axles, gross weight of 300 kN) and **lorry A13** (2 axles, gross weight of 130 kN) results the values of stress range $\Delta\sigma_P$ and then the equivalent stress range $\Delta\sigma_{E2}$, so that is shown in Table 4. It is to mention that the recorded traffic is smaller that the traffic takes as background for λ_2 factor calibration (very hard type of Auxerre traffic).

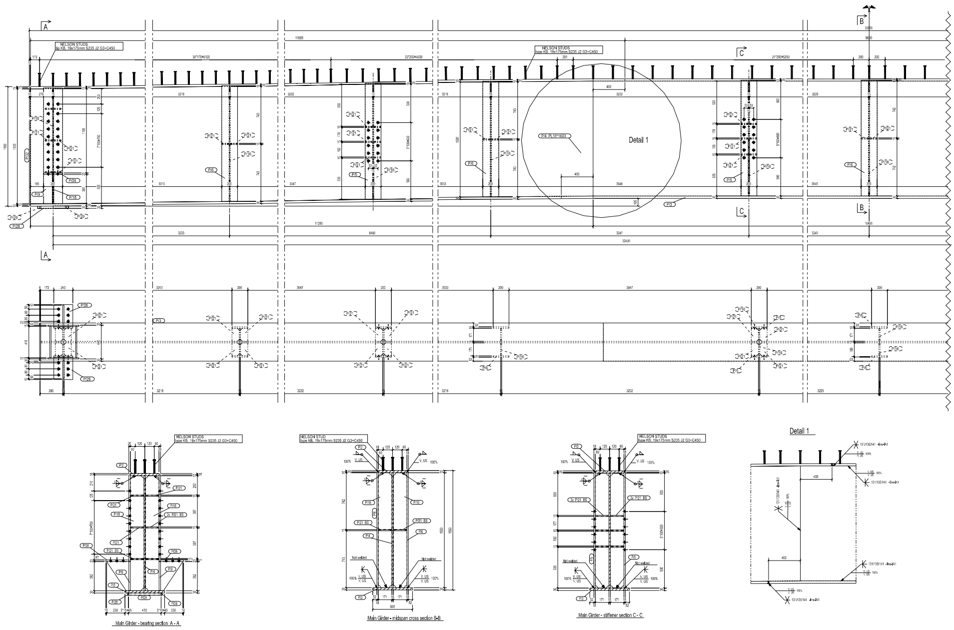


Figure 5 − Bocsa-Ramna road bridge: welded built-up main girder

The designed vertical stiffener correspond to detail category with fatigue strength $\Delta\sigma_C$=56N/mm^2, an unfavorable situation.

Design case	Notch case	Design Load Model	Fatigue damage						Fatigue resistance			Safety factor
Critical section			Calcu-lated $\Delta\sigma_P$ [N/mm^2]	$\lambda = \prod_1^4 \lambda_i$	Φ_2	equiv. $\Delta\sigma_{E2}$ [N/mm^2]	γ_{Ff}	$\frac{\Delta\sigma_{E2}}{1/\gamma_{Ff}}$ [N/mm^2]	$\Delta\sigma_C$ [N/mm^2]	γ_{Mf}	$\frac{\Delta\sigma_c}{\gamma_{Mf}}$ [N/mm^2]	(12)/(9)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
projected		V80	88.3									
Midspan	56	A30	76.5	0.92	1.0	81.2	1.0	81.2	56	1.15	48.7	0.599
X=16.20m		A13	47.0									
		V80	88.3									
Midspan	71	A30	76.5	0.92	1.0	81.2	1.0	81.2	71	1.15	61.7	0.759
X=16.20m		A13	47.0									
		V80	88.3									
Midspan	80	A30	76.5	0.92	1.0	81.2	1.0	81.2	80	1.15	69.6	0.857
X=16.20m		A13	47.0									
Midspan	56	FLM 3	51.25	0.92	1.0	47.15	1.0	47.15	56	1.15	48.7	1.03

Table 4 —Fatigue life assessment

A better solution in order to increase the fatigue strength is to decrease the stiffener flange width at bottom welded side. Reducing the stiffener's flange width, e.g. as a feather to a total length at bottom side less of 80mm, increase the fatigue strength $\Delta\sigma_C$ to

71N/mm^2 or to 80 N/mm^2. The positive influence of right choosing of constructive detail to fatigue behavior is presented in Table 4.
Performing the calculus using FLM 3 adopted by Eurocode the fatigue behavior is also improved.

4.2. Fatigue assessment using recorded traffic data

With the help obtained from the County Traffic Department the analysis and tipification of traffic was done. The recorded data of traffic is shown in Table 5.
Classifying the stress history by the reservoir method and using the PLM-cumulative damage, it results for each analyzed case the total damage due in structure during the estimated service life.

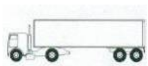
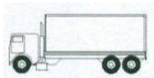
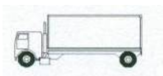
Notch case		TRAFFIC DATA				FATIGUE CYCLES			Estimated Damage D_i=ni/N_i
		Lorry silhouette	%	Lorry geometry [m]	Axle Load [kN]	Nr./day	Service Life Number n_i	Wöhler Curve cycles N_i	
c)	1	V80 	2	A=1.20m B=1.20m C=1.20m esp=80m	P_1=200 P_2=200 P_3=200 P_4=200	n_{80} = 1	35000	510.17	0.07
	2	A30 	15	A=6.00m B=1.20 esp=10m	P_1=60 P_2=120 P_3=120	n_{30} = 8	292000	784530	0.43
	3	A13 	83	A=4.00 m esp= 8 m	P_1=39 P_2=91	n_{13} = 46	1679000	3382988	0.49
	Total Damage							D=	0.99
c1) 71	1					V80		1.039.736	0.035
	2					A30		1.598.899	0.183
	3					A13		6.894.638	0.24
	Total Damage							D=	0.458
c2) 80	1					V80		1.487.366	0.024
	2					A30		2.287.261	0.127
	3					A13		9.862.940	0.17
	Total Damage							D=	0.321

Table 5 —Recorded traffic data and their damage

Choosing the adequate Wöhler-curve corresponding to analyzed notch case (in accordance with detail category, $\Delta\sigma_C$=56N/mm^2 or $\Delta\sigma_C$=71N/mm^2 $\Delta\sigma_C$=80N/mm^2), the total cycles number N_i, admitted by detail category under stress range $\Delta\sigma_i$, will be calculate with:

$$N_i = N_C \left(\frac{\Delta\sigma_c}{\Delta\sigma_i} \right)^m \tag{10}$$

With this number can be estimated the fatigue damage produced by every stress range.

5. CONCLUSIONS

Eurocode 1 and 3 offers a comprehensive method for the verification against fatigue collapse. The final aim is to realize economic structures, fatigue resistant, easy to protect against corrosion, to inspect and to maintain.

Topics mentioned in this paper are summarized as follows:

1) With the increasing traffic intensity fatigue behavior become the main factor determining the safety of welded steel bridges;
2) The main Eurocode's design methods against fatigue are presented;
3) The right choice of constructive details, so that utilize a better notch case (higher fatigue resistance $\Delta\sigma_C$) become decisive for fatigue life prolongation;
4) For local road with small traffic (total tonnage per year under 1 million tones) the design after Eurocode traffic and stress range method, is too much secure and non economic; a national coefficient according to real traffic must be introduced.

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TENTH NATIONAL AND
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iNDiS 2006

NOVI SAD, NOVEMBER 22-24,2006.

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THE TRAIAN SQUARE OF TIMISOARA – FROM PROJECT TO EXECUTION

Timisoara, a city situated in the western part of Romania, holds a valuable patrimony of architecture. Distinguished points are the Traian square and the neighbourhoods as an important node in the existent urban net, representing an alternative for the value of the patrimony of the Borough Quarter or other interesting areas of the city. The Traian square is situated in the center of the Fabric quarter, one of the most anchient quarters of the city of Timisoara. At this time the Traian square is a vast area partially covered with an asphaltic lining and partially covered with green areas, bounded on the eastern and northern part by two traffic ways paved with hard rock. The importance of the Traian square for Timisoara makes an intervention in this area opportune, intervention with an important social role, namely the one of renewing an interest center which should remind the comunity of the Fabric quarter identity. Moreover the historical values will be kept alive and the area will be adapted to the contemporary live through new functions and activities. The existing infrastructure and the utilities have to obey and to fit in the historical aspects.

Key words: *square, rehabilitation, grid, blocks, drain, urban furnishing.*

TRAJANOV TRG U TEMIŠVARU– OD PROJEKTA DO REALIZACIJE

Temišvar je lociran na zapadu Rumunije sa vrednim arhitektonskim nasleđem. Najizrazitiji je Trajanov trg i okolina kao značajni punkt urbane mreže, reprezentujući alternativne vrednosti nasleđe Barokne četvrti i drugih interesantnih područja grada. Trajanov trg je lociran u težištu fabričkog kvarta jednom od najstarijih kvartova Temišvara. Sada je ovaj trg prostran pokriven asfaltom i delimično zelenim površinama ograničenim sa istočne i severne strane sa dve saobraćajnice popločane kamenom. Značaj Trajanovog trga čini intervenciju ove oblasti korisnom, sa značajnom društvenom ulogom u kojem će se obnoviti interesantan centar koji će sačuvati identitet fabričkog kvarta. Štaviše, istorijska vrednost će biti očuvana a površina biti adaptirana za savremeni način života sa novim funkcijama i aktivnostima. Postojeća infrastruktura i utilitarnost će se podvrći i prilagoditi istorijskom aspektu.

Ključne reči: *trg,rehabilitacija, blokovi, dreniranje, urbani mobilijar*

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1. INITIAL SITUATION

The Traian Square is located in Timisoara city, in the center of the Fabric quarter, one of the first quarters in the city.

Timişoara is a city with a valuable architectural patrimony, out of which the Train Square is distinguished as an important node in the existing urban net, representing an alternative to the value given by the Borough quarter or other areas of interest of the city.

At the moment, the Traian Square presents itself as a large partialy paved surface, and partially covered with green areas, bordered on the eastern and northern side by two carriageable areas paved with hard rock. The entire area of the square is heavily degraded due to the quartering of the buildingyard organisation for the modernizing works of the streets Dacilor and Mihalache. (Fig. 1)

Figure 1. Piaţa Traian. Initial aspect

In the perimeter of the Traian Square there are two locations with historical value, one in the western extremity and the other in the eastern part.

The vegetation of the square consists of small trees placed on the eastern side and on the south-eastern corner, respectively shrubs in the middle area of the square.

This situation makes the intervening in the Train Square opportune, which also has an important social role, namely to renew an area of high interest in the city.

In addition to this the historical values in the area will be maintained and the area will be adapted to the contemporaneous live through new functionalities and activities. The infrastructure and the existent utilities have to fit in the historical pattern.

2. CONSTRUCTIVE STRUCTURE

The square area will be devided in quadrate zones (9 m x 9 m) covered with new andesite paving bricks of 9 x 9 x 9 cm. These areas will be bordered by a 1 m wide grid, realized out of hard rock pavement, recovered from the already modernized streets in the city (Fig. 2). Beyond its aesthetic role, the grid also fulfils a functional drain role for collecting and guiding the rainwater towards the outflow.

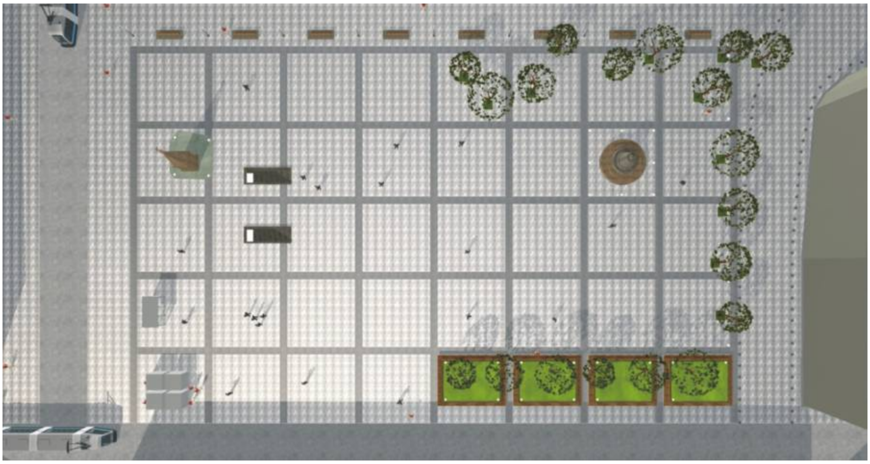

Figure 2. Top view project

The Traian Square will receive well delimited turfed new green areas, which consider the emplacement of the existent vegetation (Fig. 3).

The valuable existent trees will be kept and their number will be enriched in order to make a complete landscape composition. Moreover, in the northern part there will be planted a minimum of 9 trees completing the existent vegetation.. The existing trees and also the new ones will have a turfed delimitation of 1 m^2.

The rehabilitation and the entire repositioning of the urban public network in the square will be followed by a coherent finishing, maintaining the initial valuable architectural appearance and also by positioning different urban furniture together with the removal of the cables hanging down on the facade of the buildings.

Figure 3. Proposed green areas

A part of the urban furniture in the square, will be realized around the existing trees. Thus the green areas will be emphasized by bordering them with banks (Fig. 4) designed not to suffer damages in time and also to fit in the architecture of the square (Fig. 5).

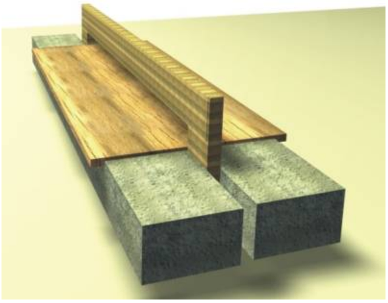

Figure 4. Street banch

Figure 5. View towards the northern side of the square

It has been proposed that the necessary amount of street tidies should be provided on the perimetral area of the square, from 25 to 25 m. The seating places (the street banches) should also have each one a street tidy. These will be personalized and integrated in the historical context of Timişoara.

The solution for the street lightening refers to positioning it on the facades of the buildings by means of some steel cantilivers (Fig. 6). The existent cantilivers will be preserved and, if necessary repaired. The new cantilivers will be placed above each entrance, where they are missing, especially on the northern side of the square. Their number will be encreased if the facade is to long.

On the square area it is allowed to use lightening piers (Fig. 7). These will have diminished heights, of approximately 3-3,50 m, in order to lighten the paved area. They will be in accordance with the historical spirit of the place and will be placed in such manner as to sustain the arranging composition of the square.

Figure 6. Steel cantiliver for public lightening

Figure 7. Pier for street lightening

The two existent monuments will be lightened artistically with floodlights embaded in the pavement. By accessing the street ligthening, which is deficitary right now, the improvement of the necessary safety degree of a pedestrian area of high interes is aimed and in addition to this, also a revival of the first city with street lightening in Europe.

The bollards separating the two traffic types, will also receive a special pattern for the whole studied area.

On the building facades, in the intersection areas directional plates, meant to give information about the name of the street, have been foreseen. Like european urban centers, noteworthy due to their architecture and history, buildings important from architectural and historical point of view will be marked with narative information plates, which should shortly present either the year of edification, the architect/engineer, the architectural style of the building, or events or personalities bond to that certain edifice.

The details regarding the urban furniture, the materials, the lightening devices will all be in accordance and harmony with the facade architecture, in order to preserve the history aspect of the place.

Regarding the architectural request the Traian Square to be transformed in a public area designed mainly for pedestrian traffic, the works leaned towards the realisation of a paved area with natural stones in order to preserve the specific character of this anchient area of Timişoara.

The quadrate surfaces from the pedestrian area ar thought to be realized as pyramids with a functional role guiding the rain water towards the bordering drains.

Around the toilets and the monuments near the Dacilor street, over the length of three bricks, the joints will be sealed with cement mortar for the impermeabilization of the area. The first step for the toilet accesses will be 40 cm wide, consisting in two pavement rows sealed with cement mortar.

The carriageway from the northern side will transform in an exclusively pedestrian area and the pavement will be realized at the same level with the rest of the square. On this area a 1 m wide drain will be foreseen on a distance of 5 m from the pedestrian surface. The drain is realized out of paving of hard rock placed on a concrete layer and sealed with cement mortar, fulfilling the role of guiding the rain water towards the outflows. For the impermeabilization of the building area at the footway level, the first three joint lines will be sealed off with cement mortar.

On the eastern side the carriageway area will be kept, although a pavement at the same level will be realized. The delimitation of the carriageway is made with ornamental cast iron bollards and adequate marking. The rain water will be collected in a 40 cm wide drain, bordering the carriageway.

The access of vehicles in the square will be limited, being allowed only in emergency cases (police, ambulance, firefighters) and also to supply the commercial areas which will be functioning in the buildings around.

An increased attention will receive the protection process of the existent trees and assuring of an area of minimum 1 m^2 without pavement, delimited through a border out of sealed paving with cement mortar.

In order to obtain a high efficiency for the collecting and guiding of the rain water, the entire surface of the square will be realized in dislevelment having declivities towards collecting drains.

This work aims to emphasize an important urban network in the city, by renewing the center of this area. The Traian square can be a starting point for the renewal of the entire area, which encounters big problems regarding the buildings.

The project plays also a big social role, namely the renewing of a center of interest in the area, through which the community shell again uncover the identity of the Fabric quarter.

3. ASPECTS DURING THE EXECUTION

Figure 8. Aerial view of Traian Square during the execution

Figure 10. Benches and new turfed green areas

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TENTH NATIONAL AND
FOURTH INTERNATIONAL
SCIENTIFIC MEETING

iNDiS 2006

NOVI SAD, NOVEMBER 22-24,2006.

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SPECIFICAL PROBLEMS AT AN OLD CONCRETE BRIDGE REHABILITATION

Summary: One of the oldest concrete bridges of the city, Traian Bridge, was built in 1916. The paper presents the initial rehabilitation solution and the intercurrent changes in the execution process, due to technical conditions imposed by the 90 years old reinforced concrete structure. Causes leading to the abandoning of the consolidation solution of the central bridge deck of this Gerber structure by external prestressing as also the concrete quality determinations performed to the structure of the central bridge deck are also being presented. In the central span a new steel-concrete composite bridge deck is being realized, which has to respect the geometry of the old bridge deck and the spirit of the entire structure. In order to keep the gauge for the pedestrian traffic and also for the bike lane, it has been acted to the demolition of the old footway console and to its designing in a new metallic solution. We present the protection system applied to the old concrete structure as also the modern tram line fixing system on the bridge deck. The rehabilitation concept consists of bearing capacity increasing in conditions of maximum safety, keeping though the original architectonical appearance and attractivity.
Key words: bridge, rehabilitation, consolidation, steel-concrete composite bridge deck, footway.

POSEBNI PROBLEMI PRI REHABILITACIJI STAROG BETONSKOG MOSTA

Rezime: Jedan od najstarijih mostova u gradu, Trajanov most, izgrađen je 1916. godine. U radu je prikazano početno rešenje rehabilitacija i promene u procesu izvođenja u tehničkim uslovima izazvanih sa starošću armiranobetonske konstrukcije od 90 godina. Uzroci koji su doveli do napuštanja rešenja konsolidacije srednje kolovozne ploče ovog Gerberovog nosača spoljašnjim porednaprezanjem a opisan je i način određivanja kvaliteta betona središnje kolovozne ploče. U srednjem rasponu izveden je nova spregnuta konstrukcija ploča mosta, koja je iste geometrije ploče starog mosta i suština cele konstrukcije. Da bi se sačuvao pešački saobraćaj kao i staza za bicikliste, morle su biti uklonjene i zamenjene stare pešačke konzole. Prikazan je zaštitni sistem primenjen na stari most i za liniju modernog tramvaja preko kolovozne ploče. Koncept rehabilitacije povećao je kapacitet nosivosti u uslovima maksimalne sigurnosti, uz očuvanje originalni arhitektonski izgled i atraktivnost.
Key words: most, rehabilitacija, konsolidacija, čelik-beton spregnuta kolovozna ploča, peš. staza

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1. HE BRIDGES OF TIMISOARA

Timisoara, the biggest city in the western part of Romania, does not have many bridges. The bridges of the city are not hundreds of years old. Some of them, the oldest ones, were built only 30…40 years after the reinforced concrete started to be used in the world. As a conclusion it can be said, that these bridges of reinforced concrete having an age of 80 years are “old bridges”.

The borough of Timisoara, was placed at the beginning in a privileged area, a little bit higher, while the city developed outside the walls, was placed in the lower area, which was very soon parceled out by the capricious arms of the Timis River and sunken in the bordering moor (Fig.1).

Figure 10. Timisoara, the borough from 1686

The urbane communication ways arose providing facilities but also finally having holes, dust and creating pollution. At the beginning the bridges were realized out of wood and masonry, materials which came handy for the wrights of those days.

The end of the XIX century, found Timisoara in this stage, having an urban dowry, which was continuously enriched by building passing structures. Timisoara presents it self at the beginning of the XX century with 71 bridges, 5 steel bridges, 9 brick masonry bridges and 28 wood bridges.

Concerning the urbanism, the end of the XIX century means making a crucial decision regarding the future development of the city: the abolishing of the Timis arms and building the Bega channel. The civic design and modernizing plan were elaborated in 1893 by the architect Lajos YBL and the royal engineer Aladár KOVÁCS-SEBESTÉNY.

The year 1905 symbolizes the debut of the realizing program of these city-planning endowments, vital for Timisoara. The construction of the Bega channel at the beginning of the XX century has meant the replacement of the old bridges with new ones that should have satisfied the passing of the 35 meters wide channel and also to allow the development of the navigation.

The constructive model of the bridges is being given by the structure with three spans, out of which the central one, the largest, assures the passing of the channel. The ancient bridges were build as Gerber beam, and the new ones as simple supported beam with cantilevers or 3 spans continuous structures. 44% of the bridges are intended for the auto traffic and 56% support a mixed traffic tram & auto, fact that requires them intense and divers.

Timisoara's bridges symbolize the reinforced concrete technical level in crossing structures field at the beginning of the XX century. They are also representing our fellow citizens (engineers and architects) conception, achievements and their eagerness for aesthetic. As a result of inspections on the crossing structures from Timisoara city and archives consulting, there can be pointed out a series of aspects regarding the constructive parameters and their functionality. Almost half (44%) of a total of 9 bridges crossing the Bega channel, were built before 1950 especially in the first quarter of the century, proof being "Decebal" Bridge 1908, "Tineretii" Bridge – 1914, "Traian" Bridge – 1916, "Eroilor" Bridge – 1938.

The reinforced concrete, exclusively used for the old structures, was partially replaced with steel in the mixed structures, which took the place of the old bridges, considered unable to bear the existent traffic. What a shame for the loss of these architectonical monuments!

2. THE OLD TRAIAN BRIDGE CONSOLIDATION

The Traian Bridge is the first old bridge included in the rehabilitation program of the tramways, sustained through European and local financing. Over time, until the bridge was named "Traian", it was known as: Hunyadi-hid, Huniade Bridge, respectively Hunyadi-Brücke (Fig.2).

Figure 2. The Traian bridge before starting the rehabilitation works

The town's engineer LÁD Károly jr., former assistant of prof. Aladár KOVÁCS-SEBESTÉNY, designed the bridge.

The bridge, the execution of which was public auctioned in 1911, was built in medieval style, which had to fit in the neogothic architecture from the left bank of the Bega channel. The structure was finished in 1914, but, because of the war, the bridge was given in exploitation only starting with 1916, being considered an "ancient" bridge of the city.

It is made as a Gerber beam of reinforced concrete, with spans (10 + 12,25) + 13 + (12,25 + 10) m and a total gauge of 17,00 m, being designed for the pedestrian traffic, auto and tram. The main span that assures the passing of the channel consists of the marginal superstructures cantilevers and an isostatic central span. The cross section reveals 9 girders of reinforced concrete with variable dimensions. The traffic develops on 4 lanes. The footways have technical channels and pedestrian parapet made out of wrought iron on the central span, respectively of shaped stone on the end spans.

The Traian Bridge was expertised in 1972 and in 1987 its bearing capacity was tested (Fig.3). The results of the tests have shown that the structure still has a sufficient bearing capacity regarding the truck and tram traffic.

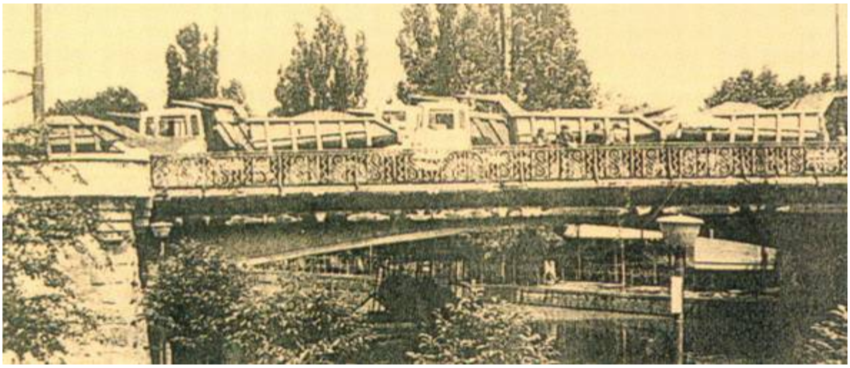

Figure 3. Traian bridge – bearing test

Due to the fact that the maintenance and rehabilitation works were not executed in time, a new expertise was necessary in 2000.

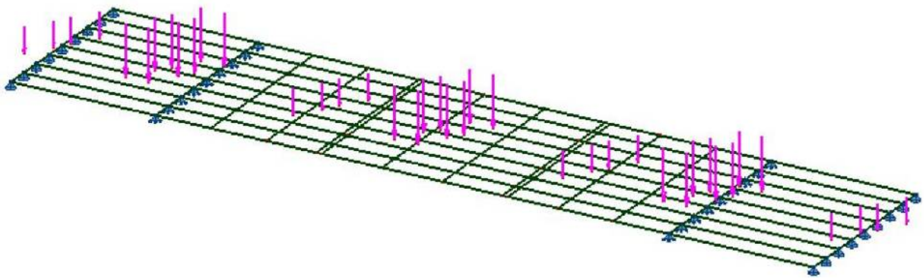

Figure 4. The spatial structure and its live loadings

The structural analyse realized on a spatial model (Fig.4), using live loadings from tram and truck convoys, showed that a consolidation of the central span and that of the marginal girders, is imperative.

The execution of all the necessary repairs was also requested. The bridge was included in the tramways rehabilitation program mentioned earlier. The most important detected damages are:

- the low bearing capacity due to the old design rules (90 years);
- not viable systems for fixing the tram rails on the superstructure;
- degradations of the pedestrian forged iron parapet, as well as the stone parapet;
- not viable and destroyed expansion joints and bearings;
- degraded water-proofing on wide areas;
- degraded short cantilevers, with improper supports and corroded reinforcement;
- massive degradations of concrete and reinforcement of the footway cantilevers.

2.1. Consolidation solutions

By consolidating the structure it was aimed to increase the bearing capacity of the structure at the level of A30 trucks, simultaneously with trams. The carriageway will also be extended up to standard values by eliminating the technical channels [2].

2.1.1. Initial design solution

The increasing of the bearing capacity of the structure, will be achieved through longitudinal external prestressing of the central superstructure girders.

In this solution the width of the footway would have been reduced due to the increasing of the carriageway simultaneously with the maintenance of the total bridge width (Fig.5).

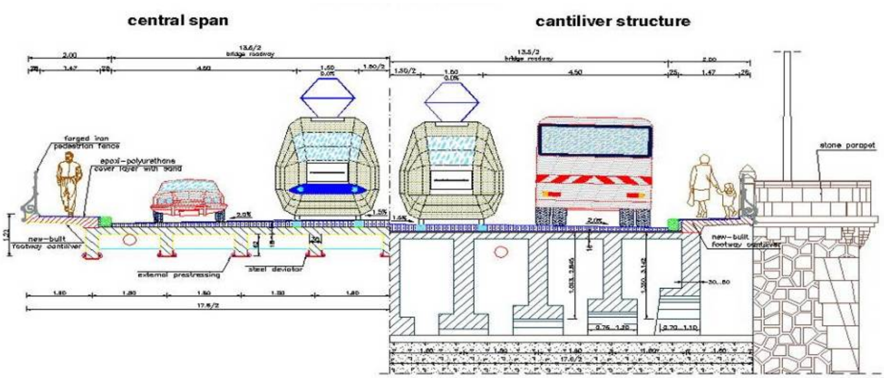

Figure 5. Traian bridge. Cross section in initial design solution.

In order to rehabilitate the central span, the following must be done: the superstructure will be raised with special hydraulic jacks; the short cantilevers will be reconcreted; in the core of the beam will be prepared special areas for fixing the prestressing cables; each beam will be external prestressed with 2 strands (about 400 kN); the cantilevers of the footway will be entirely rebuild; the expansion joints are going to be replaced. For the execution of the prestressing system, cast steel anchorages and deviators, having a special form, were designed in order not to change the architecture of the structure very much (Fig.6).

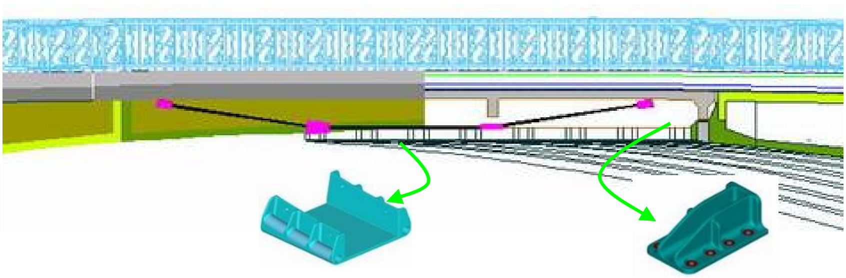

Figure 6. Central span prestressing

For the rehabilitation of the end superstructure were foreseen: the reconcreting of the short cantilevers; the consolidation of the short cantilevers with prestressed bars; the mounting of neoprene bearings; rebuilding of the pavement cantilevers.

Specific equipment works will be executed: rearrangement of the waterproofing, mounting of the tram rail in Sika-Icosit system, concrete layer on the carriageway and protection of the footway with epoxi-polyurethanic system. During the working time the traffic will be interrupted.

It will be strongly taken care of the preservation of the architectural personality of the structure.

In June 2005, anchorages blocks for the prestressed reinforcement, placed on the web, started to be executed. With this occasion unpleasant surprises arose on the web, namely multiple cracks with different orientations, hidden for the eye to see with a cement grout applied on the struture ca. 25 years ago. Considering this, laboratory determinations on concrete samples extracted from the web were ordered (Fig.7).

Figure 7. Extraction of samples and laboratory tests

By examing the extracted samples, the existance of cracks has been confirmed. The laboratory tests of the samples confirmed the low quality of the concrete Rcmed=13.7 N/mm2 (C8/10). The possibility of consolidating the central span with carbon fibres was studied although the chances this system to be a success are minimal because of the low concrete quality of the structure.

Due to the fact that the colmatation of the cracks had a very high incertitude degree (numerous cracks, with different orientations and hidden by cement grout), we made the decision to abandon the consolidation solution of the central bridge deck.

Figure 8. Demounting of the initial central bridge deck

2.1.2. Redesigned solution

The central bridge deck, out of reinforced concrete was demolished through longitudinal fractionation (Fig.8).

After the clearing of the emplacement folowed the consolidation of the short cantilivers by executing horizontal drillings and reinforced bearing blocks out of C25/30 concrete. In the horizontal drillings high tensile alloy steel bars of 20 mm were introduced and tensioned at 120 kN. Thus the rezistance at the shearing force of the short cantilivers was increased (Fig.9).

Figure 9. Consolidation of the short cantilivers by prestressing with high tensile alloy bars

Thus the rezistance at the shearing force of the short cantilivers was increased (Fig.9). The bearing areas, which will become inaccesible after mounting the central bridge floor, were repaired and protected with a Sika multiple layer system (Fig.10).

Figure 10. The short cantilivers were repaired, protected and consolidated

The initial bridge floor out of reinforced concrete, was replaced with a bridge floor realized in a mixed concrete-iron solution, wich respected the initial structure geometry of 9 girders and 4 cross beams, provided with holes designed for the electrical cables and one water pipe. The steel girders are HE 550A (S275) profiles and the cross beams in the field are HE 340A (S275), respectively the end cross beams HE 280B (S275) profiles.

The reinforced concrete slab of a C25/30 concrete reach the compound effect with the steel structure through connectors. The relatively short distance between the girders of the bridge floor (1.80 m) made possibile to realize a concrete slab of 18 cm.

This cental bridge floor was beared on reinforced neoprene bearings, of high reliability.

In contrast to the initial solution, the beneficiary, the Cityhall of Timisoara, asked for the enlargement of the footway, in order to be able to add a cycleway parallel with the pedestrian way. In order to grant the requestement, adequate steel cantilivers were designed which have to support the footway realized out of a reinforced concrete slab of 9 cm. The footway plate will be realized in two steps by using precasted concrete slabs of 4 cm a monolyth concrete layer of 5 cm.

Figure 11. The new steel footway cantilivers

Figure 12. The new chalkstone parapet

In order to realize the protection of the footway the initial solution will be kept. The protection of the pedestrians will be assured with a separating tube breastwork and the pedestrian parapet will be reconditioned and mounted back on the bridge with a adequate, more complex fixation system on the concrete girder.

Finally the entire structure, the anchient reinforced concrete one and also the new mixed structure, will be protected in a Sika system.

3. CONCLUSIONS

Bridges always contributed at the development of the communities, economy and the human society as a whole. For these “Ouvrages d’art” the re-evaluation of the existing structures, concerning aesthetics as well as bearing capacity needed in the future, has to lead to modern solutions, elegant and reliable, so that they won’t create major maintenance problems and they should use the existing elements of resistance.

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TENTH NATIONAL AND
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SCIENTIFIC MEETING

iNDiS 2006

NOVI SAD, NOVEMBER 22-24,2006.

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A NEW OVERPASS IN TIMISOARA FOR CLOSING A TRAFFIC RING

Summary: The traffic in Timisoara city becomes more and more complex, due to the disproportion between the increase of the auto traffic and the capacity of being supported by the existent street network. The wish of the municipality to improve the traffic conditions, is translated through the completeness of one traffic ring over the rail road Bucureşti – Timişoara, by realizing an overpass with a length of approximately 300 m. The paper presents the constructive solutions analyzed for the over passing structure and the variants taken in account for the accesses to the over pass. The assembly over pass - access was analyzed considering the future development of the road network in the northern side of the city, but also considering the existent real estates in the area.

***Key words**: embedded steel girders, over pass, gyration*

NOVI NADVOŽNJAK U TIMIŠVARU NA UKRŠTANJU SA SAOBRAJNIM PRSTENOM-OBILAZNICOM

Rezima: Saobraćaj u Temišvara postaje sve kompleksniji kao posledica disproporcije porasta broja vozila u saobraćaju i kapaciteta postojeće mreže gradskih saobraćajnica. U cilju poboljšanja saobraćajnih uslova ovaj saobraćaj se prebacuje na obilaznicu-prsten koji se prevodi preko železničke pruge Bukurešt-Temišvar, sa nadvožnjakom dužine od oko 300m. U radu je prikazano rešenje konstrukcije i analize nadvožnjaka sa upoređenjem više varijantnih rešenja. Montaža - sklapanje pristupne konstrukcije je analizirana uzimajući u obzir budući razvoj saobraćajne mreže u severnom delu grada, ali i uvažavajući postojeće realno stanje u razmatranom području.

***Ključne reči**: ugrađeni čelični nosači, nadvožnjak, kružni tok saobraćaja*

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1. WHY AN OVER PASS?

The proposed over pass situated in the central area of Timişoara, is situated in the Baader str. (south) and passes the rail road, respectivelly the Demetriade str. The aim is to eliminate the intersection at same level eith the rail road and thus to solve the traffic on the relation north - south, making the connection between the central area and the ring road Arad - Lugoj. The over pass will assure the continuity of ring II over the rail road Bucureşti – Timişoara, which suffers, at the moment, very often barrier closings.

2. CONSTRUCTIVE STRUCTURE

In order to choose a solution for realizing the over passing structures, which should respond the best to the technical functional, aesthetical and economical requests, a series of solutions regarding the superstructure, infrastructure, approaching slopes, road works and road structure, were analyzed.

2.1. Route in plane

For the connection of Demertiade street at the over pass approach slopes and implicit to the route of the ring II, two variants are proposed:

Variant 1

The access to and from the Demetriade street to the over pass approaching slopes will be realized through one sector connected to a gyration with an interior radius of 12,00 m, which will be connected by means of two slip roads to the ring II respectively to the over pass (Figure 1).

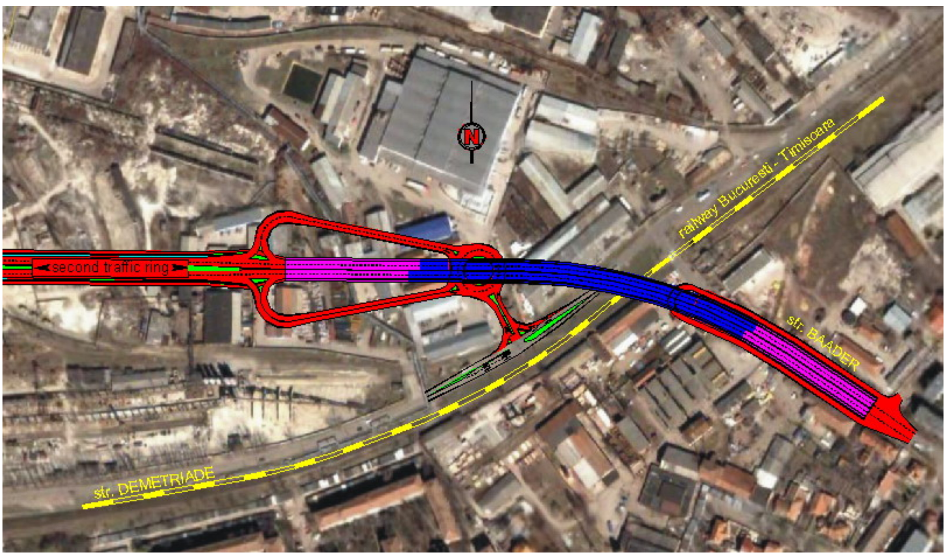

Figure 1. Variant 1 - plane view

Variant 2

The access in and from the Demetriade street will be realized with two sectors, connected independently to the ring II, respectively to the over pass (Figure 2).

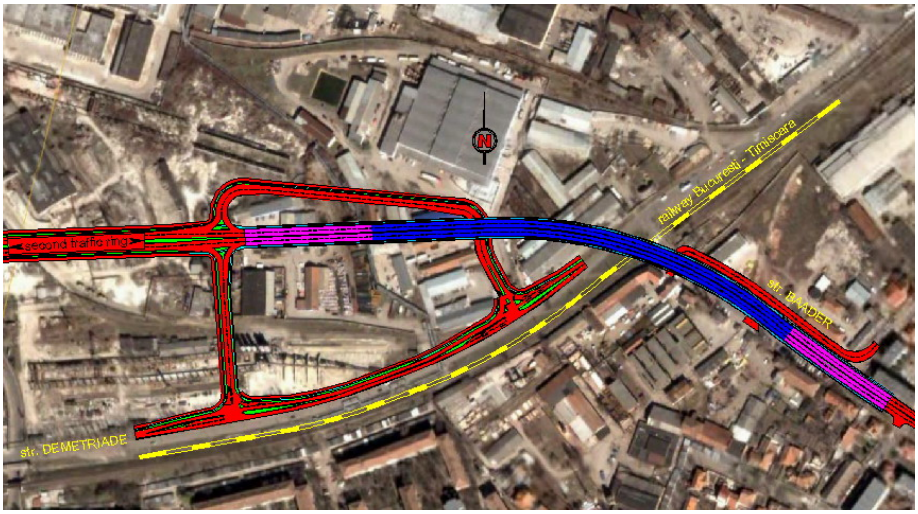

Figure 2. Variant 2 - plane view

The over pass is realized out of two symmetrical parallel structures, situated on a curved route with a radius of 370 m.

2.2. Superstructures

In order to choose the optimal solution, three variants for the superstructures were analyzed:

Variant I - Superstructure with embedded steel girders

In this configuration the maximum length of the superstructures, measured on the exterior is 394 m, and the total length of the over pass including the approaching slopes, measured in the axes of the over pass, is 606 m.

The resistance structure of the superstructure (for one traffic direction) consists of 11 steel HEA girders, embedded in concrete.

The total width of the superstructures for one traffic direction is 9,48 m, with a gauge of 9,00 m.

The total height of the superstructure is 0,94 m.

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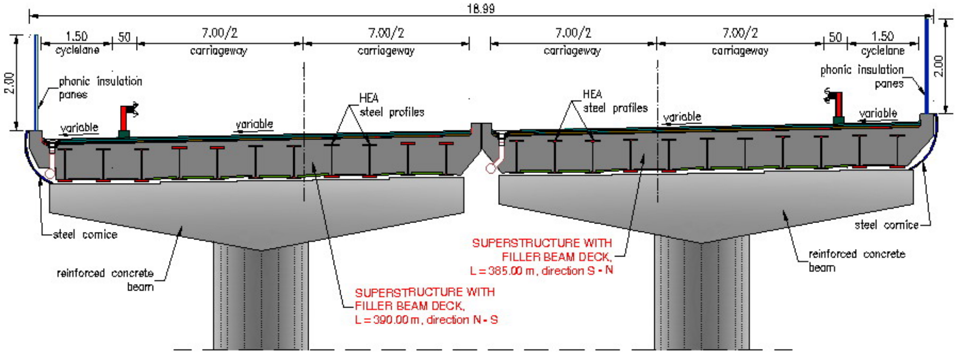

*Figure 3. Superstructure with embedded steel girder*s

Variant II - Superstructure with prestressed concrete girders

The maximum length of the superstructure measured on the exterior is 318 m, while the total length of the over pass, including the approaching slopes, measured in the axes of the over pass, is 672 m.

The superstructure for each traffic direction is realized out of 7 prestressed concrete girders with a height of h = 1,03 m, which work together in a over concreting plate executed out of 12…30 cm thick reinforced concrete. The total height of the superstructure is 1,45 m.

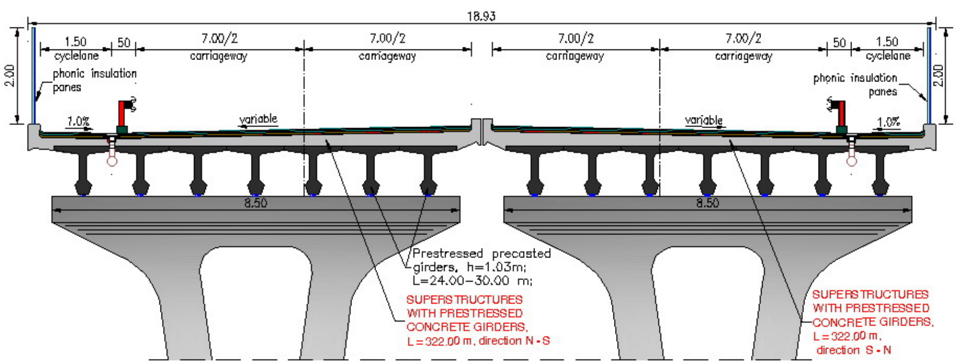

Figure 4. Superstructure with precasted concrete girders

Variant III - Steel - concrete composite superstructure

In this configuration the maximum length of the superstructure is 394 m (measured on the exterior of the over pass), and the total length of the over pass (including the approaching slopes), measured in the axes of the over pass is 646 m.

The 3 full web steel girders work together with a reinforced concrete slab of class C25/30, having a minimum thickness of 22 cm and a variable cross declivity. The total height of the superstructures is 1,30 m.

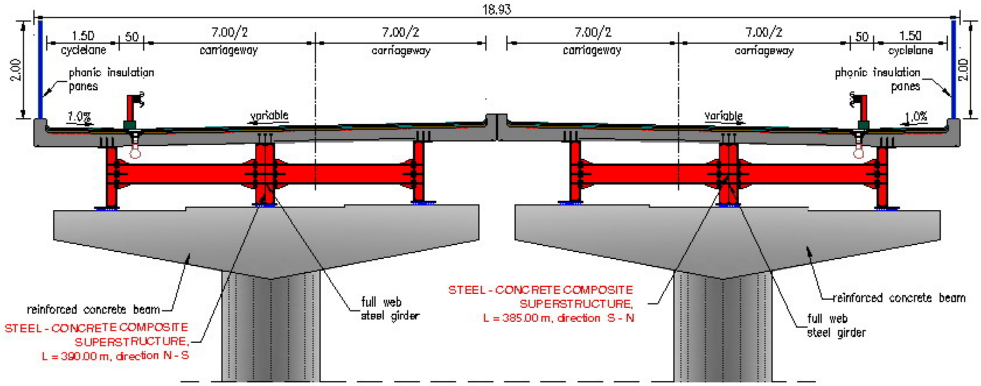

Figure 5. Steel-concrete composite superstructure

Due to the fact that near to the over pass there are several buildings which need to be protected against noise, special panes for phonic insulation have been foreseen on the entire south approaching slopes and on the entire length of the superstructure of the over pass.

2.3. Infrastructure

The elevating of the piers is a result of the request from the Technical Economical Board of the Regional Rail Road Timisoara, that the free height under the over pass in the area where it over passes the rail road to have a minimum of 8500 mm.

For the 1st variant and the 3rd one regarding the superstructure, the elevation of the piers is realized as a circular pier and a girder of variable height.

For the 2nd variant regarding the superstructure, the solution with lamellar piers with cavities, out of reinforced concrete, was chosen.

The foundation of the infrastructures is indirect, made on cast in place piers with big diameter (columns). The columns with a length of approximately 15 m, are distributed on two parallel rows, counting 8 pieces at the abutments and 6 pieces at each pier, at the upper part being embedded in a reinforced concrete girder.

2.4. Approaching slopes

For the northern approaching slope four variants were studied:

Variant I - approaching slope with a reinforced concrete retaining wall, having a cycle lane on the cantilever (Figure 6).

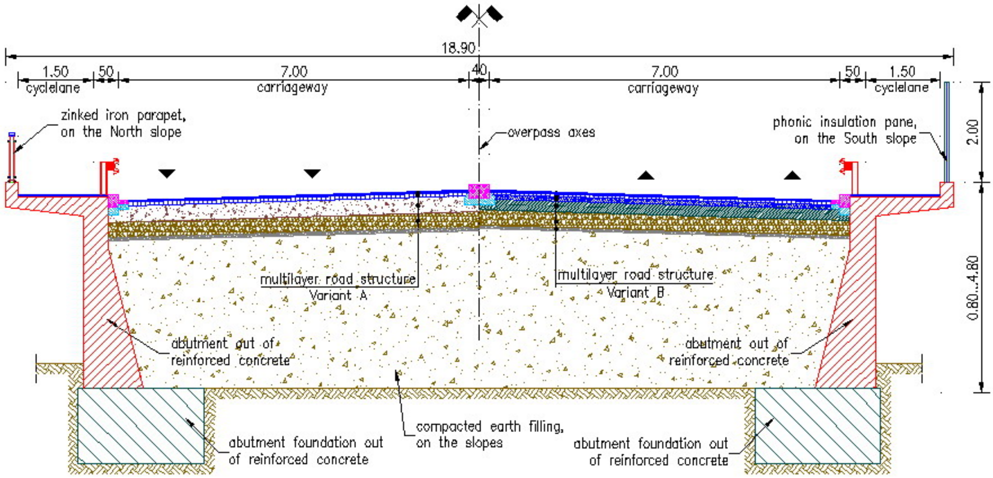

Figure 6.

Variant II - Reinforced concrete approaching slope, without cantilevers for the cycle lane (Figure 7)

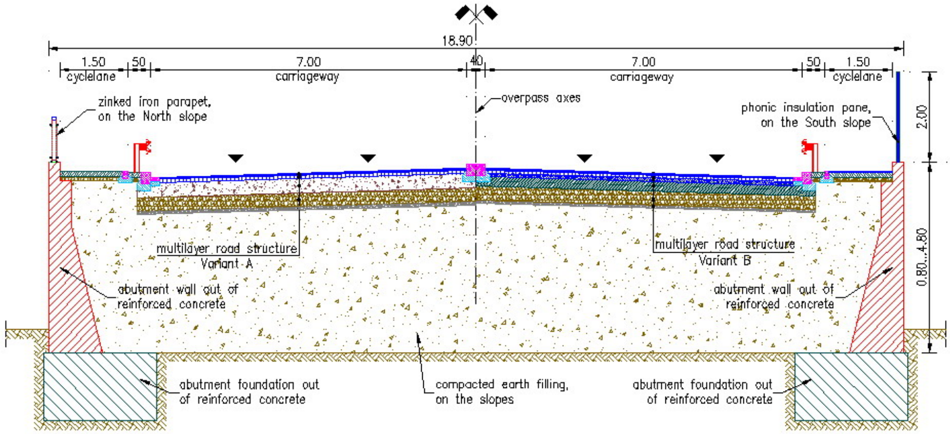

Figure 7.

Variant III - approaching slope with natural chamfering (Figure 8)

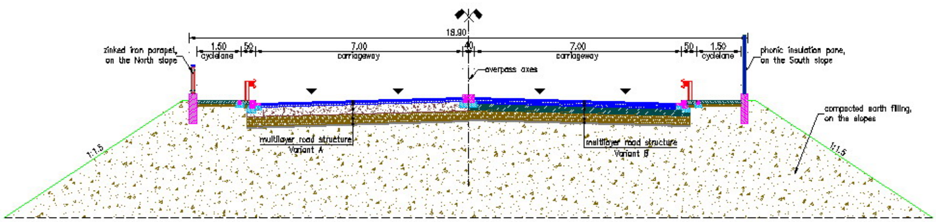

Figure 8.

Variant IV - approaching slope with precasted concrete blocks and earthwork reinforced with geogrid (Figure 9)

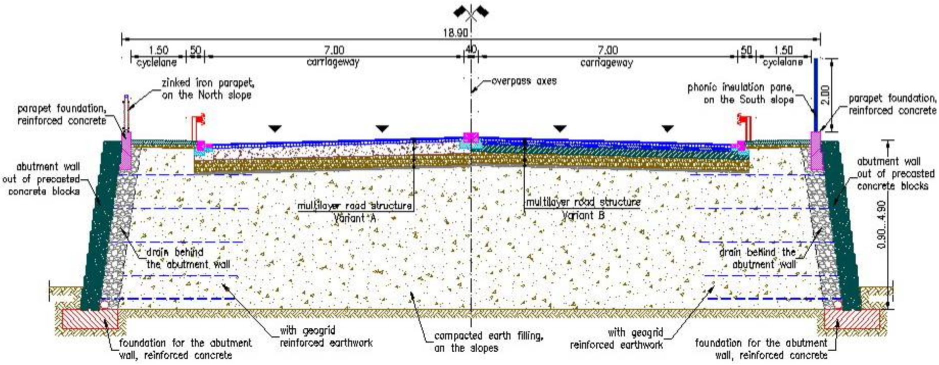

Figure 9.

In order to reduce the occupied surface, the approaching slopes to the over pass realized with reinforced concrete retaining walls, with a cycle lane. The southern approaching slope will be realized in the mentioned solution.

Due to the high number of urban networks in the area, especially on the sector between the approaching slope and the apartment block from the right side, for following the conditions which impose minimal distances between cables and pipelines, only this solution for realizing the approaching slope is accepted.

The study covered also the necessity of regulating the urban network from the over pass emplacement affected by its construction.

From the contrasting study of these variants and considering the legal aspects regarding:

- Transportation facilities, storaging, manipulation and execution for the main resistance elements;
- Maintenance simplicity;
- Higher durability concerning corrosion of the main resistance elements (the prestressed reinforcement suffers degradations faster than a laminated profile with anticorrosive protection);
- Building height,

It is appreciated that the most advantageous variant from technical economical point of view is the variant “Superstructure with embedded steel girders, approaching slopes with reinforced concrete retaining wall with cycle lane in cantilever and road works in the 1st variant”.

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TENTH NATIONAL AND
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iNDiS 2006

NOVI SAD, NOVEMBER 22-24,2006.

Cristina Campian [1]

FINITE ELEMENT MODEL FOR COMPOSITE STEEL-CONCRETE COLUMN

Summary: *In Technical University of Cluj Laboratory, several tests according to ECCS loading procedure were carried out. The tested specimens were composite columns of fully encased type subject to a variable transverse load at one end while keeping a constant value of the axial compression force into them. The complex evolution of the hysteretic cyclic curves representing the transverse force versus the associated displacement up to a pronounced deterioration of the column bases leads to a analytical simulation in finite elements program. A comparation is made and several conclusions end the presentation.*
Key words: *Composite columns, finite elements, monotonic behavior.*

PRORAČUNSKI MODEL SPREGNUTOG STUBA PRIMENOM METODE KONAČNIH ELEMENATA

Rezime: *Na tehničkom univerzitetu u Cluj-Napoca (Rumunija) sprovedeno nekoliko testova prema proceduri opterećenja ECCS. Testirani modeli su spregnuti stubovi sa poprečnim presekom od šupljih čeličnih profila, koji su izloženi promenljivim poprečnim opterećenjem na jednom kraju i konstantom aksijalnom silom. Složeni razvoj histerezisnih cikličnih petlji, koje rerezentuju zavisnost između transverzalne sile i odgovarajućeg poprečnog pomeranja, dovode do analitičke simulacije primenom programa na bazi konačnih elemenata. Sprovedeno je upoređenje rezultata i više različitih zaključaka je prikazano.*
Ključne reči: *Sregnuti stubovi, konačni elementi, monotono ponašanje.*

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1. INTRODUCTION

The experimental program was made for 12 columns with the same cross-section, rallied in three groups of four columns each called SI, SII and SIII according to their length. The testing procedure was the one recommended by ECCS for characterizing the behaviour of steel elements with respect to seismic action (ECCS-TWG 1.3, 1986).

2. TESTED SPECIMENS

The element were fabricated from a Romanian steel section I12 (which is quasi similar to IPN 120 section) fully covered with reinforced concrete including 4Φ10 longitudinal bars (shown in Figure 1). The mechanical model is a cantilever element subject to an axial force N in compression and to a transverse variable force H located at the free top. To ensure a suitable full restraining at the column base, the elements were ended by a sudden cross-section enlargement acting as a foundation (the flexural stiffness ratio between the element and the so-realized foundation was about 1/5). The external dimensions of the composite cross-section were 170x200 in mm. The three column lengths were:

l = 2.00 m for the column series SI,
l = 2.50 m for the column series SII,
l = 3.00 m for the column series SIII.

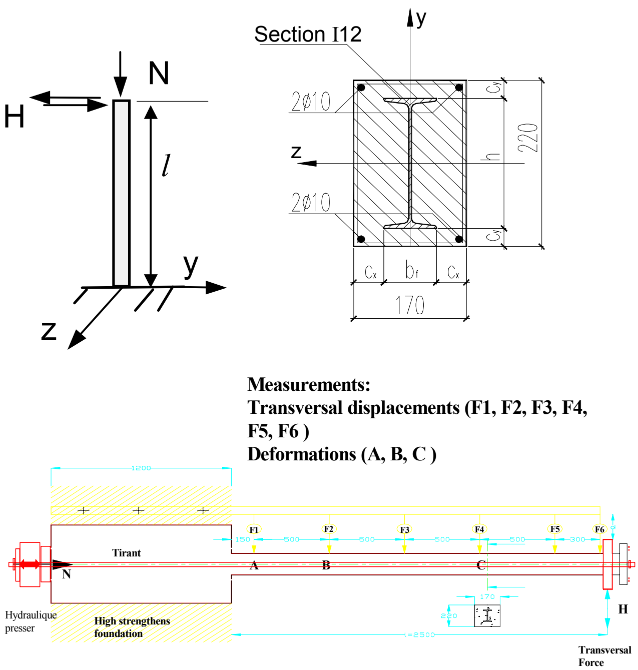

Figure 1. Mechanical model and composite cross-section of the elements

The used materials were:

OL37 structural steel (which may be compared to S235 European grade) for the I12 section;

PC52 reinforcing steel (which may be compared to S550 European grade);

C20/25 concrete class, as defined in Eurocode (for example, see clause 3.1.2 in CEN-EUROCODE 4, 1992).

To satisfy the plastic rotation demand at the column base and to compensate for possible loss of resistance due to spalling of cover concrete, a transverse reinforcement for an efficient concrete confinement was ensured by closed rectangular stirrups of 10 mm diameter with a spacing s of 10 cm along the whole column length as well as the foundation. It should be noted that this spacing is just a bit wider than the value s=7 cm within the critical length l_{cr} = 60 cm of the columns such as specified in Eurocode 8 (see clause 7.6.4 in CEN-EUROCODE 8, 2002) for belonging to class H of high ductility.

Moreover, checks confirmed that the longitudinal reinforcing bars were of high ductility (ε_{su}>5% and f_{su}/f_{sk}>1.08).

3. EXPERIMENTAL RESULTS

The columns were loaded in horizontal position, with the compression axial force N of constant intensity (100 kN or 200 kN) and the variable force H at the free end controlled by imposed displacements, as shown in photo of Figure 2 and Figure 3. Force N does not keep the same direction rigorously because it passes approximately through the base of the column (more exactly through the center of the external face of the foundation).

All the tests were conducted up to the maximum damage of concrete at the column bases; generally, at this stage, buckling of the longitudinal reinforcing bars occurred in the critical zone due to a quasi full deterioration of concrete encasement.

Table 1 collects the different types of test carried out and the maximum values of transverse force $H_u^{(exp)}$ reached under monotonic or cyclic loading.

Figure 2. Testing installation

Tests	l (m)	Axial force N (kN)	Testing procedure	Maximal transverse force $H_u^{(exp)}$(KN)
SI-1	2.00	200	Monotonic	28.1
SI-2	″	200	Cyclic	25.7
SI-3	″	200	(ECCS) with	24.1
SI-4	″	100	elastic limit 40 mm	26.0
SII-1	2.50	200	Monotonic	20.0
SII-2	″	200	Cyclic	19.6
SII-3	″	200	(ECCS) with	20.5
SII-4	″	100	elastic limit 40 mm	20.5
SIII-1	3.00	200	Monotonic	17.2
SIII-2	″	200	Cyclic	17.5
SIII-3	″	200	(ECCS) with	16.5
SIII-4	″	100	elastic limit 40 mm	17.8

Table 1

Figure 3 shows the transverse force H-displacement v curves registered for the three monotonic tests, each curve including a clear softing branch below $H_u^{(exp)}$ due to buckling.

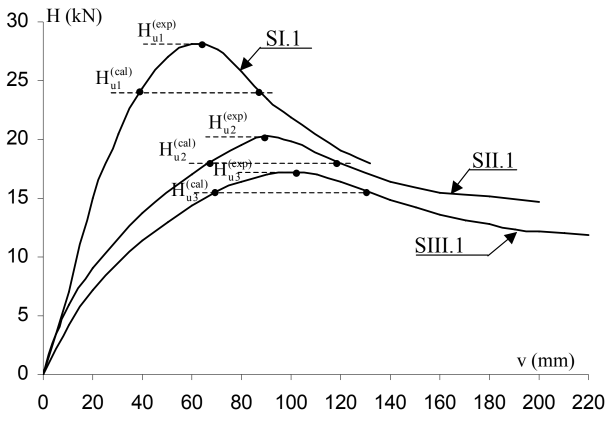

Figure 3. Curves of the monotonic tests

4. NUMERICAL MODELLING USING DRAIN-2DX

4.1. Generalities

The DRAIN 2DX is an EF program developed by Berkley University of California. In this paper the 1.10 version is utilized.

The column has a fixed end and a free end. The realized grid is adapted to geometrical points on which we had measurements points.

The number of points on longitudinal direction x is given by table 2 , for each studied points

Column	No of elements
SI.1	41
SII.1	51
SIII.1	61

Table 2

For the modelisation we chose from the library of EF the « Fiber beam-column element 15 » an inelastic finite element.

This element is recommended for the behavior of elements made from different materials , subjected to compression and flexion.

The transversal cross section of the column is divided in fibers on the web direction.

The element is supposed to elastic in shear and the P-Δ effects can be taken into account

The calculus hypothesis are the next :

The section remains plane after deformation

The behavior in shear is supposed to be elastic.

This is a so called a 'low order' finite element, which give a more stable behavior.

The utilized analyze is that called "event by event". An event corresponds to a change of rigidity of the element, cause by traction, an inelastic uncharged, etc of a fiber. So, the rigidity of the structure is calculated each time, for each "event".

We adopted a behavior model for each material, concrete, steel and reinforcement.

For concrete, we used 5 characteristic points for compression and 2 characteristic points for tension.

For the uncharged part, we used a uncharged factor (FU) of 0.5 , who is very close to the real behavior of the utilized concrete.

For the steel, as tests were made in laboratory, we had the real curves for the behavior of the material. The introduced curves are very close to the ones really obtained

4.2. Modelisation in the cross section

In the cross section, the element was modulated by 30 fibers, fibers called CO1 for the concrete, SO2 for steel and SO1 for reinforcement.

So, on the width of the cross section, the fibers were defined by their surface and their coordinates from the gravity center of the cross section for each fiber.

The reinforcement was assimilated with a very thin layer of steel. The profile steel was assimilated with two flange-layers and many thin layers for the web, alternatively with layers of concrete.

The figure 4 shows the cross section modeling.

The charge was applied in two phases:

A statically one (axial compression of constant value of 00KN),

A « dynamic » one of transversal step by step growing force

The transversal forces correspond to forces values measured in the experimental test.

The experimental displacement- force curve allowed the making of a given force values folder, for the calculus with approximately 100 points.

A lot of tests were carried out to calibrate the model.

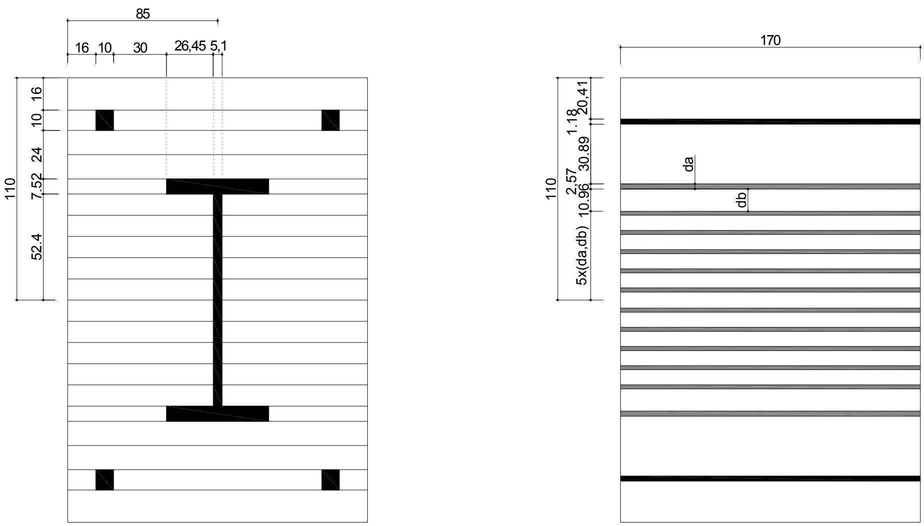

Figure 4 The assimilated cross-section

4.3. Results

For monotonic charge, the next figures give the displacements-forces curves obtained with DRAIN and those obtained by the experimental tests.

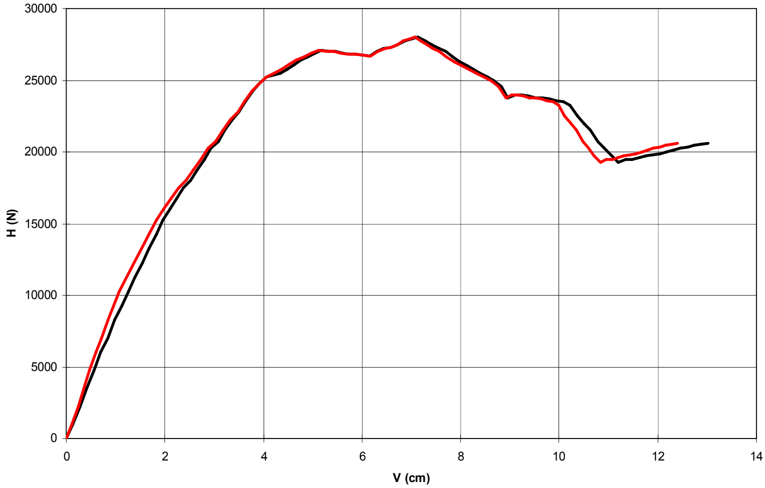

Figure 5. Comparation for the SI.1 column

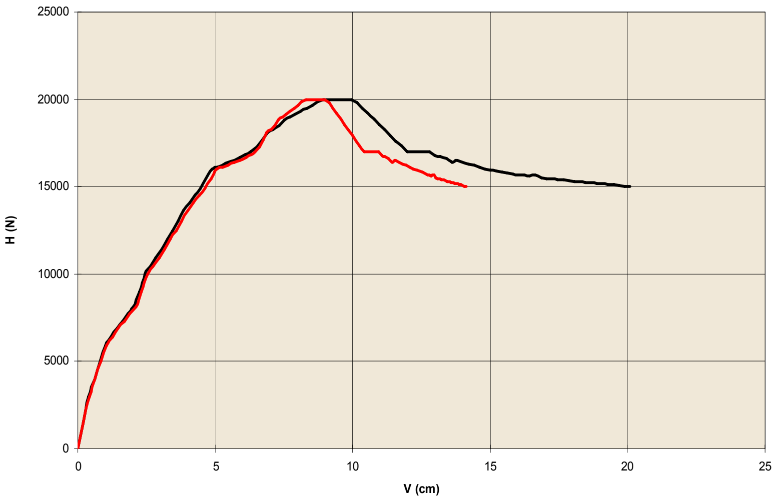

Figure 6. Comparation for the SII.1 Column

It can be observed that the differences in displacements are under the 5% in value of maximal force. The biggest difference appeared on the descending branch, until the considered collapse.

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TENTH NATIONAL AND
FOURTH INTERNATIONAL
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NOVI SAD, NOVEMBER 22-24,2006.

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BEHAVIOUR OF DIFFERENT TYPE OF CONNECTORS UNDER MONOTONIC LOADING

Summary: *The investigation of the composite structures with connection between the concrete and the steel structure is of scientific interest nowadays, due to the economical reasons resulted from the practical application of the solution. In the majority, these solutions are applied in developed countries, and mainly under static gravitational loads. The paper presents the results of a laboratory test program on standard push specimens, on which various parameters have been monitored, such as the type of connectors (UPN profiles, LL profiles, Φ22 shear studs, Φ16 shear studs, hooks of reinforcing bars), steel profile flange class (class 1, class 2 and class 3 respectively) concrete class (C25/30 and C30/37). The results are commented in terms of resistance, ductility and judged in function of their capabilities to sustain a shear deformation between the concrete slab and steel profile.*

Key words: *push tests, connectors, resistance, slip monotonic loading.*

PONAŠANJE RAZLIČITIH VRSTA MOŽDANIKA POD MONOTONIM OPTEREĆENJEM

Rezime: *Istraživanja vezana za spregnute konstrukcije su danas od posebnog naučnog interesa zbog ekonomskih razloga koja proističu iz praktične primene određenih rešenja. Ova rešenja se primenjuju, pre svega, u razvijenim zemljama, uglavnom za slučaj statičkog vertikalnog opterećenja. U radu su prikazani rezultati laboratorijskih ispitivanja sprovedenih na standardnim modelima, pri čemu su nadzirani različiti parametri kao što su tip konektora (UPN profili, LL profili, moždanici ∅22 i ∅16, kuke od armaturnog gvožđa), klasa profila čeličnih nožica (klase 1, 2 i 3, respektivno) i kvalitet betona (C25/30 i C30/37). Rezultati su objašnjeni u smislu otpornosti, duktilnosti i ocene u funkciji njihove sposobnosti da podnesu smičuće deformacije između betonske ploče i čeličnog profila.*

Ključne reči: *ispitivanje moždanika, konektori, otprnost, opterećenje za određivanje proklizavanja.*

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1. INTRODUCTION

Until recently, the design of composite structures has been regarded only as a generalization of the reinforced concrete design, by considering that the reinforced concrete slab remains perfectly connected with the metallic beam. In an obvious way, this point of view imposes the use of a completely rigid connecting system, in order to hinder any relative sliding between the two types of material. Nowadays, there can be distinguished two different types of connections for the composite beams:

- complete connection;
- partial connection.

For the composite beams with partial connection, the relative sliding between the two materials is a compulsory criterion in the design of the structures. Nevertheless, different studies do not recommend the use of the partial connection in seismic areas.

In addition, in order to avoid the introduction of exact behaviour laws at the level of the connectors, the norms operate a very clear distinction between the ductile and the non-ductile connectors. One connector is considered ductile if it presents a sufficient capacity of deformation by sliding, in order to justify the hypothesis of a perfectly plastic behaviour of the connection in shear. If one refers to the curve of the shearing – relative slide for a given connector, as can be deduced from the series of experiments on samples of the push-out type, the previous definition implies the fact that its curve present an elastic-plastic type image with a plastic plateau corresponding to the characteristic resistance $P_{R,K}$ of the connector and to a ultimate sliding capacity δ_u considered as important. Eurocode 4 considers that a value of δ_u superior or equal to 6mm allows in the practice the definition of a connector as ductile, with the additional condition that there is a sufficient degree of connection depending on the opening of the span of the beam.

For the push-out type tests, the Annex B of the Eurocode 4 allows the calculus of non-standardized connectors starting from laboratory tests. These tests must bring information about the nature of the connection of shear, needed for the design of composite sections.

We have to mention that this method of testing doesn’t provide any information on the behaviour under seismic actions. This implies the necessity of a consideration of the present testing method of the Eurocode, in view to the testing of some new categories of connectors. Such an approach is in fact going on in European countries that have seismic activity.

The present paper reports the results of eleven push shear specimens under monotonic loading. This represents the first part of an ongoing research activity on the behaviour of shear studs that is under progress in the CEMSIG Laboratory at the Politehnica University of Timisoara, Romania. The results of the monotonic behaviour would be completed by the investigation of the same type of tests under cyclic loading and a numerical simulation by Finite Element Methods. The final purpose of the research activity is to find the seismic behaviour of different types of connectors under cyclic loading as is induced by seismic activity.

2. SPECIMENS, TESTING SET-UP AND INTERPRETATION OF RESULTS

2.1. General configuration of specimens

The dimensions of specimens for the push-out standard tests (including the metallic section and the reinforcing plan) are given in Figure 1. The concrete and steel profile dimensions were kept constant for all the tests, as well as the configuration and diameter of the reinforcing bars (Φ 10mm) according to Figure 1.

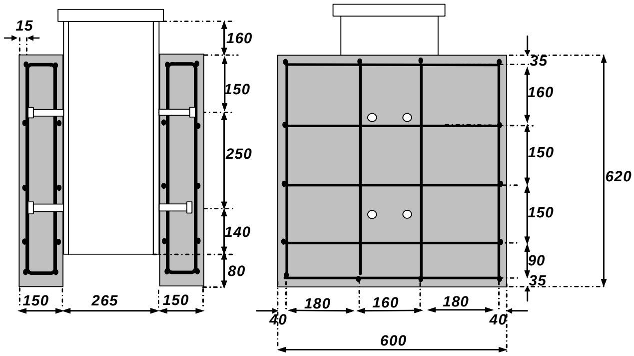

Figure 1. Test specimen for standard push test according to Eurocode 4

2.2. Description of specimens

Table 1 shows the description of push-out specimens. Mainly, three parameters have been taken into consideration in the conception of specimens:

- type of connectors (Φ16 headed studs on two rows; Φ22 headed studs on one row; reinforcement anchor hooks of Φ 10mm, perforated steel plate; L120x80x8 corner profile and UNP 120 channel profile);
- concrete strength class (C25/30; C30/37);
- steel profile class (class 1 corresponding to standard HEB 260 profile, class 2 by considering a 10mm steel flange and class 3, by considering a flange of 8mm respectively).

Figure 2 shows a 3D view of the steel specimens.

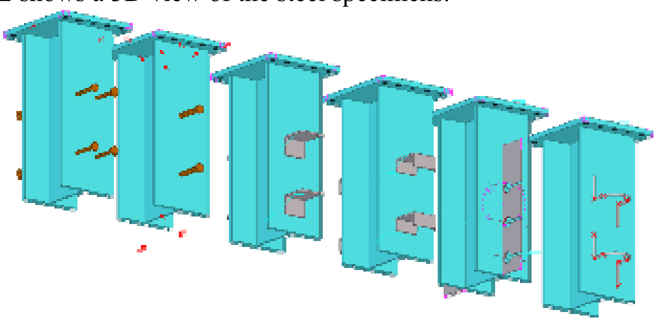

Figure 2. Disposition of connectors on steel specimens.

SPECIMEN	Type of connectors	No. of connectors	Concrete class	Steel profile
PT-16/I-M	8Φ16 (2 rows)	8	C25/30	HEB 260
PT-16/II-M	8Φ16 (2 rows)	8	C25/30	Class 2*
PT-16/III-M	8Φ16 (2 rows)	8	C25/30	Class 3**
PT-16/S-M	8Φ16 (2 rows)	8	C30/37	HEB 260
PT-22-M	4Φ22 (1 row)	4	C25/30	HEB 260
PT-A-M	Reinforcement anchors (Φ 10mm)	4	C25/30	HEB 260
PT-A/S-M	Reinforcement hooks (Φ 10mm)	4	C30/37	HEB 260
PT-II-M	Perforated steel plate ***	2	C25/30	HEB 260
PT-LS/II-M	L120x80x8	4	C25/30	Class 2*
PT-LS/III-M	L120x80x8	4	C25/30	Class 3**
PT-US-M	UNP 120	4	C25/30	HEB 260

** 260X260 profile t_f=10mm*
*** 260X260 profile t_f=8mm*
**** longitudinal steel plate (t=8mm) on each side of steel profile, perforated for the passage of reinforcement*

Table 1. Description of push-out specimens

2.3. Testing arrangement and loading procedure

Figure 3 shows the testing set-up and the disposition of the displacement transducers. They have been disposed so that to measure the relative slip between the concrete slabs and the steel profile, but also the separation between the two elements. The load is applied through a compression actuator in displacement control.

Figure 3. Testing arrangement and disposition of displacement transducers

The load was first applied in increments up to 40% of the expected failure load and then cycled 25 times between 5% and 40% of the expected failure load, in accordance to the Eurocode 4 stipulations. Subsequent load increments have been imposed up to failure.

2.4. Tests evaluation

Figure 4 shows the basic interpretation of the results. The connector shear capacity $P_{R,k}$ represents the maximum load capacity reduced by 10% and divided to the number of shear connectors. The connector's slip capacity is taken from the load-slip deformation curve, as corresponding to the shear capacity $P_{R,k}$ (see Figure 4).

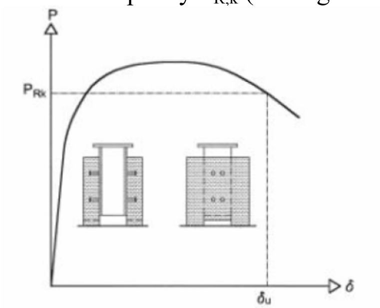

Figure 4. Determination of slip capacity δ_u according to Eurocode 4.

3. EXPERIMENTAL RESULTS

SPECIMEN	F_y [kN]	δ_y [mm]	$S_{j,ini}$ [kN/mm]	F_{max} [kN]	δ_u[mm]	$P_{R,k}$/conn. [kN]
PT-16/I-M	396.0	0.20	1983.4	801.1	4.91	100.1
PT-16/II-M	514.6	0.37	1349.8	862.6	7.52	107.8
PT-16/III-M	501.2	0.34	1399.6	831.8	6.42	104.0
PT-16/S-M	436.2	0.17	2359.8	844.0	7.18	105.5
PT-22-M	405.8	0.35	1093.5	737.6	14.3	184.4
PT-A-M	396.0	0.20	1813.0	590.6	7.92	147.7
PT-A/S-M	356.9	0.08	4331.1	649.6	6.52	174.0
PT-II-M	557.5	0.13	3701.4	1033.0	21.9	516.5
PT-LS/II-M	586.8	0.21	2677.1	794.4	2.35	198.6
PT-LS/III-M	494.6	0.10	4726.9	790.4	3.09	197.6
PT-US-M	804.9	0.41	2052.9	1256.0	9.65	314.0

Table 2. Main test results derived from the interpretation of experimental data.

Table 2 presents the main interpretation results derived from the monotonic curves (given in Figure 5), in which:

- F_y represents the yielding force, in the sense of ECCS loading procedure (1986);
- δ_y the corresponding yielding displacement;
- $S_{j,ini}$ the initial stiffness of the F-δ curve;
- F_{max} is the maximum recorded force during testing;
- δ_u is the slip capacity of the connectors;
- $P_{R,k}$ is the connector's shear capacity.

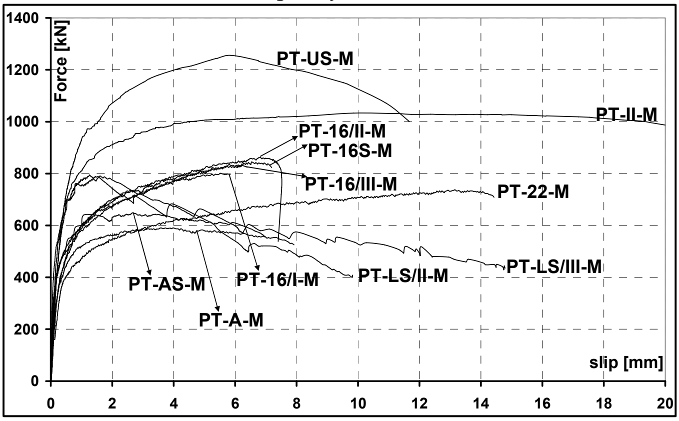

Figure 5. Comparison of the monotonic curves of push-out tests.

The results of the monotonic curves are quite dispersed, ranging in terms of maximum applied load from 590kN (PT-A-M specimen) to 1256kN for PT-US-M specimen. In the same time, the slip capacity range from 2.35mm (PT-LS/II-M) to almost 22 mm (PT-II-M) specimen. In the following, we will make an analysis, function of the parameters varied in the conception of specimens.

3.1. Influence of type of connectors

For the purpose of our study, six types of different connectors were used. All of the six connection typologies are presented in Figure 6, having the same steel profile cross-section and concrete class.

If talking in terms of resistance, the specimen having channel connectors have shown the greater resisting force, while the specimen with anchoring bars had the resistance less than half from the first specimen. In terms of ductility, the PT-II-M specimen had the best ductility, but it has to be noted the fact that it represents the ductility of reinforcing bars - concrete system (the failure in this case was by crushing of concrete nearby the perforated steel plate).

The specimen having LL120 connectors behaved rather badly, the failure being in this case very early by puling-out of the corner connectors from the concrete. This is in fact demonstrated by the very rapid descending of the monotonic curve characteristic to the specimen.

The headed stud connectors proved an expected behaviour, by a rather good ductility and a resistance according to their classic design. However, there are quite important differences between the two specimens (PT-16/I and PT-22) although their shear area are almost the same, both in terms of resistance and ductility.

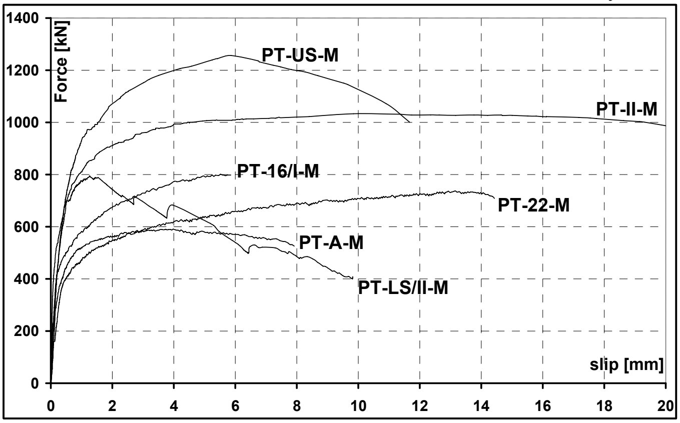

Figure 6. Force-slip curves on different types of connectors.

3.2. Influence of concrete class

The differences between the characteristics of specimens with different concrete classes could be seen in Table 2 and are shown graphically in Figure 7 (only for specimens PT-16/I and PT-A there have been conceived similar specimens with a higher concrete class).

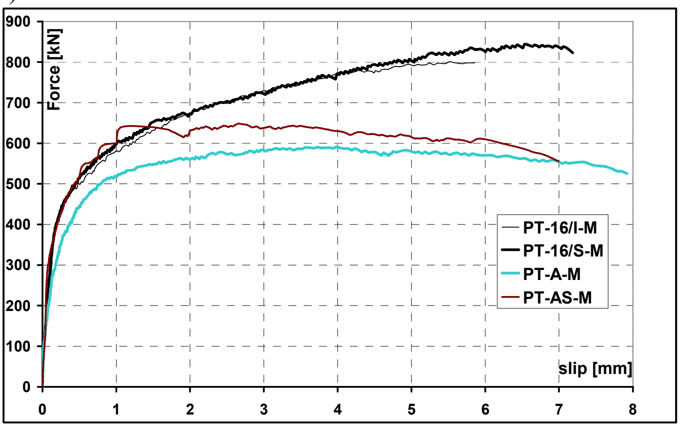

Figure 7. Force-slip curves showing concrete class influence

As the computed failure was in these cases by shear failure of the steel connectors and not by concrete crushing, there is no clear evidence in the increase of the specimen's shear resistance, neither of the ductility with the increase of the concrete class. However, the initial stiffness of the curves specific to the specimens with C30/37 concrete class is significantly greater than the usual concrete specimens and seems to be the only influence of the concrete class in these cases.

3.3. Influence of steel flange class

The 16mm headed stud and the L specimen series have been chose to have different thicknesses of steel flange (17.5mm for HEB 260 profile representing the class I, 10 mm for class II and 8mm for class III respectively). The results given in Table 2 and also graphically shown in Figure 8, shows the fact that there is no major difference among the series specimen's behaviour. This is valid for both resistance and ductility. However, the initial stiffness is rather different within the specimens of each series, but this does not follow a certain rule with the change of the thickness of the steel flange.

It seems that the reduction in the steel profile flange thickness should be more important (to about 6 or even 4mm) in order get a logical increase of the ductility due to local bending of the flange in the vicinity of the connector.

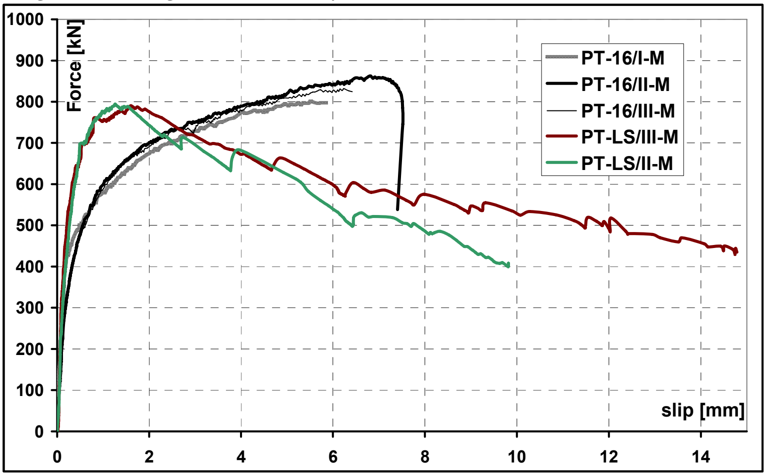

Figure 8. Force-slip curves showing steel flange class influence

3.4. Failure modes

Figure 9. Typical shear failure for headed studs.

The Figures 9 to 11 shows the typical failure modes for different types of connectors.

In case of shear studs (Figure 9), although the global behaviour could be characterized as ductile, the failure was brittle in nature, by the shearing of the headed studs situated on one side of the specimen. It is to be added that there are not evidences of concrete crushing, with the exception of the base of the shear stud.

The LL and UPN specimens, the failure occurred by bending of the shear profiles (Figure 10), but without a real fracture of the steel material. In the case of LL specimens, the flange embedded in concrete was pulled out very rapidly from the concrete after reaching the maximum load. Maybe the use of a sear reinforcement through the embedded flange could retard the failure. Besides, the channel connector was very well embedded in concrete, and in this case, the failure mode was combined, by local crushing of the concrete, and bending of the channel web near the welding to the steel flange.

Figure 10. Shear failure for LL and UPN profiles

Failure mode of specimens having hook reinforcement (Figure 11) was by shear of the reinforcement hooks. No crushing of concrete was observed. In nature, this type of failure was considered to be ductile, due to the fact that not all the shear fractures were in the same time.

Figure 11. Shear failure type for reinforcement anchor hoops and for profiled steel plate.

The CP-II-M specimen represents a special case, due to the fact that the failure was not belonging to the steel connector, but to the concrete slab and reinforcement. Practically, it acted as a „knife“ in the body of the concrete slab. The first signs of the failure were by longitudinal cracks of the concrete slabs in the middle of the slabs, and continued by the crushing of concrete at the bottom of the slab. After loading, there was found that the transversal reinforcing bars were bent by the perforated steel plates. In nature, this type of connection gave a ductile behaviour and in this situation, for obtaining a better resistance, more reinforcement should be disposed into the concrete slab.

4. CONCLUSIONS

The presented paper reports the results of eleven monotonic push-out tests with three main parameters varied namely the type of connectors, the concrete resistance class and the steel flange class of the steel profile:

- the variation of connectors produced practically great differences in terms of Force-Slip curve response, as shown in Table 2 and Figure 5. The best performance in terms of resistance was obtained for the channel connectors, followed by the specimen with perforated steel plate. A bad performance in terms of ductility was obtained for LL specimens, which should be avoided without using additional transversal reinforcement passing through the LL flange. The headed stud specimens provided a good response both in terms of resistance and ductility, in accordance to their well-known design characteristics and expected failure. The anchor hook specimens provided a good global ductility but a rather small resistance. For the practical use of engineers, this solution should be applied in the cases where the composite floor is attached to the steel beam, but not computed as a composite one;
- the influence of the concrete class is evident in the studied cases, but only in what concerns the initial stiffness of the Force-Slip curve. The other parameters seem not to be affected by the change of concrete class, unless the failure mode of the specimens is changed (by crushing of concrete for example);
- for the studied specimens, the change in the steel flange class (thicknesses from 17.5, 10 and 8 mm respectively) do not affect the response in terms of Force-Slip deformation curve.

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GEOTECHNICAL PROPERTIES OF LOESSIAL COLLAPSIBLE SOILS ALONG THE LOWER PART OF RIVER DANUBE BESIDES BELGRADE

Summary: *Loess soil of different thickness spreads over the entire teritory of Belgrade. It is often more than 15 m thick, especially in places where erosion, landsliding or human activities have not destroyed it. The largest thickness is upstream of the Sava river mounth where its height is till 35 m and where loess soils form the large Belgrade's plateau.*
The real finding about machanical behaviour of loessial colapsible soils is very important for correct foundation in it. This is shown on one case example.
In this paper results of extensive investigations on mechanical behaviour of plateau loess soil are presented. Traxial CD tests on samples d=100 mm, h=200 mm,consolidated under a large range of cell pressures were carried out on three representative loess types It was concluded that macroporus loess behaviour depends largely on consolidation pressurees and moisture contents. The stress-strain behaviour of loess investigated has been presented graphycally.

Key words: *macroporous loess, Belgrade's plateau, triaxial test, consolidation pressure, moisture*

GEOTEHNIČKA SVOJSTVA KOLAPSIBILNOG LESA PORED DUNAVA U OKOLINI BEOGRADA

Rezime: *Lesno tlo različite debljine prostire se na čitavoj teritoriji Beograda. Njegova debljina često je veća od 15 m i to posebno na onim mestima gde ga erozija, klizanje ili ljudska aktivnost nisu uništili. Najveća debljina lesa je pored Dunava, uzvodno od ušća Save gde iznosi i do 35 m, formirajući tako veliki beogradski lesni plato.*
Poznavanje fizičko mehaničkih karakteristika lesa vrlo je važno zbog problema fundiranja objekata na njemu. To je pokazano na jednom primeru.
U ovom radu prikazani su rezultati obimnih istraživanja mehaničkog ponašanja lesa sa ovog platoa. Urađeni su brojni triaksijalni CD opiti na uzorcima prečnika d=100 mm i visine 200 mm –u širokom opsegu konsolidacionih pritisaka. Ova ispitivanja vršena su na tri reprezentativna tipa lesa. Na osnovu njih je zaključeno da ponašanje kolapsibilnog -makroporoznog lesa u velikoj meri zavisi od konsolidacionih pritisaka i vlažnosti lesa. Naponsko-deformacijsko ponašanje ispitivanog lesa prikazano je grafički na odgovarajućim dijagramima.

Ključne reči: *makroporozni les, Beogradski plato, triaksijalni opit, konsolidacioni pritisak, vlažnost*

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1. INTRODUCTION

Loess soil is widespread in Serbia. It covers approximately 18000 km^2 and that is about 20% of the country (Fig 1). From the geographical point of view it could be separated in two area units. The first is Pannonian plain. This is the largest area of macroporous collapsible loess which covers approximately 15500 km^2 and that is about 17.5% of the country. River valleys make the second unit in which loess soils are like oases, mostly at river terraces.

Fig. 1 Distribution of loess soil in Serbia

Along river Danube the thickness of loess is usually several meters (3-6 m), reaching sometimes up to 15 m. The largest thickness is besides Belgrade, upstream of the Sava river mouth, where its height is till 35 m and where loess soils form the large Belgrade's plateau*. At this plateau eolian loess sediments make steep right bank of the Danube river (Fig. 2). Plateau's elevations vary from 90-115 m. The loess sediments incorporate, depending on the site elevation, three to five loess levels separated respectively by two to four levels of brown soils.

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* In Serbian literature it is often referred as Zemun loess plateau, because that suburb of city is named Zemun.

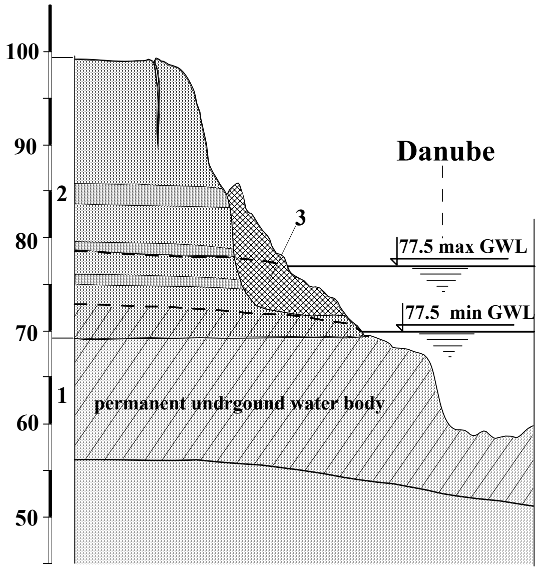

Fig. 2 Belgrade loess plateau: Typical cross section perpendicular to the right bank of the Danube river -upstream of the Sava river mouth

Legend: 1 fluvial sediments 2 eolian deposits3 slumps

The first and the second loess levels are always above the Danube river maximum level and above oscillation zone of existing ground water level, too. The other loess levels are in the permanent ground water zone.

2. ENGINEERING PROPERTIES OF LOESSIAL COLLAPSIBLE SOILS

Belgrade loess plateau is for years one of basic directions of the city spreading (Marković, 1984). In seventies there had been built a number of tall apartment houses which were shallow founded and it caused many foundation problems.

In the paper will be given an example of a 13-storey apartment building which had been built from May 1971 till August 1972 (Koprivica, 1975). The building was 1.5 km away from right bank of the Danube and the elevation of surrounding ground surface was about 97.0 m. It was founded on strip footings and contact pressure was 130 kPa. The site investigations have determined that the thickness of macroporous loess below the ground surface was about 15 m.

Calculations of settlements which were performed on the assumptions of homogeneous, isotropic and elastic half space have shown the maximum amount of 11 cm and differential settlements up to 3 cm. It should be noted that loess samples were taken by auger and they were mechanically disturbed. Accordingly, deformation properties obtained from laboratory tests and used in the settlement analysis were not correctly estimated.

Survey monitoring, however, had recorded the values which were several times larger than calculated ones. The latter analyses had shown that these settlements were caused by nonuniform and subsequent wetting up of loess beneath foundation level.

Namely, during construction and even after its completion, the ground surface around building was not adequately arranged. This allowed rainwater to infiltrate into the subsoil. Survey was performed during two years -from December 1971 till December 1973 (Fig. 3).

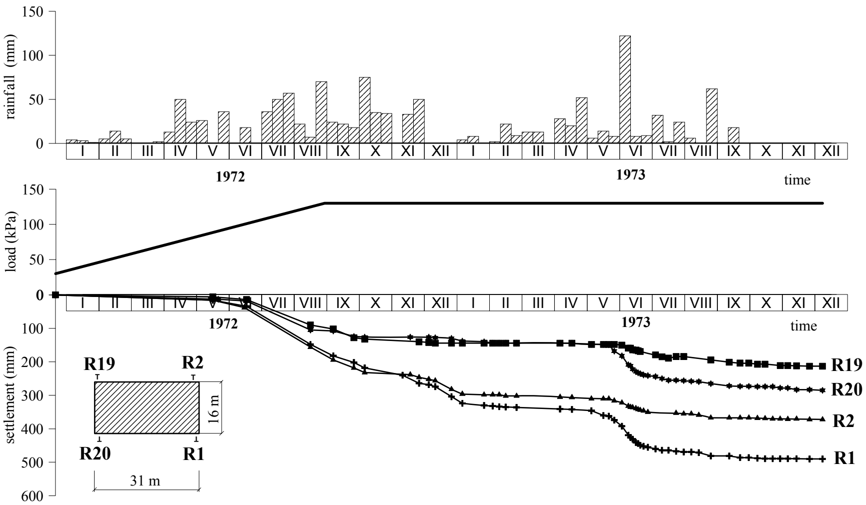

Fig. 3 Relationships between time and recorded data

a) Rainfalls (January 1972 - September 1973) b) Loading of 13-storey building (December 1971 - December 1973) c) Surveyed settlement of the monitoring points of 13-storey building (December 1971 - December 1973)

The values of maximum displacements of the monitoring points are given in Table I.

Monitoring point	Settlement (mm)
R-1	490
R-2	372
R-19	213
R-20	285

Table I. Settlements of the four monitoring points of 13-storey building shown in Fig. 3

In April 1975 survey was done once again and the following extreme displacements were recorded:

Maximum settlement: R-1 =510 mm

Minimum settlement: R-19=227 mm

This and some other similar examples caused the need for systematic investigations of macroporous collapsible loess soil (Jennings & Knight, 1975). The investigations have been performed in the Soil Mechanics Laboratory at the Belgrade Faculty of Mining and Geology in the last twenty years period. The laboratory tests were carried out on the undisturbed loess samples taken by piston tubes. Results of these investigations have shown that according to phase composition, for two highest loess horizons which are above the maximum ground water level, there are three representative loess types (Fig. 4). Their average physical properties are given in Table II.

Type of Loess	A	B	C
Unit weight (kN/m^3)	14.5-16.0	16.0-17.5	13.2-15.8
Dry unit weight (kN/m^3)	11.5-13.5	13.0-15.0	12.0-14.5
Water content (%)	16-23	16-23	6-12
Porosity (%)	50-55	45-51	46-54
Degree of saturation (%)	41-54	54-65	18-31
Unit wight of solid particles (kN/m^3)	26.8		
Clay fraction (<0.002mm) (%)	8-10		
Silt fraction ((0.02-0.002mm) (%)	45-60		
Liquid limit (%)	30-40		
Plastic index (%)	10-18		
AC Classification	CI/CI		
Dominant clay minerals	Illite and montmorillonite		
Consistency index	0.9-1.10		
Colloidal activity	>1.25		

Table II Average physical properties of macroporous loess soil

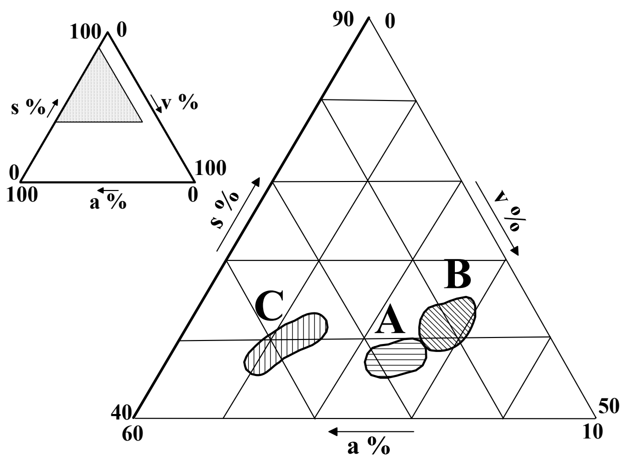

Fig. 4 Phase composition of macroporous loess soil

For successful foundation on macroporous loess it is not enough to identify it as collapsing soil, but also its stress-strain relationships must be determined. In such a way it is possible to define mechanical behaviour of loess during the application of load and to determine interaction between the terrain and the building. Therefore, representative samples of a diameter 100 mm and a height 200 mm were examined by standard consolidated drained compression tests. In order to define properly their behaviour each specimen was subjected to one cycle of unloading and reloading. All tests continued until the samples have been broken (Ćorić, 1995).

The results so far obtained have shown that at moisture content $w<w_p$ i.e. Ic>1, the stress-strain behaviour of macroporous loess highly depends on consolidation pressure. So, in a case when the pressure was less than the corresponding critical stress state, loess behave like a stiff material while, for consolidation pressures above these ones, its behaviour was plastic. Such behaviour is the consequence of failure in the internal loess structure exposed to the corresponding critical stress state i.e. the state of stress which separates ranges of "low" stresses and "high" ones. This is due to the different intergranular bonds partly by hard $CaCO_3$ coating and partly by comparatively softer i.e. weaker hydrocolloidal (clayey) films. Resistance to deformation and rupture which offer dominating rigid calcareous bonds ($CaCO_3$ 15-20%), controls the value of critical consolidation pressure.

It should be noted that the difference in density between types of loess A and B did not influence on the value of critical pressure. Accordingly, loess samples with w=18-23% behave for $\sigma_3'\leq 75$ kPa like a stiff material and for $\sigma_3'\geq 100$ kPa like a plastic one (Fig. 5 and 6). But, almost the dry loess with w=6-12% (type C) for $\sigma_3'\leq 100$ kPa behave like a stiff material and only for $\sigma_3'\geq 300$ kPa like plastic one (Fig. 7). Besides, any decrease of moisture content causes a high increase of Young's modulus.

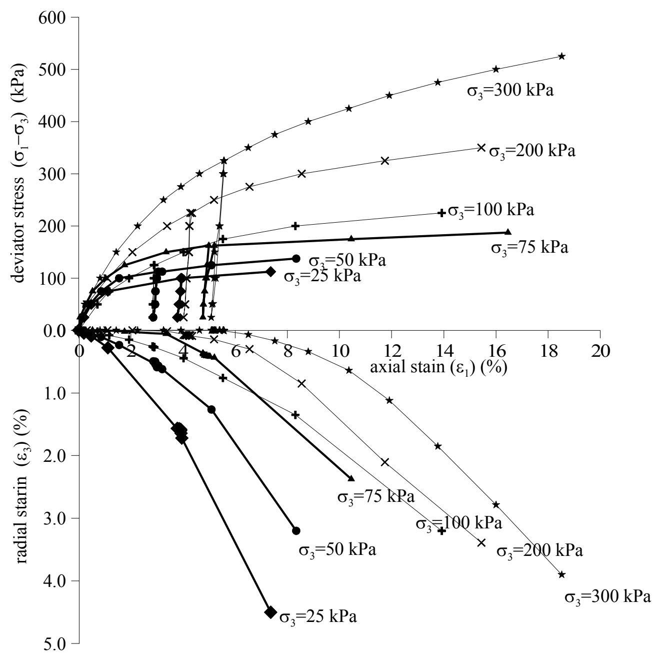

Fig. 5 Stress-strain relationships for type of loess A

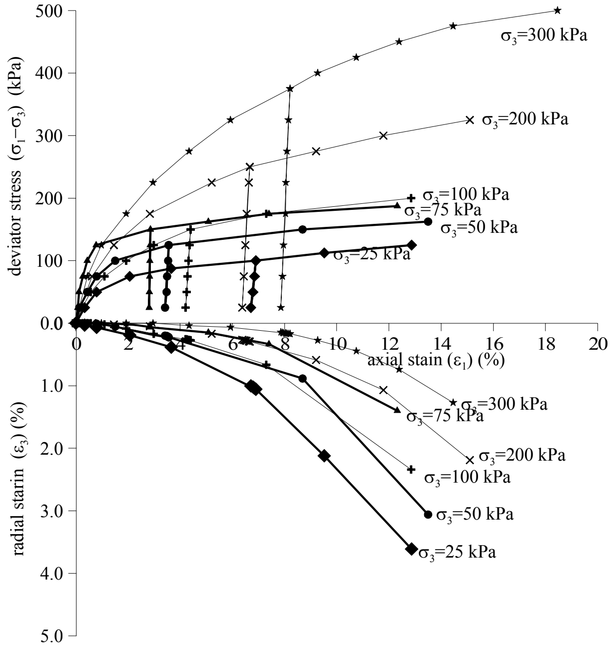

Fig. 6 Stress-strain relationships for type of loess B

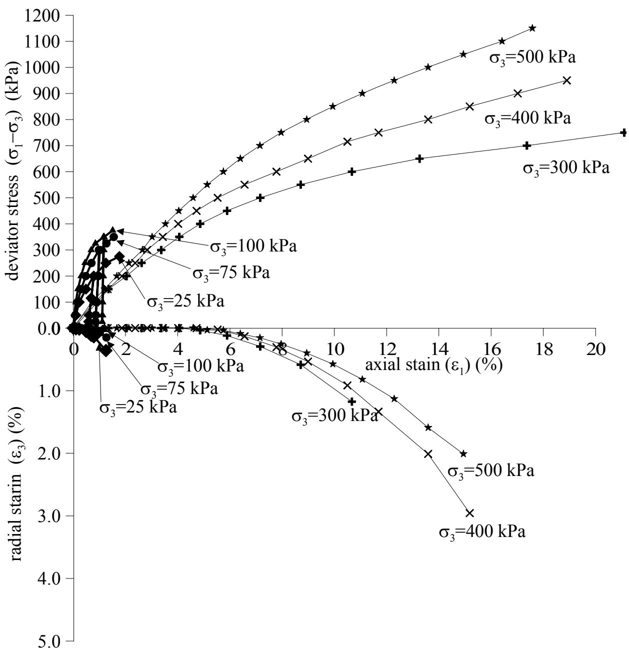

Fig. 7 Stress-strain relationships for type of loess C

The results obtained have clearly shown that the decrease of moisture content increases the critical stress. In accordance with that, our investigations have confirmed that when moisture content approaches the plastic limit and overtakes it ($w \geq w_p$),loess behaviour is plastic through the entire range of consolidation pressures. This is the consequence of successive failure in the internal loess bonds caused by the critical moisture increase from w till w_p, sometimes for only 2-3% -not being dependent on stress state. This is due to the presence of sensitive clayey bonds. Although the least in loess, often even neglected, when slightly moisture clay particles swell thus breaking the stiff calcareous links and changing the primary macroporous loess structure.

3. CONCLUSION

The failure of loess structure, either caused by critical stress state or critical moisture increase is followed with large volume change which may cause high total and differential settlements of buildings. Because of that macroporous loess has to be protected of structural failure i.e. retaining it in the range of low stresses and protecting it from subsequent increase of moisture content. If it is not possible, shallow foundations have to be replaced with pile foundations or some stabilization method of loessial collapsible soils has to be used.

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THREE-DIMENSIONAL EFFECTS OF SLOPE STABILITY ANALYSIS

Summary: *Slope stability analysis is, in fact, three-dimensional problem. This is a consequence of unregular shape of sliding body and it is characteristic, expecially, for landslides in Serbia –for example large landslides: Umka , Duboko, Beška etc.*
However, till now a days, there isn't the generalized procedure of three-dimensional stability calculations which involve slip surfaces of arbitary shape. Instead of that, it is common to make two-dimensional analysis for a most critical cross section throughout the three-dimensional sliding mass.
But if the lateral effects i.e. three-dimensional effects of the slope are significant than they should be involved in the stability analysis. In this paper are presented approximate procedures of introduction three-dimensional effects in stability calculations and a three-dimensional infinite slope stability analysis, too.
In the cases where the the three-dimensional effects are cosiderable, their introduction in stability calculations ensure to obtain more correct findings about stability of slopes. This is expecially important in evaluating remedial measures for landslides.

Key words: *slope stability analysis, three-dimensional effecs, approximate procedures*

TRODIMENZIONALNI EFEKTI ANALIZE STABILNOSTI PADINA I KOSINA

Rezime: *Analiza stabilnosti padina i kosina je u sušini trodimenzionalni problem. To je neposredna posledica nepravilnog oblika kliznog tela i posebno se odnosi na klizišta u Srbiji –kao na primer na velika klizišta: Umka, Duboko, Beška i dr.*
Međutim, sve do danas, nije razvijen opšti postupak trodimenzionale analize stabilnosti koji bi važio za klizna tela proizvoljnog oblika. Umesto toga, najčešće se vrši dvodmezionalna analiza kritčnog poprečnog preseka trodimenzionalnog kliznog tela.
Međutim, ako su bočni tj. trodimenzionalni efekti padine značajni, onda i njih treba uključiti u analizu stabilnosti. U ovom radu prikazani su aproksimativni postupci koji uvode trodimenzionalne efekte u proračun stabilnosti, a urađena je i trodimenzionalna analiza stabilnost beskonačne padine.
U slučajevima kada su trodimenzionalni efekti značajni, njihovo uključivanje u analizu stabilnosti omogućava da se dobiju realnije podaci o stabilnosti kosina. To je od posebnog značaja prilikom određivanja sanacionih mera kod klizišta.

Ključne reči: *analiza stabilnosti padina i kosina, trodimenzionalni efekti, aproksimativni postupci.*

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1. INTRODUCTION

The analyzing of slope stability is, in fact, three dimensional problem. This is a consequence of unregular shape of the sliding body and it is characteristic, expecially, for many landslides in Serbia. However, the methods of slope stability analyses are formulated in two dimensions and till now a days, there isn`t the general procedure of three dimensional stability calculations wich involve slip surfaces of arbitrary shape.

In the paper will be presented stability analysis conditions and procedures and some techniques of introduction three-dimensional effects in stability calculations, too.

2. STABILITY ANALYSIS CONDITIONS

The basis point for performing effctive slope stability analysis is to formulate the right problem and to formulate it correctly. It requires knowledge of geological characteristics of the terrain, shape of slip body and underground water conditions, too. In the view of that, it is known that landslides in Serbia are, in most cases, closely dependent of geological conditions of the terrain and on the fact that the activity of any landslide, during large period of its development restore periodically, more or less, along pre-existing slip surface. This is definitely a consequence of regional geological processes that took place during formation of the present relief and structure of the terrain. As the result of that the solis i.e. predominately overconsolidated clays and marls are quite anisotropic and heterogeneous and the shapes of sliding surfaces are quite unregular and composite, too. At present, slides take place, in most cases, along the existing, favourable oriented discontinuites, along which shear resistance has been reduced by earlier sliding to to residual value. Because of that, it is emphasized the usefulness of trial pits and shafts as exploratory works.

Selecting appropriate conditions for analyses of slopes requires consideration of conditions that will control drainage in the terrain. The basis for estimating the degree of drainage during construction or loading is the value of dimensionless time factor T which is expresed as:

$$T = \frac{c_v * t}{D^2} \tag{1}$$

in where:

- c_v -coefficient of consolidation
- t -construction or loading time
- D -length of drainage path

If the value of T exceeds 3.0 it is reasonable to treat soil as drained. If value of T is smaller than 0.01 it is reasonable to treat soil as undrained. If the value of T is between these limits, both possibilities should be considered. For natural slopes, the most severe conditions are often associated with high pore pressures and water pressures in cracks during wet periods and such problems should be analyzed as drained conditons. Thus the shear strength along slip surface of natural slopes, which are in focus of our interest, can be determined from laboratory drained tests: direct shear test, ring shear test, CD triaxial test or CU traxial test in which pore pressures and the effctive stressess at failure are measured. The tests are made on undisturbed, representative specimens which have been taken directly from sliding surface. There are, at least, two reasons for that:

-the soil properties in sliding surface, in principle, are not the same as for the soil in sliding body or under it.

-the primary strength of the soil in the slip surface may be modified by cementation or by weathering.

If the movement of slip mass has been reactivated along pre-existing slip surface, then the shear strength is residual one. This is characteristic for large landslides in Serbia: Umka, Duboko, Beška, at bridge “Sloboda”. In such cases it is often most effective to determine soil strength by back analysis. Then the landslide is treated like a direct shear test in a true scale (Fig 1.). The strength, so obtained, is the average residual one along slip surface and it is utilized to analyze repair measures.

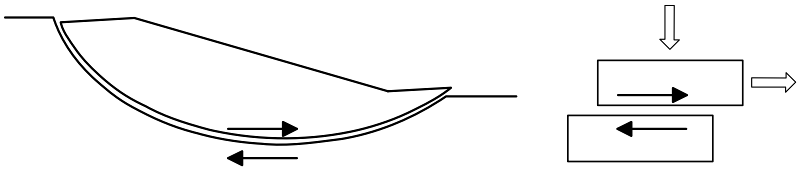

Fig. 1 Shear strength back calculation principle

The accuracy of back analysis approach will clearly depends on the accuracy by which geometry and loading of the sliding body are determined and by which the pore pressure on the surface at failure are reconstructed.

The undrained strength for saturated soils can be determined from UU triaxial test.

In stability calculations drained conditions are analyzed in terms of effective stresses and undrained conditions are analyzed in terms of total stresses.

3. STABILTY ANALYSIS PROCEDURES

The analyzing of slope stability and determining the correct safety factor are, in fact, three-dimensional problems. This is a consequence of unregular shape of the sliding body and it is characteristic, expecially, for many landslides in Serbia. However, till now a days, there aren’t the generalized procedures of three-dimensional (3D) stability calculations which involve slip surfaces of arbitrary shape. Instead of that, it is common to make two-dimensional (2D) analyses (i.e. plain strain problem) and they are carried out for the most critical cross section throughout the 3D potential sliding mass. Stability analyses should be made by limit equilibrium methods which satisfy all conditions of equilibrium. These are following methods of slices: Janbu’s GPS, Morgestern-Price;s, Spencer’s, Maksimović’s, Fredlund-Krahn’s methods. In the context of limit equilibrium approach they give a value of safety factor which differs by no more than ±6 percent from what may be considered the “corect” value (Duncan, 1996). This is certainly close enough for practical purposes, because slope geometry, warter pressures and shear strenghts can seldom be determined with accuracy as good as ±6 percent. Thus if the geotechnical engineer performs slope stability analyses using methods that satisfy all conditions of equilibrium he is justified in following:

-calculated safety factor is “correct” in terms of the mechanics of the problem and

-he can devote his attention to accurate evaluation of the properties of soils and remedial measures.

However, the question arises about the accuracy and reliability of 2D analyses to 3D problems. Research studies (Duncan, 1996) have shown clearly that factors of safety calculated using 3D analyses are larger than those calculated using 2D analyses. Implicit in this conclusion is the notion that the 2D section analyzed is the most critical section through the 3D potential sliding mass. Moreover, the error involved in appling 2D analysis to a 3D slide is not high, and in most cases is less then 10 percents (Skempton &Hutchinson, 1969).

From above it follows that 2D analyses give somewhat conservative results regarding safety factor and because of that, they provide reasonable and sufficiently accurate approach to most practical problems of slope stability. It should be born in mind, however, that regarding the shear strength calculated from back analysis they give somewhat uncorservative values i. e. they are not on the safe side.

4. APPROXIMATE PROCEDURES OF THREE-DIMENSIONAL STABILITY ANALYSIS

When lateral effects of the slope are significant then the 2D analysis is not appropriate, because 3D effects need to be taken into account. These cases are following:

- slip surfaces are bowl-shaped or lobate
- slip surfaces vary in depth from on side of the slide to another
- the width of slide in the plan is less than about twice the length in plan from head to toe

There are no rigorous method available for the analysis of a generalized 3D slip mass, but there are some procedures of introduction 3D effects in stability calculations. They will be presented.

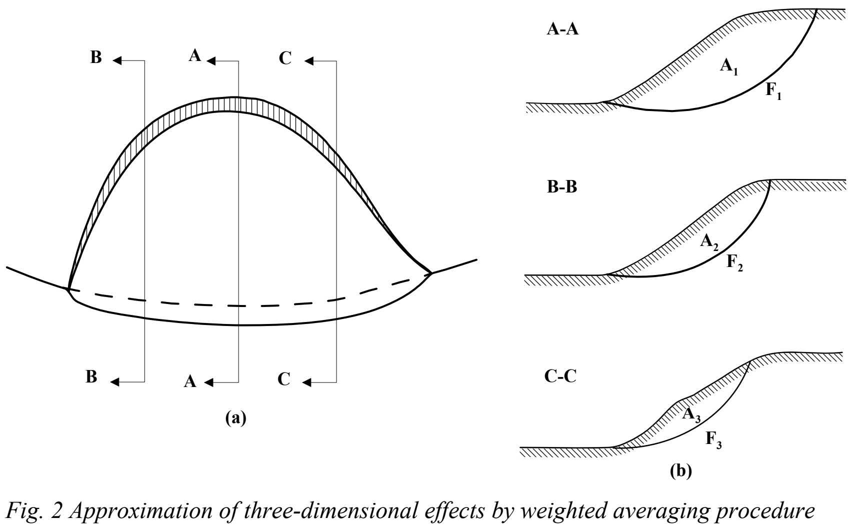

Fig. 2 Approximation of three-dimensional effects by weighted averaging procedure

4.1. Procedure of weighted averaging

Lambe and Whitman (1969) suggest to consider several parallel cross sections through the slope (Fig. 2), compute safety factor for each and then to compute a weighted safety factor

$$F_s = \frac{F_1 * A_1 + F_2 * A_2 + F_3 * A_3}{A_1 + A_2 + A_3} \tag{2}$$

4.2. Procedure of reducing shear stress

Skempton (1985) suggests that afther making 2D analysis for characteristic cross section it has to make a reduction in shear stress by the factor

$$\frac{1}{1 + K * \frac{D}{B}} \tag{3}$$

where:

D-the average depth of slope
B-the average width of slope
K-earth pressure coefficient

Skempton took K to be 0.5 and found correction of typically 5% in a number of case examples.

4.3. Procedure of approximating slip mass to regular shape

Hutchinson (1987) suggests that the slip mass can be approximated to regular shape and for such idealized, equivalent slip mass, three-dimensional factor of safety can be calculated. Following this idea actual irregular slip mass is approximated to regular sliding body (Fig. 3).

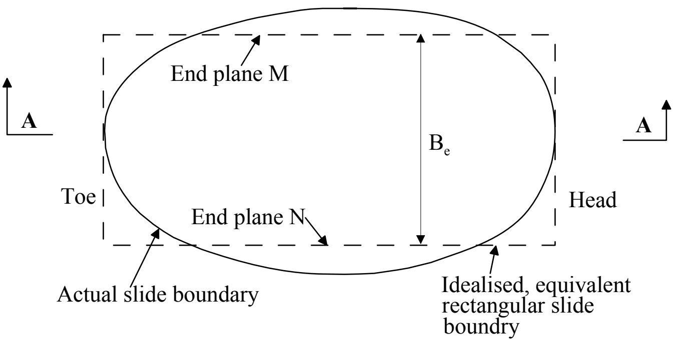

Fig. 3 Approximation of irregular slip mass to regular slide

The two-dimensional factor of safety F_2 on characteristic cross-section A-A:

$$F_2 = \frac{resisting\ moments\ or\ forces\ (\Sigma R)}{disturbing\ moments\ or\ forces\ (\Sigma D)} \tag{4}$$

Then the three-dimensional factor of safety F_3 is given approximately by

$$F_3 \approx \frac{B_e * \Sigma R + R_M + R_N}{B_e * \Sigma D} \tag{5}$$

where:

B_e –breadth of equivalent rectangular sliding body
R_M –restoring moment or force on end plane M
R_N –restoring moment or force on end plane N

This method may be applied for circular or noncircular slopes and in terms of total or effective stresses. Use of this method requires that the sliding body analyzed be approximated using engineering judgment.

In the following section it will be presented aproximation of infinite slip mass to regular body and calculation of safety factor F_3.

5. THREE-DIMENSIONAL INFINITE SLOPE STABILITY ANALYSIS

If the terrain conditions are sometimes encountered in which a layer of firm soil or rock lies parallel to the surface of the slope at shallow depth.This is, for example, characteristic for some natural slopes in settlements Medaković and Mirijevo in Belgrade.

In such conditions the the slip surface is constrained to parallel the slope. When such slip surfaces are long compared with their depth, they can be approximated accurately by infinte slope analysis. Such analysis ignore the inter-slice forces, driving force at the upper end of the slide mass and the resisting force at the lower end (Fig. 4 & Fig. 5).

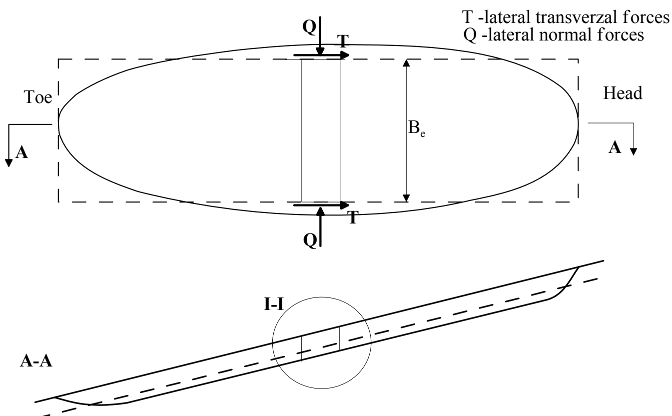

Fig. 4 Approximation of infinite slip mass to regular slide

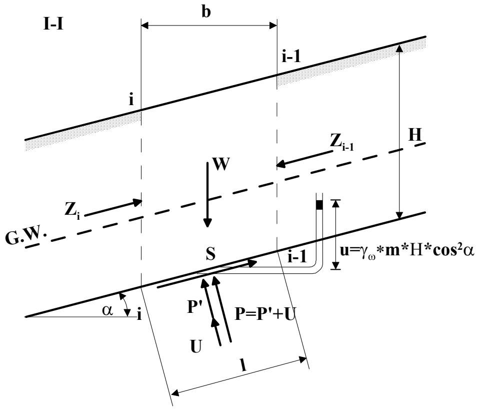

Fig. 5 Forces in two-dimensional infinite slope analysis

Two-dimensional safety factor F_2 on section A-A (Ćorić, 2001) is:

$$F_2 = \frac{c + H * \cos^2 \alpha * (\gamma_z - \gamma_w * m) * tg\varphi}{\gamma_z * H * \sin \alpha * \cos \alpha} \tag{6}$$

c, φ	-shear strength parameters for slip surface
α	-slope angle
γ_z	-saturated unit weight of sliding mass
γ_w	-unit weight of water
H	-vertical height from slope surface to slip surface
m*H	-vertical height from slip surface to ground water table ($0 \le m \le 1$)

Three-dimensional safety factor F_3, in which are included the effects of lateral normal and transverzal forces is:

$$F_3 = \frac{1}{\sin \alpha} * \left\{ \left(\frac{1}{H * \cos \alpha} + \frac{2}{B_e} \right) * \frac{c}{\gamma_z} + \right.$$

$$\left. + \left[\cos \alpha * \left(1 - m * \frac{\gamma_w}{\gamma_z} \right) + \frac{K * H}{B_e} * \left(1 - m^2 * \frac{\gamma_w}{\gamma_z} * \cos^2 \alpha \right) \right] * tg\varphi \right\} \tag{7}$$

where:

K -earth pressure coefficient; typical values are: 0.5-0.7

For effective stress analysis in equations (6) and (7) are used effective shear strength parameters i.e.

$$c = c'; \qquad \varphi = \varphi' \tag{8}$$

For total stress analysis in equatons (6) and (7) are used undrained shear strength parameters i.e.

$$c = c_u; \qquad \varphi = \varphi_u = 0 \tag{9}$$

6. CONCLUSIONS

From this paper the following conclusions can be derived:

-There are no rigorous method for the analysis of a generalized 3D slip mass but, for most practical problems, 2D stability analyses provide resonable and sufficiently accurate approach of slope stability.

-If the 3D effects of the slope are significant, than they should be involved in the stability analysis and in the paper are presented

a) approximative procedures of 3D analyses

b) 3D stability analysis for infinitive 3D slip mass

-Safety factor calculated using 3D analyses is always higher than the safety factor calculated using 2D analyses i.e. $F_3>F_2$

-If the 3D effects are neglected in analyses to back calculate shear strengths, than the back calculated strengths will be to high.

On the basis of that it can concluded that 3D analyses are expecially important in evaluting remedial measures for landslides along pre-existing slip surfaces i.e. in all cases in which shear strengths are calculated from the back analyses.

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TENTH NATIONAL AND
FOURTH INTERNATIONAL
SCIENTIFIC MEETING

iNDiS 2006

NOVI SAD, NOVEMBER 22-24,2006.

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NUMERICAL AND EXPERIMENTAL DETERMINATION OF RESIDUAL CONCRETE STRENGTH AFTER ACTION OF FIRE

Summary: *In February 2005 a building structure was in fire and two apartments were completely burned. Temperature over 400^{0}C causes irreversible reduction of the compressive strength and other mechanical properties of concrete. According to this statement and for realization of the repair project of the RC structure, nonlinear and transient thermal analysis, nonlinear stress-strain analysis of the frame structures and experimental determination of the residual concrete strength after action of fire were recommended. Results obtained by these analysis are presented in this paper. The residual concrete strength is determinated nubericaly and the results are compared with the results obtained by laboratory tests of specimens taken from the RC walls and RC slabs. These results show that during the fire action, as well as in the cooling period, the strength and stiffness of structural elements were continually reduced and adequate repair of the damaged elements has to be made.*
Key words: *Fire, temperature, thermal analysis, structural analysis, residual concrete strength*

NUMERIČKO I EKSPERIMENTALNO ODREĐIVANJE REZIDUALNE ČVRSTOĆE BETONA PRI PRITISK NAKON DEJSTVA POŽARA

Rezime: *Februara 2005 godine konstrukcija zgrade je bila zahvaćena požarom i dva stana su kompletno izgorela. Temperatura veća od 400^{o}C prouzrokovala je nepovratnu redukciju čvrstoće na pritisak i drugih mehaničkih karakteristika betona. Iz tih razloga, kao i za potrebe sanacije AB konstrukcije predložena je nelinearna i nestacionarna termička i nelinearna statička analiza ramovske konstrukcije, kao i eksperimentalno određivanje rezidualne čvrstoće betona pri pritisku nakon dejstva požara. U radu su prezentirani numerički rezultati dobijeni ovom analizom i isti su upoređeni sa rezultatima dobijenih laboratorijskim testiranjem primeraka uzetih iz AB zidova i AB ploča. Na bazi dobijenih rezultata utvrđeno je da su za vreme požara, kao i u periodu hlađenja, nosivosti i krutosti konstruktivnih elemenata značajno degradirane, usled čega je neophodno njihovo saniranje.*
Ključne reči: *Požar, temperatura, termička analiza, konstruktivna analiza, rezidualna čvrstoća betona*

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1. INTRODUCTION

In February 2005 a twenty storey building structure was in fire and two apartments at the seventh and eighth floor were completely burned. Primary, the fire was caused by gas explosion, but the synthetic materials in the apartment were additional fire load, so very high temperatures were reached and the fire time was more than four hours. According to the damages recorded in situ, it was found out that fire caused severe damage to the reinforced concrete bearing structure, while the interior and the installations were completely destroyed. Based on the data collected through detailed visual survey of the bearing structural elements, the following characteristic damages were recorded: change of the concrete color (red, grey-yellow, yellow, fig.1); fissures and cracks inside the concrete mass; cracks along main reinforcement in columns, beams and slabs; crushing of concrete and falling off of the concrete parts along the edges of linear elements up to the reinforcement.

Temperature over 400^0C causes reduction of the compressive strength and other mechanical properties of concrete and this process is irreversible (the strength of concrete does not recover in the cooling phase). The mechanical properties of steel decrease as well, but in the cooling phase they increase again. According to these statements and for realization of the repair project of the RC structure of the 7^{th} and 8^{th} floor the following steps were recommended: nonlinear and transient thermal analysis; nonlinear stress-strain analysis of the frame structures [1,2] and experimental determination of the residual concrete strength after action of fire [4]. Results obtained by this analysis show that during the fire action, as well as in the cooling period, the strength and stiffness of the structural elements were continually reduced and adequate repair of the damaged elements had to be made [3].

Figure 1. Change of concrete color of RC elements

2. EXPERIMENTAL DETERMINATION OF THE RESIDUAL CONCRETE STRENGTH

The experimental testing of the residual concrete strength after the fire action was done by the Institute for Materials Testing and Development of New Technologies "Skopje"-Skopje. According to the previous Schmidt hammer testing results, the locations of the eight concrete specimens, taken only from RC walls and RC slabs, were defined. Hammer testing of concrete elements from apartments that were not fired confirmed that the compressive strength of the concrete before the action of fire was f_c=40MPa (MB30).

Table 1. Concrete strength testing results

No.	Position	Dimensions of kerns (cm) total H	h (testing)	D	density (kg/m^3)	Test load (KN)	Concrete compressive strength (MPa) cylinder	reduced to cube 20/20/20	Age of Concrete (years)	Concrete strength (MB) according to MKS U.M1.048
1	RC wall (B8)	15	9.9	9.9	2214	200	**26.0**	26.5	22	**20.0**
	(after fire expose)		3.5		2150	390	**15.8***	16.1		**12.0**
2	RC slab (T2)	10 +7cm plaster	10	9.9	2300	215	**28.0**	28.6	22	**21.4**
	(after fire expose)		/		/	/	**/**	/		**/**
3	RC slab (MS)	18	10.3	9.9	2340	260	**33.8**	34.5	22	**26.0**
	(after fire expose)		6.5		2310	145	**12.5***	12.8		**9.5**
4	RC slab (SII)	18	9.6	9.9	2315	295	**38.4**	39.1	22	**29.4**
	(after fire expose)		4		2269	160	**13.0***	13.3		**10.0**
5	RC wall (V7)	25	9.9	9.9	2117	272	**35.4**	36.1	22	**27.0**
	(after fire expose)		5		2138	265	**14.5***	14.8		**11.0**
6	RC wall (G7)	19.4	10	9.9	2179	195	**25.4**	25.9	22	**20.0**
	(after fire expose)		3.5		2168	290	**14.5***	14.5		**11.0**
7	RC wall (D7)	18	10	9.9	2212	236	**30.7**	31.3	22	**24.0**
	(after fire expose)		3.5		2168	290	**14.5***	14.8		**11.0**
8	RC wall (E7)	16	10	9.9	2259	246	**31.9**	32.6	22	**25.0**
	(after fire expose)		3		2225	380	**14.1**	14.4		**10.5**

* Values are reduced with coefficients depending on the shape and height (h) of the deteriorated concrete specimens

The concrete specimens taken from the RC walls (B8, V7, G7, D7, E7) were exposed to fire from one side. The corresponding surface layers (3-5cm thick) had changed the color (red, grey-yellow, yellow) and were more deteriorated than the inner layers (fig.2). The RC slabs over the 7^{th} floor were fire exposed from both sides, but they were covered with 1cm thermal isolation and 4cm lean concrete (fig.1b), that directly influenced upon the cross section temperature field, therefore the specimens taken from the slabs (T2, MC, CII) were deteriorated from one side too.

Before testing all the specimens were divided in two slices. The deteriorated (burned) slices had small height (3-6cm) and rough surface therefore they were specially prepared by adding plaster layers (fig.3). In that case the measured values for the compressive concrete strength were reduced with coefficients depending on the shape and height (h) of the deteriorated concrete specimens [4]. Test results are presented in Table 1.

Figure 2. Change of concrete color and deterioration of surface layers of specimens

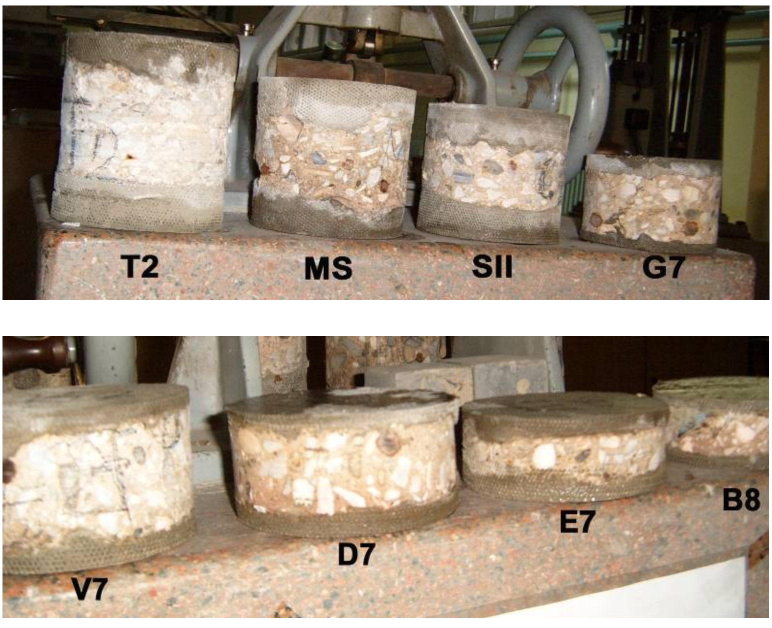

Figure 3. Deteriorated concrete specimens, prepared for testing

3. NUMERICAL DETERMINATION OF THE RESIDUAL CONCRETE STRENGTH

Thermal and structural response of four-bay, five-story reinforced concrete frame (only one part of the whole frame with defined support conditions) exposed to fire scenario at the two floors only has been investigated analytically, too (fig.4). Elements geometry; support conditions; concrete cover thickness; type of aggregate; compression strength of concrete; steel ratio and defined fire scenario were taken into account while the nonlinear and transient temperature field and the concrete strength reduction in the cross section of the elements exposed to fire were determined.

The computer program FIRE [1] was used to solve this problem and the following assumptions were made:

- Fire was modeled by a single valued gas temperature history and in this case ISO 834 fire model was used. According to data gathered through detailed visual survey of the burned structural elements and the change of concrete color it was assumed that the maximum fire temperature of 1000^{o}C was reached at the moment t=1.2hour and after that the cooling period started.
- Temperature dependent material properties were known (recommended in EC2)
- Two dimensional heat transfer was assumed.
- The fire boundary conditions were modelled in terms of both convective and radiating heat transfer mechanisms. For the surfaces directly exposed to fire the coefficient of convection was assumed h_c=25 W/m^2 ^{0}C and for the unexposed surfaces h_c=9 W/m^2 ^{0}C, as it is recommended in Eurocode 2, part 1.2.
- No contact resistance to heat transmission at the interface between the reinforcing steel and concrete occurred.
- The easy heat penetration, after cracks had appeared, or some parts of the cross section had crushed, was neglected.

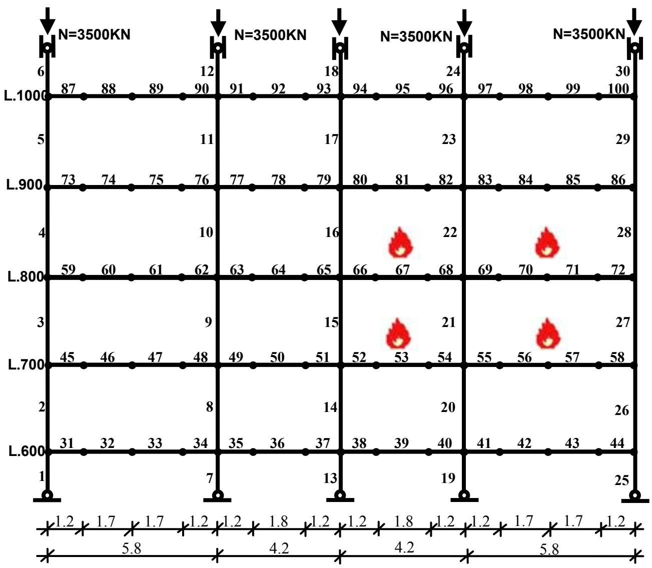

Figure 4. Schematic presentation of the fired frame (elements discretization)

The most damaged column was incorporated into the wall, so it was exposed to fire only from the inside of the compartment, but the temperature on the other side (in the hall) was raising proportionally to the temperature in the fire compartment (the fire flames were coming out through the open door) and the heating was from the both sides, but not with the same intensity. The dimensions of the cross section of this column (fig.5) are 60×60cm, the compressive strength of the concrete before action of fire was f_c=40MPa (MB30). It is symmetrically reinforced with 18ϕ16. The yield strength of the reinforcing bars is $f_y(20^oC)$=240Mpa. Before the action of fire the column was loaded by axial force N=3000KN and the compressive stress was 8 Mpa (20% of f_c).

The concrete strength was reduced as a result of high temperatures in the surface layers of the cross section (fig.5a). Calculation results indicate that on the side of the fire, in 4-5cm thick layer, which is 17% of the cross section of the column, the strength reduction is significant and the residual strength of concrete is 15Mpa (MB11.5) in average (fig.5b). These results correspond well with experimental results obtained by laboratory testing of specimens taken from the nearest RC wall (B8, Table 1). In the cross section core the strength of concrete is not reduced.

Columns S2 and S3 were exposed to fire almost from all sides, so it caused almost symmetrical cross section temperature field. In the cross section surface layers, as a result of high temperatures, the concrete strength was reduced. Calculation results indicate that in 4-5cm thick layer, which is 30% of the cross section of the column, the strength reduction is significant and the residual strength of concrete is 6Mpa (MB12, fig.7b) in average.

Results obtained by the thermal and static analysis of the frame structure show that fire had the most negative influence upon the beam elements from level 800 (over the 7th floor). These results correspond well with experimental results obtained by laboratory testing of specimens taken from the elements from that floor and with visually recorded changes in concrete color (V7, G7, D7, E7, Table 1). The fire intensity on the 8th floor was less then that of the 7th floor and the degree of damage was less too (for B8 the reduction of concrete strength is less then for the other specimens). According to the defined degree of damage, beams and columns from the 7th and 8th floor have to be adequately repaired.

Beams from level 800 (over the 7th floor) were fire exposed from both sides, but from the upper side, as a part of the RC slabs, they were covered with 1cm thermal isolation and 4cm lean concrete (fig.1a), that directly influenced upon the cross section temperature field (fig.6a) and the concrete strength reduction (fig.6b). Beams from level 900 (over the 8th floor) were fire exposed only from the bottom side.

Stress redistribution was caused due to the high temperature difference between the surface layers and inner layers. During the cooling phase, when the temperature in the cross section layers decreased, the negative elongation was not proportional to positive elongation when temperature increased, so it caused longitudinal cracks along the main reinforcement. These cracks were visually noticed on the surface of all columns and beams (fig.1a), but the depth was not defined.

During the fire period large deformations occurred in beam elements from level 700 (under the fire compartment) although they were not heated because of the thermal isolation and concrete cover over the RC slabs. During the cooling period all cracks were closed and there were no additional residual stresses.

The results obtained by the thermal analysis of the RC slabs type OMNIA, the detailed visual survey in-situ and the experimental results obtained by laboratory testing of specimens (MS, SII, Table 1) lead to a conclusion that the concrete strength is reduced only in the bottom layers where concrete is in tension and has no influence on the bearing capacity of the slabs. In the upper 10cm thick compressed layers the concrete strength is not significantly reduced, therefore the recommended repair will be only surface finishing of the bottom of the slabs.

The RC walls were fire exposed only from the inside of the compartment, the temperature didn't penetrate deep in the cross sections (fig.7a) and the strength reduction was significant only in the concrete cover layers (3.0-3.5cm), therefore the repair of the RC walls is not recommended. These results correspond well with experimental results obtained by laboratory testing of specimens B8, G7, D7, E7 (Table 1) taken from the RC walls from the 7^{th} and 8^{th} flour. It is not the case only for the specimen V7 (fig.2a) taken from the RC wall in the room where the fire started and the temperature was highest.

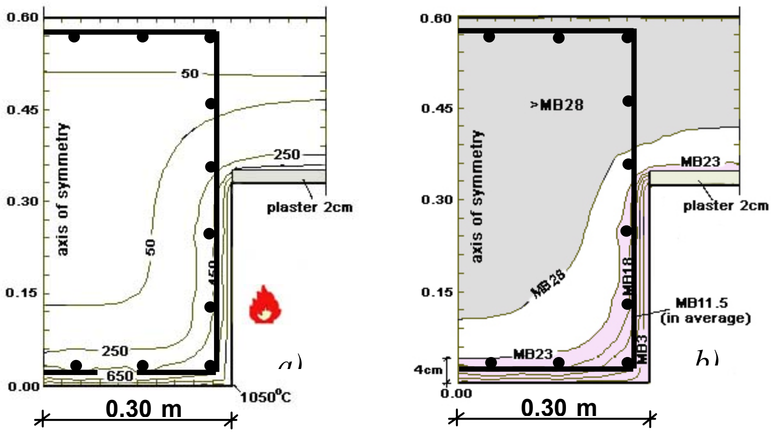

Figure 5. a) Temperature distribution; b) Residual concrete strength in cross section of most damaged column, exposed to fire only from two sides

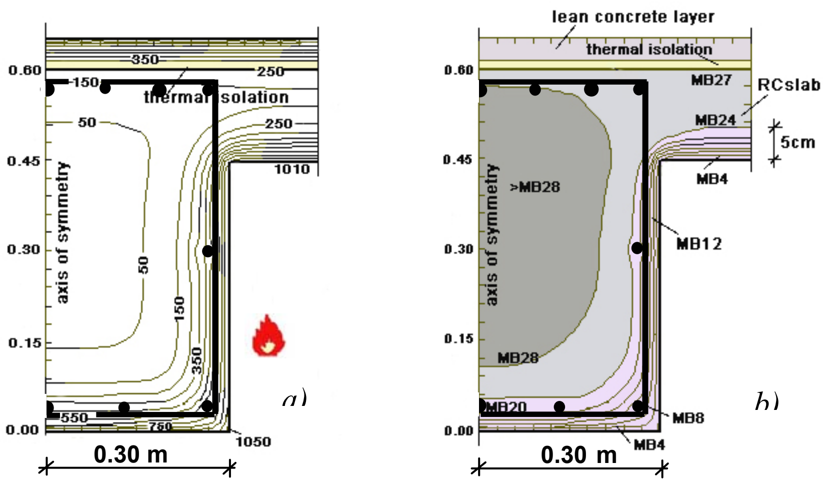

Figure 6. a) Temperature distribution; b) Residual concrete strength in cross section of beam and corresponding RC slab

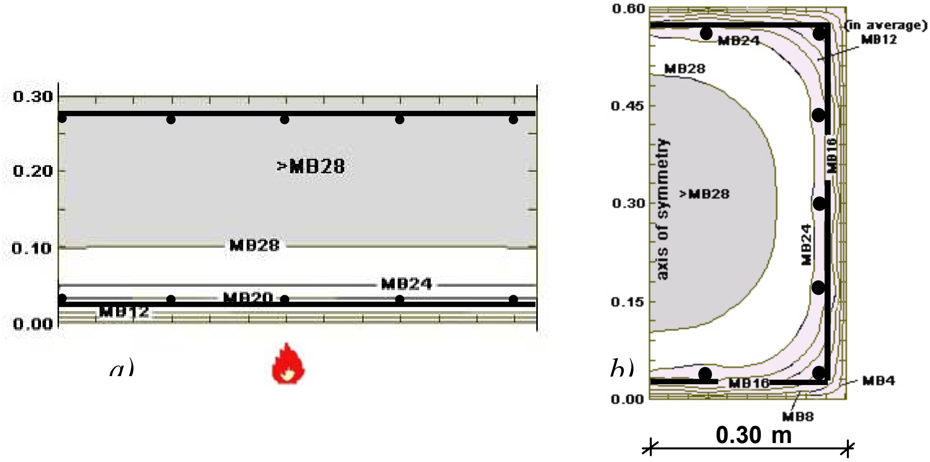

Figure 7. Residual concrete strength in cross section of
a) RC wall exposed to fire from inside; b) RC column exposed to fire from all sides

Results obtained by the thermal and static analysis and experimental testing of specimens show that during the fire action, as well as in the cooling period, the strength and stiffness of structural elements were continually reduced and adequate repair of the damaged elements has to be made.

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TENTH NATIONAL AND
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iNDiS 2006

NOVI SAD, NOVEMBER 22-24,2006.

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NUMERICAL ANALYSIS ON COMPOSITE CONNECTION BRIDGES WITH SMALL AND MEDIUM SPANS

Summary: *In the last years the composite structural solutions for bridges are widely used. This tendency need more attention in to the design of structural elements for the composite bridges. In order to promote a simplified method to evaluate the behaviour of the connection between two composite beams, the authors performed some numerical analysis. The first one was to calibrate the connection between the beam and slab. After that with the connection calibrated further numerical analysis were conducted. The results offer informations about the stress distribution in all the components of connection i.e. concrete, reinforcements and steel. The paper presents a numerical analysis for a composite connection with end plates and connectors. The analysis were performed on composite steel-concrete beams within the frame of national research project MIKTI developed at INSA Rennes – France.*

Key words: *Structures, Composite elements, Composite Joints, Finite Elements Analysis, Stress state*

NUMERIČKA ANALIZA SPREGNUTIH MOSTOVA MALIH I SREDNJIH RASPONA

Rezime: *Poslednjih godina se veoma često primenjuju rešenja sprezanja konstrukcija. Ova tendencija zahteva znatno više pažnje pri projektovanju nosećih elemenata za spregnute mostove. Autori su sproveli određene numeričke analize da bi unapredili pojednostavljeni metod za procenu ponašanja veze između dve spregnute grede. Prvo je kalibreisana veza između grede i ploče, a nakon toga je sprovedena i numerička analiza. Rezultati daju informacije u svim komponentama, tj. betonu, armanuri i čeliku. U rad je prikazana numerička analiza spregnute veze sa pločama i konektorima. Analiza je sprovedena na spregnutoj gredi u okviru nacionalnog istraživačkog projekta MIKTI razvijenog na INSA Rennes – Francuska.*

Ključne reči: *Konstrukcije, spregnuti elementi i čvorovi, metod konačnih elemenata, stanje napona.*

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1. INTRODUCTION

Based on a large bibliographic study regarding the dimensioning of the connections using simplified models, as the one presented in the paper “Model for dimensioning a connection between two composite girder beams with end plates and connectors” [1], it was determined that the information about the constructional details is missing or is very poor in the literature.

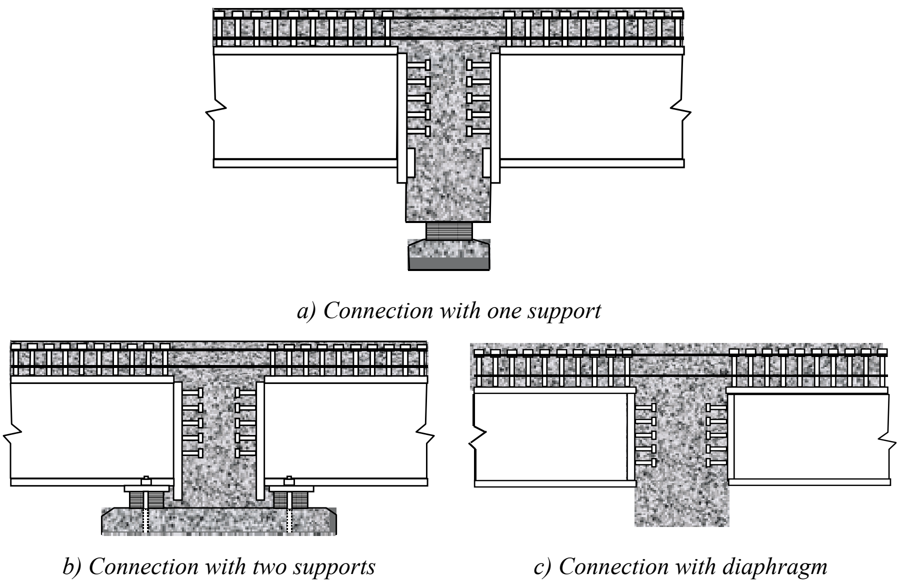
In order to overcome these problems in the standards, a series of analysis was made using the finite element program “BIOGRAF”; it allows the study of the plane state of stresses, in the nonlinear domain.

The nonlinear behaviour of reinforced concrete is due to the nonlinear properties of the concrete, of the reinforcement material and also due to the interaction between these two elements.

The overall behaviour of a composite steel-concrete element can be defined if only one will integrate in the analysis the specific behaviour for concrete, for steel and also the connection materials between the two materials (connectors, adherence).

2. CONNECTION BETWEEN COMPOSITE BEAMS WITH END PLATES AND CONNECTORS

Starting with the 1960, in Australia, due to technical and economical reasons, there was begun a research program by the State Highway Department, having the aim of establishing a new standard for dimensioning the girder beams for composite bridges.



a) Connection with one support

b) Connection with two supports *c) Connection with diaphragm*

Figure 1. Types of connections using end plates and connectors.

R.A. Kell, A. Fried and J.K. Lloyd [4] present different stages that marked the evolution of connecting techniques used in Australia, based on using special reinforced concrete elements to ensure the continuity of the bridge girder beam's.

In the case of connection using end plates and connectors, in the first phase the steel girder beams are placed, having the behaviour of a simple supported beam; in the second phase, the structure becomes a composite one by casting the reinforced concrete transversal beam and the bridge deck. In figure 1, there are presented some types of connections using end plates and connectors.

The role of the end plates is to distribute into the concrete the compressive stresses in the bottom flange of the steel beam. The upper flange connectors have the role of transmitting the tension stresses between the bridge deck and reinforcement, while the end plate connectors' aim is to transfer the shear stresses to the transversal girder.

3. FINITE ELEMENTS ANALYSIS OF CONNECTION BETWEEN TWO BEAMS WITH END PLATES CONNECTORS

3.1. Calibration Of The Beam – Slab Connection

In order to have a numerical model of the connection between two beams with end plates and connectors the first problem was to calibrate the behaviour of connection between the beam, connectors and concrete slab.

According to Chapter 10 of EC4 , it was established a numerical model applied to a "push-up" specimen, considering the same dimensions and the reinforcing percentages for the concrete slab as the ones used in the further analysis.

The aim of this study was to determine the characteristic curve for one connector.

A similar behaviour for connectors was achieved using the model in which the connectors were replaced by an equivalent layer located at the contact between the metallic beam and concrete.

This layer was calibrated after several steps (elasticity modulus E, the reinforcing percentage p) so that in the end it was obtained a similar behaviour as for the experimental model.

The closest result to the real situation was obtained using for the equivalent layer a small elasticity modulus and 45^o inclined reinforcing percentages, the layer being similar to a truss girder in this case.

The recorded displacements were located at the upper level, respectively bottom level of the equivalent layer. In the figure 2 are presented the studied model and the load-displacement curve compared to the one obtained for the "push out" experimental model using Nelson connectors.

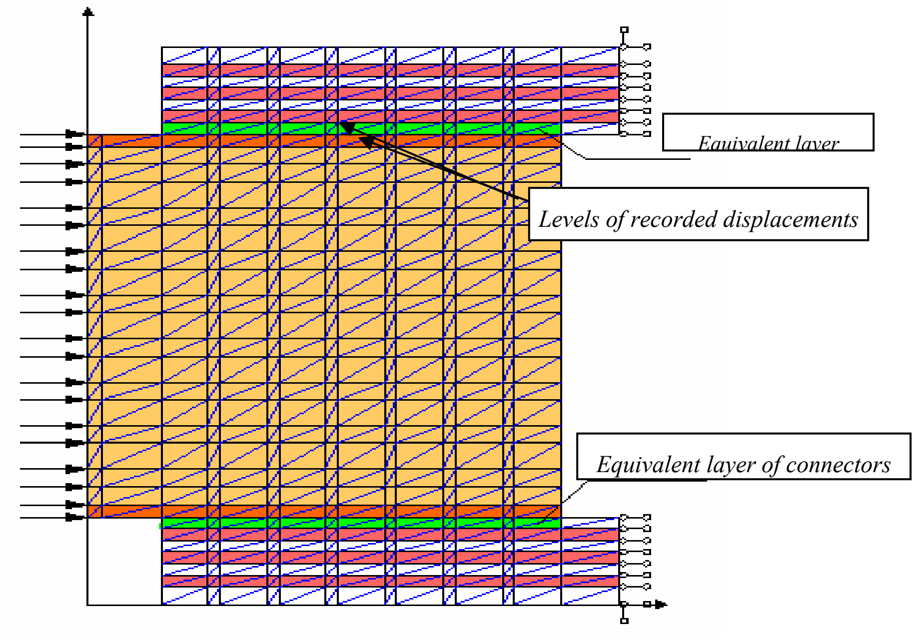


Figure 2. Finite element mesh and restrainsts conditions for the push out experimental model using equivalent layers

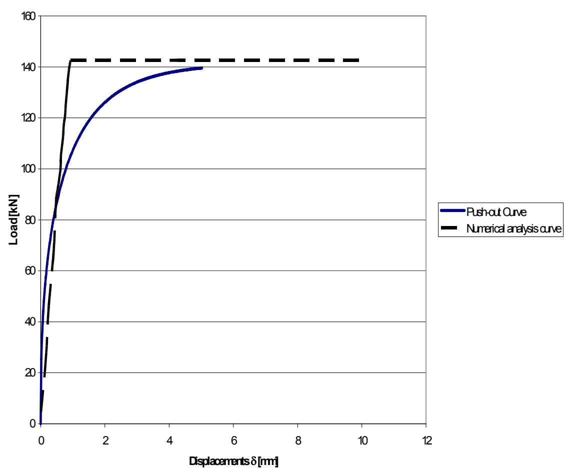


Figure 3. Load – displacement curve for one connector

3.2. Finite element model of connection between two composite beams with end plates and connectors

Having the results presented in the 2nd paragraph, we proceeded to the analysis of the state of stresses in a connection using end plates and connectors.

For the finite element analysis of this connection it was used the BIOGRAF program – developed by a study group in our department. The simplified schema is presented in figure 3.

This program allows for the analysis of the concrete elements and the composite elements (steel and concrete) under a plane state of stresses. The discretization of the structure is done using triangular elements in a plane state of stresses, the loading being applied using concentrated forces in the nodes of the net.

The nonlinear analysis is done using a biographical study based on loading increments given by the user. The elements can have different thicknesses, having only to stay inside the plane state of stresses hypothesis.

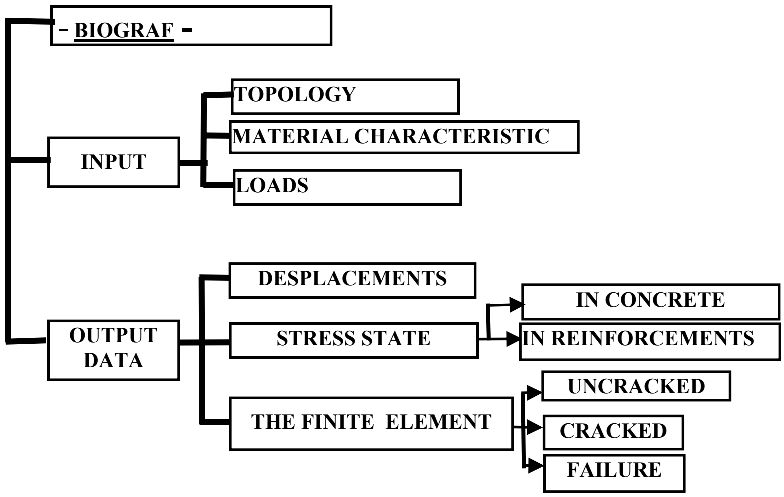


Figure 4. The simplified schema of BIOGRAF program

The cracking model for concrete can rely on different classical criterions for the yielding and breaking of an isotropic material. These criterions take into account the tension and compression strengths for concrete. The BIOGRAF program uses the Cervenka model, which combines the tension-tension Navier criterion and the compression-compression Von Mises criterion.

The studied model was the one having stirrups inside the connection and with equivalent layers both at the surface between girder beam and bridge deck and also at the surface between the end plate and the connection itself. The discretization of the element is presented in the figure 4. There were chosen 4 characteristic cross-sections to represent the σ_x unit stresses; the aim was to evaluate the state of stresses and the stress transfer between the steel beam, concrete and the elastic reinforcement.

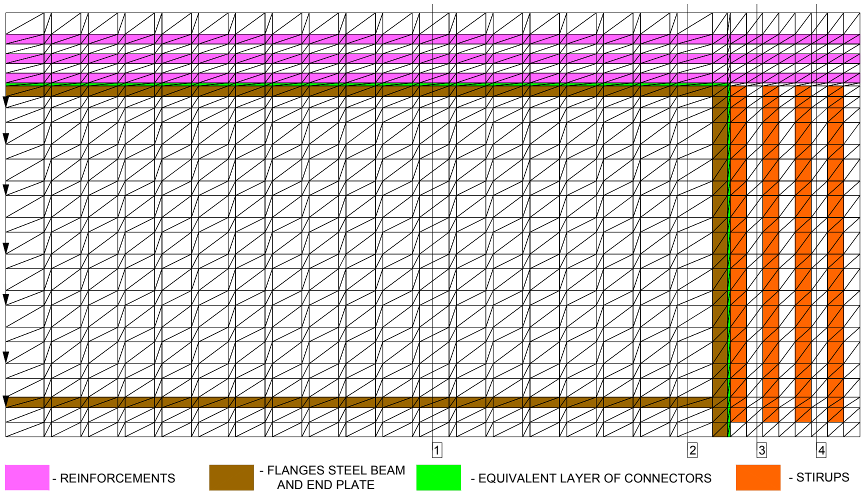


Figure 4 – Finite element mesh of connection with end plate and connectors

The cracking state in the connection and the failure mechanism for aplied load $V_s^{(u)} = 1658\text{kN}$ is presented in the figure 6.

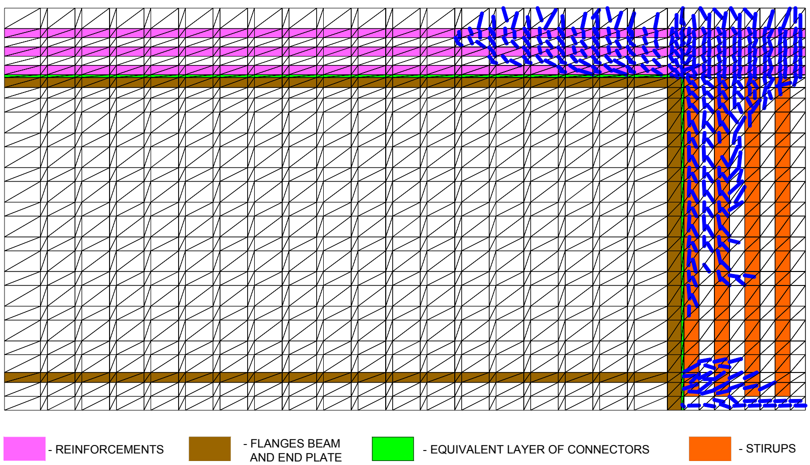


Figure 5 – The cracking state in the connection zone - failure mechanism

In the Figure 6 it is presented the evolution of the σ_x unit stresses in the characteristical sections mentioned in figure 4.

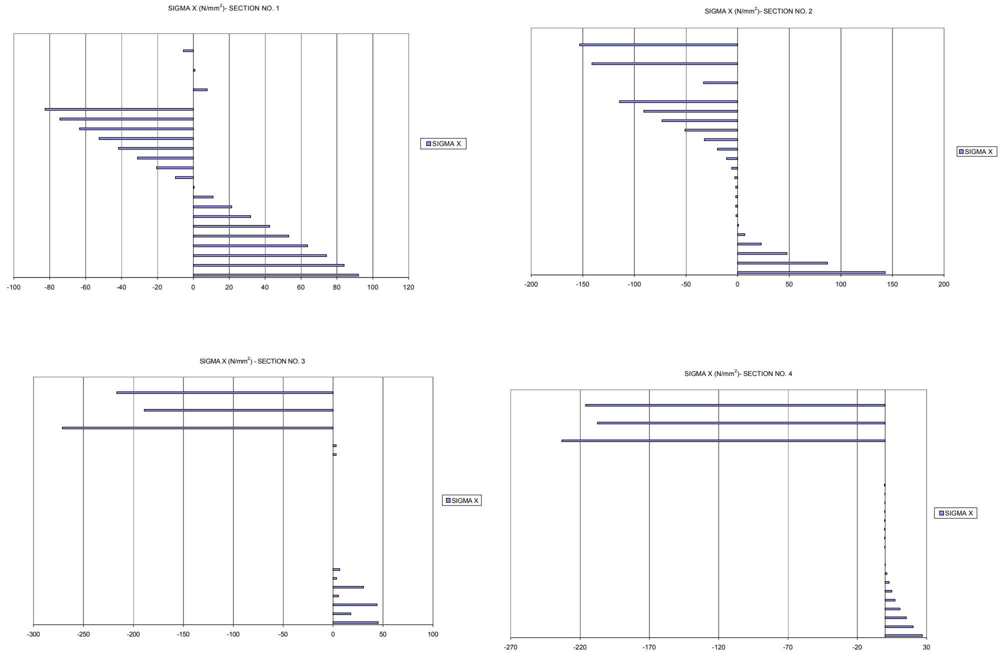


Figure 6 – Stress distribution σ_x at failure

In the Figure 7 it is presented the evolution of the σ_x unit stresses for all the three reinforcement layers at the moment of failure, on the entire length of the beam.

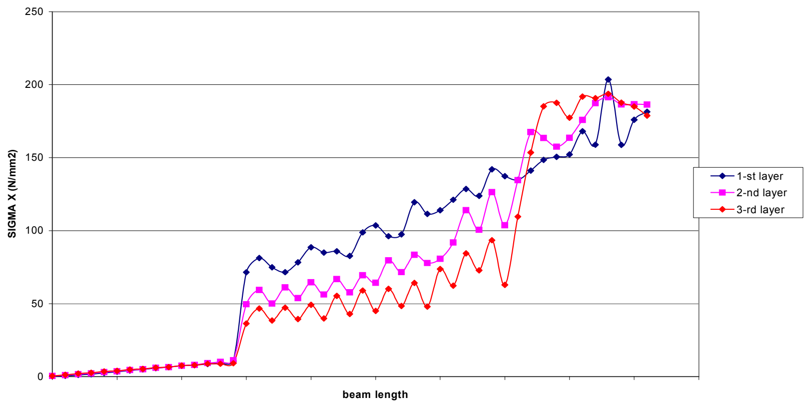


Figure 7 – Evolution of the σ_x unit stresses for all the three reinforcement layers at the moment of failure

4. CONCLUSION

Analysing the results, we can draw the following conclusions:

- the unit stresses in the first two layers of reinforcement (upper part and middle of the slab) reach 220N/mm^2, which lead to the conclusion that thy still have reserves for the bearing capacity; the unit stresses in the 3rd layer of reinforcement (at the bottom of the slab) reach 425N/mm^2, which represents the limit of the bearing capacity.
- the value of the maximum compression stresses σ_x in the concrete in connection reach 28N/mm^2 (the compression strength for concrete is 30N/mm^2)
- the unit stresses in the first two layers of reinforcement increase progressively as closest we get to the connection, having a small increase at the interface with the cracked zone in the deck, perhaps due to a faster stress transfer from the cracked concrete to the reinforcement.
- the failure is due to reaching the limit compression strength in the concrete and the limit tension strength (yielding) in the 3rd layer of reinforcement.

The study of the composite connections should not be limited only to the design aspects; there are necessary studied which regard: technology of execution; costs for production and casting in place. All these are different aspects which can lead to a more efficient connection solution.

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Jasmina Dražić[1]

STRUCTURAL SYSTEMS IN THE PHASE OF CONCEPTUAL DESIGNING OF ASEISMIC STRUCTURES

Summary: *Structural system should provide optimal transmission of all loads to the foundation, as well as to satisfy functional requirements. When designing aseismic structures, preference is directed towards satisfactory structure configurations, i.e. regular structures, since well conceptualized and qualitatively built structures have satisfactory behaviour under earthquakes. Interdependency analysis of functional and structural building characteristics for various purposes (administrative, residential, medical, educational, industrial, etc), as well as the influence of structural solution onto building configuration, enables the possibility of proposing suitable structural systems, which would be in accordance with functional requirements and provide the necessary flexibility level, simultaneously with providing necessary structure reliability for seismic action. Computer application ArhPrep, in accordance with the recommendation of EC8, opens the possibility of selecting appropriate structural system in the phase of conceptual designing of a structure.*

Key words: *conceptual designing, structural system, aseismic designing, building configuration, regular structures*

KONSTRUKCIJSKI SISTEMI U FAZI KONCEPTUALNOG PROJEKTOVANJA ASEIZMIČKE KONSTRUKCIJE

*Rezime**:** Konstrukcijski sistem treba da obezbedi optimalan prenos svih opterećenja na podlogu, ali i da zadovolji funkcionalne zahteve. Pri projektovanju aseizmičkih konstrukcija prednost se daje povoljnim konfiguracijama zgrada tj. regularnim konstrukcijama, jer se dobro koncipirane i kvalitetno izvedene zgrade povoljno ponašaju pri zemljotresnim udarima. Analiza međuzavisnosti funkcionalnih i konstrukcijskih karakteristika zgrada za više različitih namena (administrativne, stambene, zdravstvene, školske, industrijske i dr.) i uticaj funkcionalnog rešenja na konfiguraciju zgrade, omogućio je da se predlože pogodni konstrukcijski sistemi, koji će u skladu sa funkcionalnim zahtevima obezbediti potreban stepen fleksibilnosti, ali istovremeno i neophodnu pouzdanost konstrukcije za seizmička dejstva. Računarska aplikacija ArhPrep, u skladu sa preporukama EC8, otvara mogućnost izbora pogodnog konstrukcijskog sistema u fazi konceptualnog projektovanja konstrukcije.*

Ključne reči: *konceptualno projektovanje, konstrukcijski sistem, aseizmičko projektovanje, konfiguracija zgrade, regularne konstrukcije*

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1. INTRODUCTION

During the process of exploitation, depending on their function, structures can be under the influence of various actions. Depending on the time regime of their appearance, the actions can be permanent or variable. Building location can also condition dynamic actions whose intensity increases over time or has a stochastic character.

Actions cause deformations in the bearing system, and dynamic forces arouse the structure to vibrations [10]; thus, the structural system needs having adequate bearing capacity (resistance), i.e. safety abilities against various loads and actions. First of all, structural system should provide optimal transmission of all loads to foundation, with the necessary reliability factor, stability requirements, usability, durability and maintenance adequacy, as well as satisfying all functional requirements. Inadequate model or type of a structural system can be opposite to stability and resistance requirements, chosen material or building function. Structural system is required to be realized easily and fast, and to be aesthetically acceptable. Certain functional solutions allow building structure to be solved by element connections, that appear continuously with same spans (halls), in order to achieve rhythm and enable significant economic effect (calculating, investable, as well as aesthetic). Therefore, structural solution should satisfy several criteria simultaneously, to be stable, safe, logical, simple, and proportional, with thin lines, i.e. satisfactory visual effects.

For the structure of a facility there are situations when, considering the variety of factors influencing on it (of objective and subjective nature, and their mutual influence), there are more different, equally satisfactory solutions which can be adopted for the facility. Designing a structural system therefore presents a complex research process where special meaning is given to designing methodology and philosophy [8]. The process of designing structural systems is based on studying the probability to be used to determine system overload after certain actions. Besides having the knowledge on static and dynamic system calculations, structure designer needs possessing elementary knowledge to be able to design the structural system correctly in the first working phase, i.e. preliminary structure designing. Proper coordination between functional and structural building characteristics and the selection of adequate configurations in the phase of conceptual designing of aseismic structures leads to good economic results in the phase of building realization.

2. RESISTANCE OF STRUCTURAL SYSTEMS FROM SEISMIC ACTIONS

When designing and building aseismic structures, it is hard to give absolute advantage to one structural system. Adequately selected structure configuration [2], [3], well chosen material and carefully processed details can make different structural systems more resistant to the action of seismic forces [11].

A) Skeleton structures are considered to be light structures due to small weight and small weight of non-bearing partitions. Therefore, they can be built on weaker soil; also, small weight induces small inertial forces during earthquake actions. Skeleton structures are flexible and movable horizontally which enlarges their period, and long-period buildings are less affected during close and strong earthquakes, unless their foundation is on weak soil with extremely long predominant period. In this aspect, high flexibility is also the drawback of these systems. Skeleton systems with more height, due to large horizontal movement, can become sensitive on second order influences and they are susceptible to great damages (partition walls and other non-bearing elements). Damages in wall fillings appear due to right angle change between framework lintels and posts, causing shearing efforts that cannot be endured by partition walls.

a) Unstiffened skeletons sense the smallest seismic forces, though their bearing capacity is relatively small if the posts with large cross sections are not designed. Therefore, during strong earthquakes, relatively great damages should be expected with these structures. Skeleton structures without additional stiffeners (vertical diaphragms or cores) are considered to be the purest concepts in statistic sense. They provide high degree of functional flexibility, though they appear rarely, since mostly staircase walls and lift pits are utilized as stiffening elements. The exceptions are cores placed next to the facility, which is statistically inadequate solution.

b) Skeletons stiffened by stiffening walls form less flexible structures. These systems present optimal and most often applied solution with high buildings. The concept of the structural system presents frames as transmitters of gravity actions, while seismic and other horizontal forces are transmitted by special elements – stiffeners. The mass of these buildings is not much larger than the mass of unstiffened skeletons. Due to greater stiffness, vibration periods are shorter, so these systems induce stronger inertial forces. Still, they can be accepted without difficulties because the bearing capacity of these systems is higher. Stiffness reduces the flexibility of these systems, decreases inadequate effects of the second order theory and decreases damages to non-bearing elements. Since the structure presents a smaller portion of the building value, damaging non-structural elements in flexible systems means economic lost of the building even though the structure is not physically torn down. There are adequate systems for stiffening to achieve relatively small dimensions of skeleton structures even with high buildings. This is important to decrease building costs with material usage, and in the same time, there is a positive influence to the building function, because smaller dimensions of structural systems impose smaller limitations in space organizations. Flexible systems emphasise more the inadequate effects of second order theory; thus, it is considered better to build facilities with greater stiffness in seismic areas.

Bracing walls are primary bearing elements to accept seismic forces, so a special attention should be devoted to their design (to determine the order of these elements in the base of a building in accordance with the functional solution). Stiffening walls efficiency depends on their position. To accept torsional moments, peripheral walls are the most efficient. Adequate selection of stiffening walls position enables regulating the horizontal building movement, while sufficient horizontal reinforcement prevents breakage due to shearing forces.

The purpose of a structure can influence decision making on placing the elements with the function of accepting horizontal influences, following the structure capacity or the requirement to release façade of full wall masses; then, these structures and walls are placed in the foundation core (staircase walls, walls around lift pits, walls around installation canals, etc.). In all cases it is important to satisfy the condition on symmetrical stiffening position in relation to structure foundation. With asymmetric buildings, if they have to be built, the order of bearing elements should be such that the distance between rotation centre and gravity centre is as small as possible.

B) Structures with reinforced concrete walls, monolith or assembled, have more weight and stiffness than skeleton structures. These systems induce larger seismic forces, though their force transmitting capacity is higher, with the condition that connections are well solved with assembled structures. Problems can appear with panel buildings with transverse bearing walls, small number of not-loaded longitudinal walls, and weakened openings for doors and windows. Damage degree of well designed panel buildings can be smaller than damages appearing with flexible structural systems.

Floor structure, regardless the selection of vertical structural elements, should be stiff in its plane and capable of accepting horizontal forces and delivering them to the elements that transmit them to foundation. Floor structure has to have the ability to accept all influences without damages or large elastic deformations, so the force order and transmission should not be deranged, i.e. so that elements separation should not endanger building integrity and stability. Therefore, ceilings are an important element in overall building structure and their action as stiff diaphragm is significantly important in the case of complex and uneven position of vertical bearing elements or in the case of vertical elements of different deformation properties (combined systems). For ceiling to function as horizontal stiffener, it is necessary to provide the connections among elements of floor structure (adequate stiffness in their plane) and the efficient connection to vertical bearing elements. Special attention should be provided to the cases of non-compact or very elongated shapes in a base of a building and the existence of large openings in ceilings, especially if those openings are located near main vertical bearing elements, which actually deranges the efficient connection.

3. FLEXIBILITY OF STRUCTURAL SYSTEMS

Structural system should enable the design of a set of functional spatial purposes in a facility, i.e. it should provide necessary flexibility in accordance with the defined building function. With longitudinal massive systems, façade structural walls are of greater thickness, and therefore weight more. Architectural façade modelling is limited by structural wall function. With these systems, it is more difficult to break outline of building (tract shearing) or model loggias, and openings in facade planes are smaller. Building depths are limited by ceiling spans, so one-tract, two-tract, or three-tract systems are formed. More than three spans is not functional, except in special situations (hospitals, laboratories), when there can be four spans. When these systems are used, structural wall in the middle is mostly utilized for positioning installations (sewerage, chimneys, and rain-water pipes).

In trasvesal massive systems facades are free, relieved structural functions; thus, they can be easily and naturally modelled by loggias and broken by outlines of buildings; in other words, the size, position and shape of openings can be selected freely. Building depths are not limited. Walls between flats, which should be thicker due to acoustic requirements, are at the same time structural [13].

In systems with bearing walls in two directions limitations are the greatest due to dual direction of structural walls; hence, they unite within themselves all drawbacks of both longitudinal and lateral connections. Concerning their shape, they possess disadvantages that are characteristic to longitudinal systems; in functional sense, free expansion and connections of inner space are made more difficult. These systems are usually applied in buildings with greater action forces in ceilings (warehouses). Structurally, the system is suitable since it provides the support of ceiling onto walls in both directions, and in seismically active areas the acceptance of horizontal actions (earthquakes).

From the point of flexibility, the advantage of skeleton structural systems over massive systems lies in the fact that external and internal walls are released of the bearing function, enabling large dimension openings to be realized. Walls in skeleton structural system are realized mostly as fillings, to function as partitions, heat and sound insulation, and they can be realized transparently as well. The flexibility of structural systems in relation to the possibility for space organization and façade modelling is presented in Table 1.

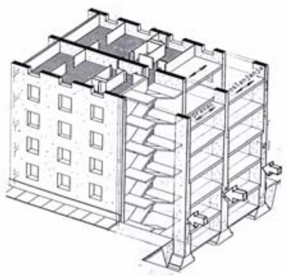
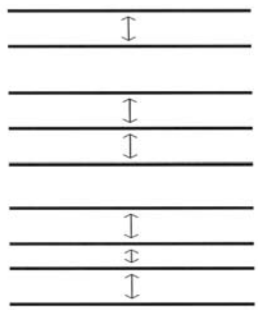
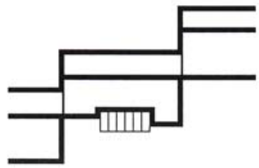
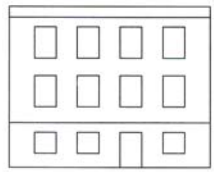
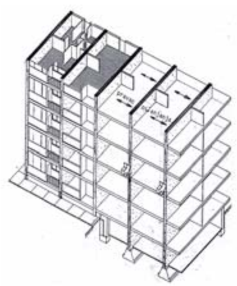
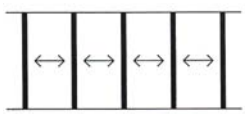
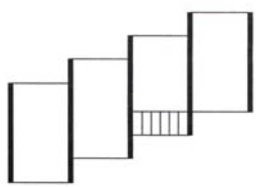
STRUCTURAL SYSTEMS	**Possibility for space organization**	**Possibility for façade modelling**
1. MASSIVE SYSTEMS **Longitudinal systems** 	The possibility for space organization is limited by the spans of longitudinal bearing walls, which condition the number of tracts (systems with one, two, or three tracts) in building depth. 	Façade modelling is more difficult, harder to break (tract shearing, loggia modelling, etc.).  Facade openings have limited dimensions due to structural wall function. 
Trasvesal systems 	The possibility for space organization is limited by the spans of lateral bearing walls.  Building depths are not limited.	Facades are released of structural role (abrupt changes of outline of building and shaping of loggias is easy).  Large openings can be dimensioned and modelled.

Table 1. Flexibility of structural systems

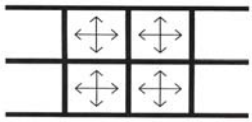
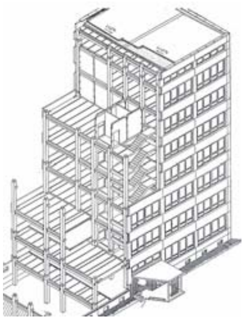
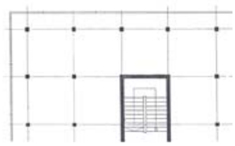
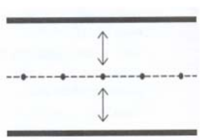
STRUCTURAL SYSTEMS	Possibility for space organization	Possibility for façade modelling
Systems with bearing walls in two directions *Massive systems are adequate to be applied with building functions requiring large number of walls (residential buildings) *With building functions requiring greater flexibility degree, skeleton systems are more adequate (administration and commercial buildings)	The largest limitations are due to dual direction of structural walls, with floor structures supported in two directions. 	Concerning modelling, they unite all drawbacks of both longitudinal and trasvesal system. These systems are suitable for seismically active areas.
SKELETON SYSTEMS 	Greater possibilities for space organization (external and internal walls are realised from bearing function; they are realized as fillings and function as partitions, heat and sound insulation, and they can be realized transparently).	Greater possibilities for facade modelling  Greater possibilities for creating large openings in walls
COMBINED SYSTEMS	Greater liberty in organizing interior space (between massive walls in central building part there is a set of posts) 	Greater possibilities for façade modelling (massive walls are in the central part, and rows of posts are on the facade) 

Table 1. Flexibility of structural systems (continues)

4. PROPOSITION FOR SELECTING A STRUCTURAL SYSTEM

Interdependency analysis of functional and structural properties of buildings for various purposes (administrative, residential, medical, educational, industrial, etc.), located in seismically active areas, has enabled the possibility to propose suitable structural systems that provide necessary reliability of a structure for seismic action, in accordance with functional requirements and necessary degree of flexibility [5]. For more qualitative and faster decision making in the phase of conceptual designing, a computer application ArhPrep has been derived; it provides the user with the proposition of possible structural system types and stiffness position in a building on the basis of the defined function, floor number, shape and dimension of a building. Based on defined criteria regularity (according to EC8) [4], [6], one also receives information on the structural treatment of the proposed design solution (analysis and calculation method). Since more complex models and calculation methods can result in lower reliability degree in the assessment of structure's real behaviour under seismic action and more investment in seismic protection, computer application ArhPrep receives more significance. It provides faster evaluation on the level of conceptual designing, enables reflecting the consequences of the proposed solution on the behaviour of the structure under seismic action, and contributes to more qualitative decision making, since wrong decisions can be corrected in the initial designing phases rather than after being realized [5].

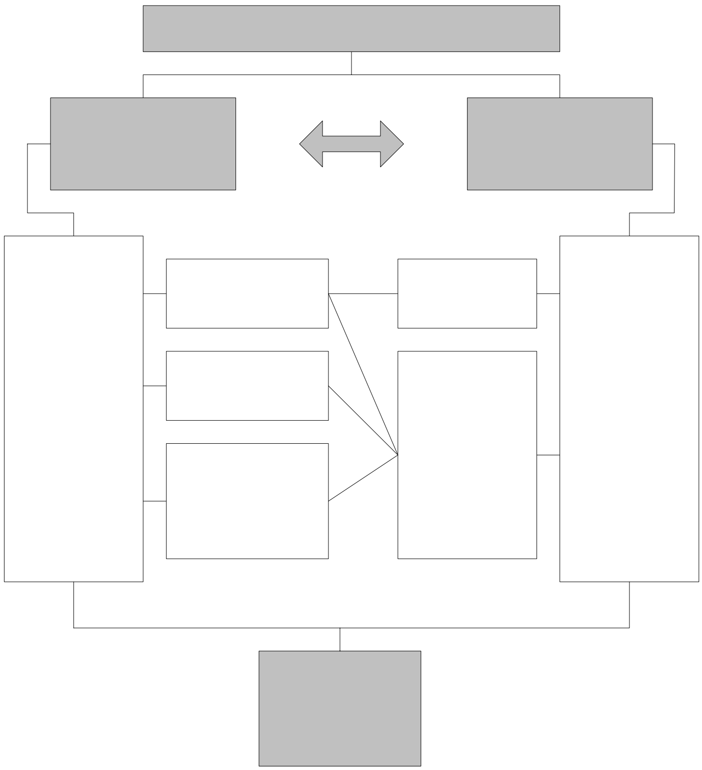


Figure 1. Scheme for selecting a structural system in the phase of conceptual designing

5. CONCLUSION

The influence of a functional solution onto building configuration and the influence of the configuration onto the behaviour of a building under seismic action are linked by functional and structural properties of buildings. First condition that needs to be fulfilled in aseismic designing is to provide proper structural concept of a building. Computer application and adequate computer application (ArhPrep) can be utilized to evaluate design solutions for buildings of different functional orientations and to consider the consequences of the proposed solution on the level of conceptual designing onto the behaviour of the building under seismic actions. Buildings with proper concept (favourable configurations, i.e. regular structures) can be relatively adequately encircled by a seismic analysis, and simpler bearing systems can be more easily modelled and analysed. Improper structural concept and/or too simplified or inadequate calculation assumptions can lead to insufficiently reliable or uneconomic solutions.

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NOVI SAD, NOVEMBER 22-24,2006.

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Zsolt Nagy[2], Aurel Stratan[3], Ludovic Fülöp[3], Viorel Ungureanu[3]

DESIGN ASSISTED BY TESTING OF COLD-FORMED STEEL FRAME STRUCTURES

Summary: *The paper summarises the results of an experimental program carried out in order to evaluate performance of pitched roof cold-formed steel portal frames of back-to-back lipped channel sections and bolted joints. Three different configurations of apex and knee joints were tested. The behaviour and failure mechanisms of joints were observed in order to evaluate their stiffness, strength and ductility. The component method was applied in order to characterise joint stiffness and moment capacity on the purpose of frame analysis and design. To check the design procedure, full-scale tests on frames were performed. Fair agreement between analytical and experimental results was obtained. Finally, some guidelines for design will be presented.*
Key words: *thin-walled cold-formed steel portal frames, back-to-back lipped channel sections, bolted joints, design assisted by testing, failure mechanisms, full-scale tests.*

PROJEKTOVANJE POTPOGNUTO TESTIRANJEM HLADNO OBLIKOVANIH ČELIČNIH OKVIRNIH KONSTRUKCIJA

Rezime: *Rad prikazuje rezultate eksperimenalnog programa sprovedenog da utvrdi performanse čeličnog krova formiranog od portalnih okvira sastavljenih od hladnooblikovanih profila i vezanih zavrtnjevima.. Razmatrane i testirane su tri različite konfiguracije čvorova preloma. Posmatrano je ponašanje i mehanizmi loma u čvorovima da bi se se utvrdila njihova krutost, nosivost i duktilnost. Komponentalni metod je primenjen da bi se odredila krutost čvorova i kapacitet nosivosti na savijanje potrebnih za analizu i projektovanja okvira. Da bi se proverila procedura za projektovanje sprovedeno je ispitivanje okvirne konstrukcije, pri čemu je dobijeno dobro slaganje analitičkih i eksprerimentalnih rezuktata. Na kraju, određene smernice za projektovanje su date i prikazane.*
Ključne reči: *tankozidni hladno oblikovani čelični portalni ram, spojeni kanali, veze zavrtnjevima, projektovanje podržano testiranjem, mehanizmi lom.*

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1. INTRODUCTION

Previous studies by Lim & Nethercot (2004) and Chung & Lau (1999) showed that bolted joints in cold-formed steel portal frames have a semi-rigid behaviour. Also, these types of joints are partially resistant (Lim & Nethercot 2003, Wong & Chung 2002). An important contribution to the global flexibility of the joints, besides the bearing effect (bolt hole elongation), is due to the deformation induced by the local buckling or distortion of the thin-walled profiles. In an unwisely configured joint premature local buckling can cause the failure of the joint itself well below the expected load bearing capacity. In case of back-to-back bolted connections, when bolts are installed only on the web of cold-formed section, the local buckling is more critical by stress concentrations, shear lag and bearing deformations around bolt holes (Dundu & Kemp 2006).

However, in case of usual cold-formed steel sections, both tests and numerical simulations have shown that bearing work of bolts associated with elastic-plastic elongation of bolt-holes is by far the most important component controlling the stiffness and capacity of such type of connections (Lim & Nethercot 2004, Yu et al. 2005, Ho & Chung 2006). The contribution of other components, such as flanges in tension and compression due to bending action, and the web in shear due to transverse action is significantly lower.

The global behaviour of cold-formed steel portal frames of bolted joints were studied experimentally by Lim (2001), Dundu & Kemp (2006), and Kwon et al. (2006). All these studies provided evidence of the crucial importance of joint performance on the global response of frames. In present paper, the stiffness and moment capacity obtained by the component method (EN1993-1-8, 2003) are used to model the global structural response. The calculation procedure is validated using full-scale tests on pitched-roof portal frames.

2. SUMMARY OF TESTING PROGRAM ON JOINT SPECIMENS

In order to be able to define realistic specimen configurations a simple pitched roof portal frame was first designed with the following configuration: span 12m; bay 5m; eaves height 4m and roof angle 10°. This frame was subjected to loads common in the Romanian design practice, totalling approximately 10kN/m uniformly distributed load on the frame. The frame was analysed and designed according to EN 1993-1-3 (2001) rules.

Elements of the portal frame resulted back-to-back built-up sections made of Lindab C350/3.0 profiles (yield strength f_y=350N/mm^2). Using these cross-section dimensions, three alternative joint configurations were designed (see Figure 11 and Figure 12), using welded bracket elements (S235: f_y=235N/mm^2).

One group of specimens (KSG and RSG) used spaced built-up gussets. In this case, bolts were provided only on the web of the C350 profile. In the other cases, where two different details were used for the connecting bracket – i.e. welded I sections only (KIS and RIS), and welded I section with plate bisector (KIP and RIP), respectively - bolts were provided on the web only, or both on the web and the flanges. Joints where bolts were provided on the web and on flanges were denoted by FB letters.

Monotonic and cyclic experiments were performed for each specimen typology, all specimens being tested statically. Details about the experimental tests were already reported by the authors (Dubina et al., 2004), present paper reviews only the main results.

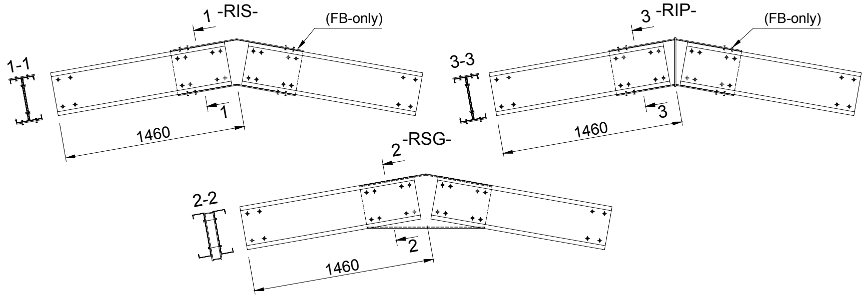


Figure 11. Configurations of ridge joints

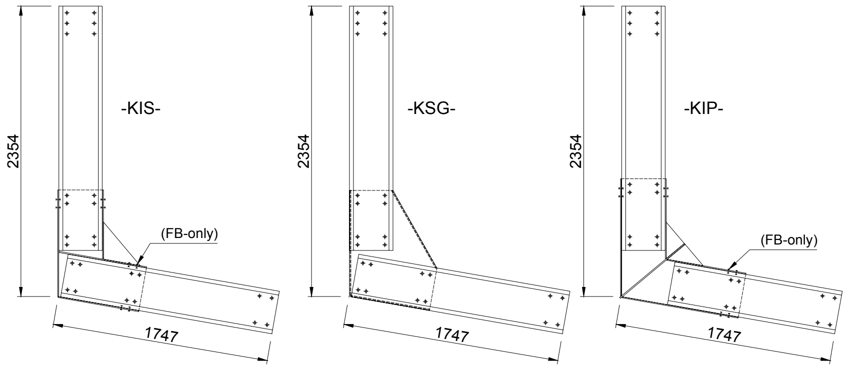


Figure 12. Configurations of knee joints

Based on tests on apex and eaves bolted joints of built-up back-to-back lipped channel sections (Dubina et al. 2004), the component method was used to characterise their stiffness and strength. The component method is a general procedure for design of strength and stiffness of joints in building frames, and is implemented in EN1993-1-8, 2003. The procedure is primarily intended for heavy-gauged construction. Its application to joints connecting light-gauge members was investigated and found appropriate with minimum set of adjustments. The modified component method for light-gauge members has been presented in detail by Nagy et al. (2006).

Based on the conclusions of experimental programme, only joints with both web and flange bolts (RIS-FB-M, KIS-FB-M, and KIP-FB-M) were investigated. Qualitative FEM simulation showed that in the case of specimens with bolts on the web only there is a stress concentration in the web, which causes premature local buckling failure. The FEM simulation also demonstrated that load distribution in the bolts is not linear. In fact, due to member flexibility and local buckling, the connected members do not behave as rigid bodies, and the centre of rotation of web bolts does not coincide with the centroid of web bolts. The centre of rotation of the connection is shifted towards the outer bolt rows, whose corresponding force is an order of magnitude higher than the force in the inner bolts. Considering this observation, only the outer bolt group was considered for determination of connection characteristics using the component method.

The configuration of the outer group of bolts being the same in the case of all three specimens with web and flange bolts (RIS-FB-M, KIS-FB-M, KIP-FB-M), a single set of analytical connection properties were determined.

A comparison of experimental vs. analytical characteristics of connections (stiffness and moment resistance) is presented in Table 2. Generally a fair agreement between experimental and analytical stiffness of the connection can be observed. Larger experimental values of stiffness can be explained by the fact that the contribution of the inner bolt group was ignored in the analytical model. Stiffness of the connection is considerably lower than the ENV1993-1-8 limits for classification of joints as rigid and therefore, these types of connections are semi-rigid, and their characteristics need to be taken into account in the global design of frame.

Specimen	Initial stiffness K_{iniC} [kNm/rad]		Moment resistance M_C [kNm]	
	experimental	analytical	experimental	analytical
RIS-FB-M	6011	5224	108.0	117.8
KIS-FB-M	6432	5224	102.9	117.8
KIP-FB-M	6957	5224	116.7	117.8

Table 2. Experimental vs. analytical connection characteristics

Moment resistance of the bolted connection $M^b_{C,Rd}$ determined by the component method amounted to 193.9kNm, which was larger than the moment resistance of the cold-formed member $M_{beam,Rd}$, amounting 117.8kNm. Therefore, this type of connection is a full-strength one. This was demonstrated also by the experimental results, failure mode being local buckling of the cold-formed member.

3. FULL-SCALE TESTS OF PITCHED-ROOF PORTAL FRAMES. COMPARISON TO NUMERICAL MODEL

Following experimental tests on cold-formed joints, two full-scale tests on frames were performed. Frames dimensions were chosen identical to the ones in the initial design used to establish the dimensions of tested joints. Considering the poor performance of joints with web bolts only, RIS-FB and KIS-FB configurations (with both web and flange bolts) were used for frame construction. Pinned supports were used at the column bases. Objective of the full-scale tests were to assess performance of pitched-roof cold-formed portal frames with moment-resisting joints under lateral loading, with particular emphasis on earthquake loading.

The test setup consisted of two frames in upward position, located 1.5 m apart. Tie bracing was provided between the two frames in order to provide out-of plane stability. Purlins were installed on the girders, but no side rails were provided on the columns. The schematic representation of test setup is shown in Figure 3. A reaction frame was used in order to apply lateral load.

In the case of the first test (C1) only lateral loading was applied. For the second test (C2), gravity loading corresponding to seismic design situation (permanent and a 0.3 fraction of the snow load) was applied, followed by increasing lateral load up to failure. Total gravity loading amounted to 31.2kN per frame, and was applied using 30 corrugated steel sheets laid on the purlins. A load cell was used in order to measure lateral load applied through a hydraulic jack. Frames were instrumented with displacement transducer to measure lateral in-plane and out-of plane displacements at the eaves, deflections at the ridge, as well as connection rotations.

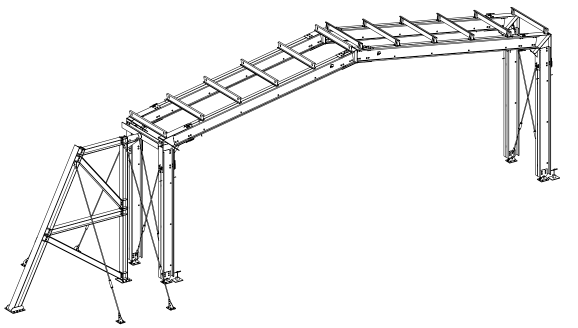
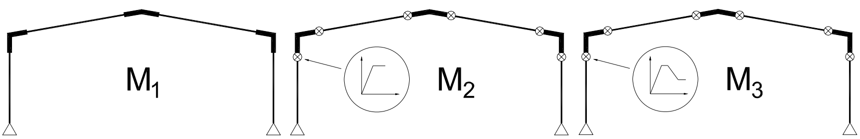


Figure 3. Experimental test setup for full-scale tests

Experimental tests on ridge and eaves joints showed that bolted connections of back-to-back lipped channels are semi-rigid, even when bolts are provided not only on the web, but also on the flanges of the lipped channel sections. Therefore, deformations can be underestimated if connections are assumed rigid for global frame analysis. To assess the influence of connection stiffness and post-buckling resistance, three frame models were analysed (see Figure 4). A nonlinear static analysis under increasing lateral load was applied to the models, and the results were compared to experimental ones.

The first model (M1, see Figure 4) was a conventional model, where connections were considered rigid. Nominal geometrical characteristics were used to model members. Finite dimensions of brackets were taken into account. Local buckling of members was modelled by rigid-plastic hinges located at the extremities of cold-formed members. Analytically determined moment capacity (M_c=117.8kNm) was considered.



(a) (b) (c)

Figure 4. Considered structural models: rigid connections - M1 (a), elastic-perfectly plastic connections - M2 (b), and degrading connections – M3 (c)

The second model (M2, see Figure 4b) was obtained from model M1 by adopting an elastic perfectly-plastic model of the connection moment-rotation response. Initial stiffness (K_{iniC}=5224kNm/rad) and moment capacity (M_c=117.8kNm) were the ones obtained using the analytical procedure described above (see Table 1).

In the case of the third model (M3, see Figure 4c), post-buckling response of the connections was modelled in addition to the initial stiffness and moment capacity. Plastic rotation (plateau) was determined assuming an ultimate rotation equal to 1.5 times yield rotation. The softening branch was determined by considering a drop of moment capacity to 50% from the maximum one, at a rotation equal to 2.5 the yield rotation.

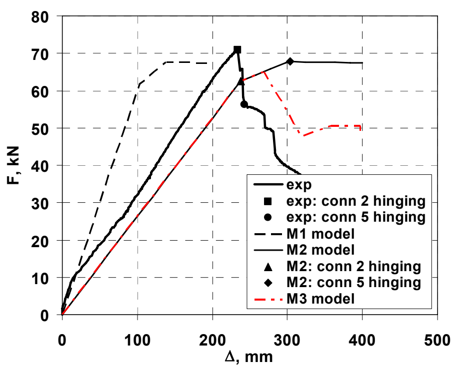
The same moment-rotation characteristics were used for all connections (for both beams and columns). Influence of axial force on the stiffness and moment resistance of the connection were ignored.

Figure 5a shows a global view of the C1 frame (tested under horizontal loading only) after the testing. Frame response during the test was characterised by an almost linear response up to the first local buckling of the beam at the connection 2 (see Figure 5b, Figure 6a, and Figure 6b), followed by a rapid loss of global frame resistance. Final collapse mechanism consisted in hinging of beam at connections 2 and 5 (see Figure 6b) near the eaves.

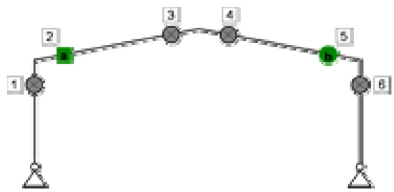
(a) (b)

Figure 5. C1 frame: global view (a) and local buckling of the left beam connection (b)

A comparison of the experimental and numerical lateral force – deformation curves for the C1 frame is shown in Figure 6. The force corresponds to one of the two frames from the experimental setup, assuming the force equally distributed between the two frames. It can be observed that the rigid model (M1) provides a good approximation of the initial response of the frame up to lateral forces of about 10kN. At larger forces, models M2 and M3, with semi-rigid connections, provide a better approximation of the experimental response. The same pattern of member hinging as in the one observed in the experiment is obtained for the numerical model (see Figure 6c for the case of the M2 model). The M3 model captures well the post-buckling response. Both M2 and M3 models slightly underestimate global frame resistance, while overestimating lateral deformations.



(a)



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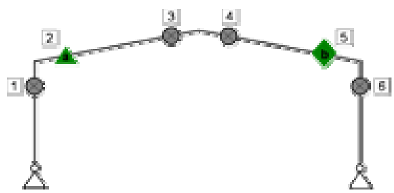

(c)

Figure 6. Frame C1: experimental vs. numerical lateral force - deformation curves (a); position of local buckling observed experimentally (b) and in the numerical model (c)

In the case of the C2 frame, gravity loading corresponding to seismic design situation was first applied, followed by increasing lateral loading up to complete failure of the frame. Figure 7a shows a view of the frame during loading. Global force-deformation response was very similar to the frame C1 up to 10-15kN lateral loading. For larger lateral loading, stiffness of the C2 frame was slightly larger than the one of the C1 frame. However, global resistance under horizontal loading was smaller in the case of the C2 frame. It was attained at the first local buckling in the beam near the right eaves (connection 5, see Figure 8b), when the lateral force resistance dropped suddenly. It was followed by a combined local buckling and lateral-torsional buckling of one of the columns at the mid-height (see Figure 7a and Figure 8b). Finally, local buckling of the beam at the left eaves was observed (at connection 2, see Figure 8b).

(a) (b)

Figure 7. C2 frame: global view (a) and local buckling of the right beam connection (b)

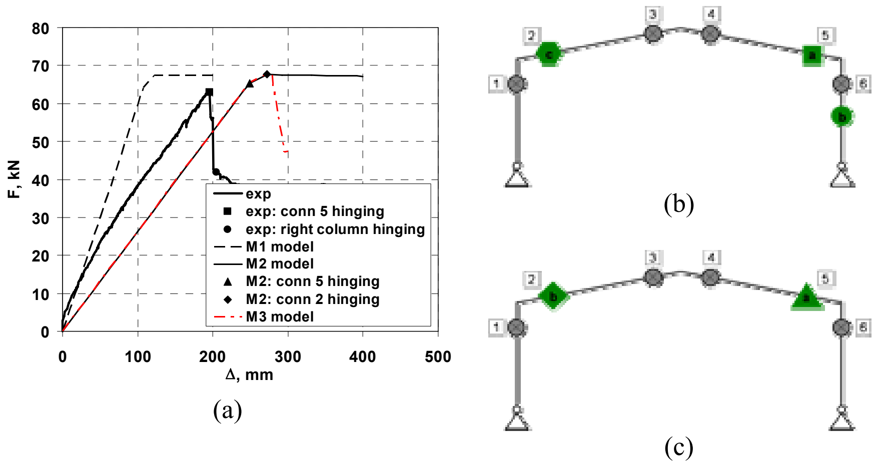

Figure 8. Frame C2: comparison of experimental and numerical lateral force - deformation curves (a), position of local buckling observed experimentally (b) and in the numerical model (c).

A comparison of the experimental and numerical lateral force – deformation response of the C2 frame is shown in Figure 8a. As in the case of frame C1, The M1 model (with rigid connections) provides a good approximation of the initial response of the frame, up to lateral forces of about 10kN. For larger forces, the other two models (M2 and M3), accounting for semi-rigid connection response, show a better approximation of experimental response for larger loads. All numerical models overestimate global frame resistance under lateral loading. This can be explained by the fact that influence of axial force was neglected when determining connection moment-rotation characteristics. Higher axial forces are present in the right column under combined effect of gravity loading and lateral loading due to the effect of overturning. While the location of first local buckling was correctly predicted by numerical model (at connection 5, see Figure 8b and c), column hinging observed in the experimental test was not confirmed by numerical models. Column hinging can be explained by neglected influence of axial force, combined with the effect of no lateral restraining at column flanges by side rails. Both of these effects were present in the experimental setup, but not in the numerical model.

It can be concluded that the M3 model seems to provide the best agreement to the experimental results, if initial stiffness, lateral resistance, and post-buckling response are envisaged. However, global frame resistance under lateral loads drops quickly after the first local buckling, when maximum force is reached. Therefore, for practical cases, response to the first local buckling in members is important, which can be estimated using a more simple frame model, incorporating only the semi-rigid connection response, eventually an elastic perfectly plastic model. Global frame stiffness determined using bilinear moment-rotation characteristics obtained analytically by the component method is smaller than the experimental stiffness. Real initial stiffness of the connection may be higher at low moments, due to restraining provided by flanges of the bracket element and/or by the inner bolt group. A connection model capable of representing this higher stiffness would provide a closer match between experimental and numerical frame stiffness.

4. DESIGN OF PITCHED-ROOF PORTAL FRAMES

In this paragraph the design of a simple pitched roof portal frame is presented. The frame dimensions were similar with the ones used for full-scale test (span 12m; bay 5m; eaves height 4m and roof angle 10°), taking into consideration for the joints the initial stiffness (K_{iniC}=5224kNm/rad) obtained using the analytical procedure of component method. For the eaves and ridge connections the configuration with both web and flange bolts was used. Pinned supports were used at the column bases. A comparison with the model where connections were considered rigid is presented.

The frame was subjected to loads according with the Romanian Standards. The frame was analysed and designed according to EN 1993-1-3 (2001) rules.

The following loads were considered in the analysis:

Permanent (P): 0.35kN/m^2
Snow load (Z): 0.72kN/m^2
Seismic load (S):13.5kN (total horizontal equivalent load).

The frame was designed for the following load combinations:

ULS: Fundamental: 1.1P+2Z
Seismic: P+0.3Z+S
SLS: Fundamental: P+Z
Seismic: P+0.3Z+S.

Table 2 presents the main results obtained for the pitched roof portal frame using the load combinations from above, for the two cases, with semi-rigid joints and rigid ones.

Main results	Frame with rigid joints	Frame with semi-rigid joints	Differences (%)
M_{eaves} (kNm)	54.51	46.62	↓16.92%
M_{ridge} (kNm)	53.83	63.3	↑17.60%
Vertical deflection (mm)	29.58 f_{adm} = 48mm	80.21 f_{adm} = 48mm	↑67.11% (compare with the admissible limit)
Horizontal displacement (mm)	28.63 Δ_{adm} = 40mm	67.1 Δ_{adm} = 40mm	↑67.75% (compare with the admissible limit)

Table 2: Comparative results

The following conclusions can be underlined:

- the design of frames is made based on fundamental load combination for both two cases (with semi-rigid joints and rigid ones);
- taking into consideration the semi-rigidity of joints, a redistribution of bending moments can be observed. For the ULS – Fundamental Combination, the moment at the eaves is almost the same as the moment at the ridge;
- taking into consideration the semi-rigidity of joints, the frame under SLS combinations became more flexible. However, according with experimental moment-rotation curves for joint specimens and full-scale tests, for the SLS combinations only the frame can be considered in the analysis with rigid joints.

5. CONCLUSIONS

For the particular case of connection studied in this paper (with both flange and web bolts), its characteristics can be determined with a reasonable accuracy if only the outer bolt group of bolts is considered. The components contributing to the stiffness and strength of the connection are: cold-formed member flange and web in compression, bolts in shear, bolts in bearing on the cold-formed member, and bolts in bearing on the bracket. It is considered appropriate to use a linear distribution of forces on bolts in the case of a connection to light-gauge members.

The connection with both flange and web bolts is semi-rigid but full-resistant. Therefore design of light-gauge portal frames with considered type of connection need to account for connection flexibility. Connection characteristics obtained using the component method (EN1993-1-8) can be easily incorporated in the structural model, in order to obtain realistic response under lateral forces. Though a detailed moment-rotation response representing the initial stiffness, moment resistance and post-buckling response provides the most realistic global response, a simple elastic structural analysis modelling connection stiffness alone can be sufficient for design purpose.

Cold-formed steel pitched-roof portal frames of back-to-back lipped channel sections and bolted joints are characterised by a rapid degradation of strength after the first local buckling in its members. Therefore, frame resistance may be estimated at the attainment of the moment capacity in the most stressed cross-section using an elastic analysis. Axial force can reduce moment resistance of cold-formed members and need be taken into account.

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TENTH NATIONAL AND
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Milinko Vasić[2]

GEOTECHNICAL CONDITIONS FOR BUILDING OF A NEW BRIDGE IN A PLACE OF THE OLD ONE ON DANUBE IN THE ZONE OF PETROVARADIN FORTRESS

Summary: *The traffic development imposes initiating of activities related to production of technical documentation for counstruction of a tunnel and a bridge in the zone of the Petrovaradin hill. With that end the preliminary analyses have been done which defined geotechnical and traffic conditions of constructing of the said communication. The old bridge was destroyed during the Second World War and now the possibilities of its reconstruction or building of a new bridge next to it are considered. In the paper are analyzed the existing data on conditions of the old bridge's foundation and geotechnical aspects of construction of a new one. For that purpose we had detailed data about the river Danube's bed, testing data of the alluvial sediments on the banks and comparatively reliable data about the line of diabases and soft rocks in the Danube's bed. All experiences and data from the field testings made for foundation of other bridges in Npvi Sad have been also used. Especially were emphasezed imprecisenesses of existing data which will be precized in the continuation of the designing and researching process.*

Key words: *bridge on the Danube, foundation, hard and soft rocks, the Danube's bed shots*

GEOTEHNIČKI USLOVI IZGRADNJE NOVOG MOSTA UMESTO STAROG NA DUNAVU U ZONI PETROVARADINSKE TVRĐAVE

Rezime: *Saobraćajne potrebe nameću pokretanje aktivnosti na izradi tehničke dokumentacije za izgradnju tunela i mosta u zoni Petrovaradinskog brega. U tom cilju su uradjene prethodne analize kojima su definisani geotehnički i saobraćajni uslovi izgradnje predmetne saobraćajnice. Stari most je u Drugom svetskom ratu srušen pa se razmatraju mogućnosti njegove obnove ili izgradnja novog pored njega. U radu su analizirani postojeći podaci o uslovima fundiranja tog starog mosta i geotehnički aspekti izgradnje novog. Za to su se posedovali detaljni podaci dna korita Dunava, podaci ispitivanja aluvijalnih sedimenata na obali i relativno pouzdani podaci o granici dijabaza i mekih stena u koritu Dunava. Takodje korišćena su iskustva i podaci ispitivanja terena koja su izvedena za potrebe fundiranja ostalih mostova u Novom Sadu. Posebno su istaknute nepreciznosti postojećih podataka koji će biti precizirani u nastavku procesa projektovanja i istraživanja.*

Ključne reči: *most na Dunavu, fundiranje, tvrde i meke stene, snimci korita Dunava*

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7. INTRODUCTION

In the General Urbanistic Plan of Novi Sad till 2001, it is planned to re-establish communication between the Backa part of Novi Sad and Petrovaradin through the Petrovaradin fortress. The destroyed bridge, where the old railroad was, will be built again on the Danube in contuation of the Tzar Lazar Boulevard (Bulevar Cara Lazara). For a new bridge the existing piers of the destroyed bridge will be renewed, or completely new bridge piers will be made, what will be precisely given by the building design. The old railroad tunnel will be widened or totally new one will be constructed and from there the traffic communication to the Preradovićeva Street. The bridge across the Danube is 437,50 m long.

The old bridge on the Danube was destroyed during the Second World War. One pier was blown up later in order to provide safe river traffic, while others are still preserved as complete. In this paper are given so far existing data about bridge base, as well as other analyses containing data of the Danube river bed, terrain stability in the bridge zone, foundation conditions of the bridge in diabases and alluvial sediments of the Danube.

8. RESULTS OF THE TERRAIN INVESTIGATION

Microlocation of the bridge is in the area composed of three seen geomorphological entities: river Danube, alluvial plane of Danube and elevated area of Petrovaradin which is final, bordering part of the northern slopes of the Fruska Gora mountain. Both the river Danube area and the Petrovaradin hill are big morphological units and present essential limits to the said traffic communication from the Tzar Lazar Boulevard (Bulevar Cara Lazara to Preradovićeva Street in Petrovaradin, i.e. along the old railroad. Namely, the Danube could be economically bridged over only by a bridge construction, and passage through the Fortress hill by means of a tunnel, as it was done in the past.

The Danube is narrowest in the location of old bridge in the zone of Patrovaradin fortress, with width of 437.5m. From that point of view the place for the bridge was well chosen. In addition to morpholodical standpoint, the bridge location is suitable regarding geotechnical properties.

The Danube’s depth is changable. Acccording to the former geodetical data the Danube’s bottom level is 65.3m, in the zone of the third pier which is in the bed on the Petrovaradin side (St. 620.75). Close to this pier (St. 579.49) the Danube is the deepest, i.e. the bottom’s level is 60.57m. In the zone of the second pier on the Novi Sad side the bottom’s level is on 67.3 m. In the zone of the first pier the bottom’s level of the Danube is 71.8 m. The water depth is changable, depending on the natural water level, and it will be subject of the hydrotechnical analyses.

With reference to riverbed bottom stability, according to conditions of geological material and terrain composition, it is certain that the riverbed bottom in zone of the third bridge pier and farther towards the Fortress is stable, because it is composed of hard rocks of diabases. They, over long period of time, were erroded by the Danube, less than case of alluvial sediments which are in contact with rest of the alluvial sediments where the Danube is the deepest. Moving of water mass of the Danube, bed’s geometry and base composition conditioned that river’s mainstream is formed exactly in that contact area of the alluvion and diabases where the river is the deepest. By means of the earlier echo sounding of the riverbed bottom and additional ones for defining the conditions of bridge renewal, also is defined the riverbed bottom geometry and it could be verified in several time intervals.

In the area about 250m upstream from the destroyed bridge there are known Danube type slides, examined in details, which imperil the road R-107. Actually they begin from the diabases' contact, attack the right valley side of the Danube and in a continuity creaye an area from the Fortress to Sremska Kamenica. The slide below the Fortress has been exemined in details, and there is an adequate project for its recovery. By its properties and characteristics it is of a know Danubian type, deep and difficult for recovery. In this paper it is presented in details, and the reason for it is the fact that those slides do not essentially impact conditions required for the building of the said bridge. Namely, that slide and the overall instable slope of Ribnjak do not directly influence the bridge. The slided material that reaches the Danube's bed also has no influence upon the bridge because the slide's displacements are specific – sliding is slow and small in quantity, as seen per the multi-year process of measuring. Erroding of the material slided into the river is such that it does not affect the downstream area, also confirmed by the fact that the old destroyed bridge never had such problems.

Morphology of the alluvial plane is comparatively simple. Namely, in the flatened part of that terrain are the Danube's branches and the youngest inundacione plains, which are hypsometrically lower for about 1-3 m. Some portions were filled in, refulled. In the zone where the bridge across the Danube prolongs into the traffic communication and farther on to Tzar Lazar Boulevard (Bulevar Cara Lazara) the terrain is arranged, while in the zone of the Danube's bank it is arranged with hard support up to level of 80.4m.

The terrain composition in the bridge zone is comparatively simple. In the riverbed the sprouts of diabases were discovered, blocks are big, solid, extremely hard, fresh. Farther, towards the Novi Sad side, there are the alluvial sediments up to depth of about 24m on the bank area. In the surface zone are alluvial-marsh sandy – dasty sediments, beneth which is sand, while the deepest part od the Danube alluvion is composed of pebles and sandy pebles.

9. GEOTECHNICAL CONDITIONS FOR BRIDGE FOUNDATION

By inspection of the huge documentation about the said bridge, including the latest detailed examinations of the bridge piers state, it was seen that there are no precise data about foundations. Around the bridge piers their is an embankment of big rocks that was discovered by the divers. According to the practice of building bridges on the Danube, from that times, it is real to assume that foundations were performed on caissons. According to all existing data the first bank pier next to Petrovaradin was founded on diabases, while others towards Novi Sad on the alluvial sediments of the Danube. By the preliminary idea design those data will be precised.

In the case of using existing piers of the destroyed bridge and need of their strenghtening, probably would be performed additional piles for piers 1 and 2 (in the Danube alluvion). In that case the piles will be consturctively connected with the existing piers.

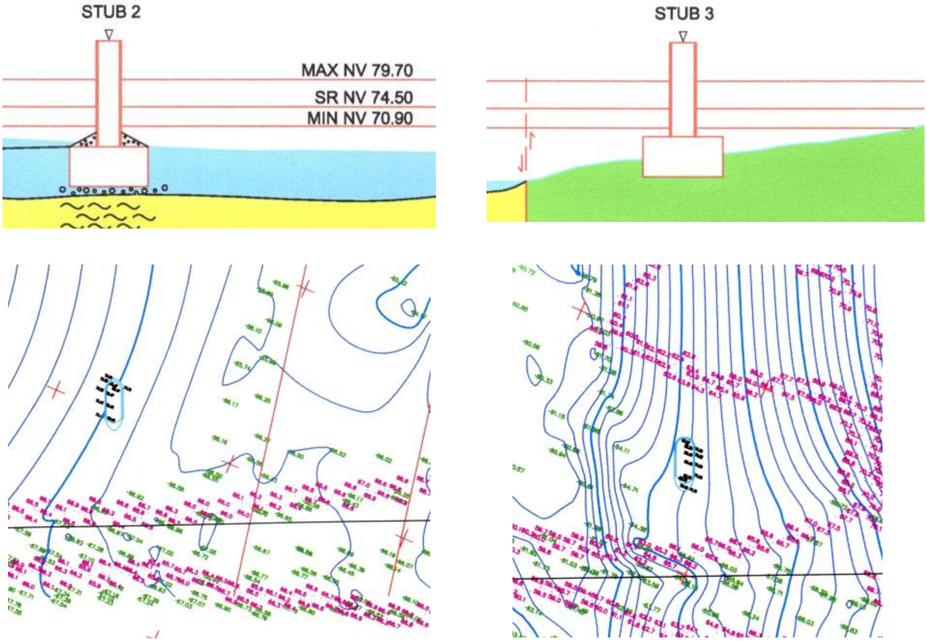

Fig. 1. Cross section of terrain in the bridge zone (Poprečni presek terena u zoni mosta)

One of the possibilities of bridging the Danube is a variant of a bridge on the newly built piers, i.e. foundations. The variant to use some of the old bridge piers, with addition of completely new piers, has not been particularly considered.

Based on the geotechnical cross-section of the terrain in the Danube's riverbed zone, as well as geotechnical map and spatial relation of the present lithologic units, there are two, clearly distinguished, geotechnical zones regarding the conditions and possibilities of the bridge's foundation. The first one compose diabases, and the second one the alluvial sandy gravel sediments. Based on the Novi Sad bridges building experience, as well as from many other numerous locations, it is certain that the new bridge should be founded so that on the diabasis are shallow foundation, while the rest of the founding shoud be on piles.

Within the zone of diabases the foundation of the future bridge must be shalow. The precised conditions that foundation will be given in the phase of the preliminary design, and after the performed drilling on the defined microlocation of the bridge's pier. According to the existing results of diabases examination they are in the zone of the Danube slightly cracked, hard, solid and represent an excellent base for foundation.

In the area of Danube alluvion, where the foundations of the future bridge will be on piles, there is enough geotechnical data from the former analyses. Namely, founding conditions of all Danube bridges in Novi Sad were analyzed and upon that a geotechnical model of terrain of the said bridge was formed. For the said model numerous data of drilling, penetration, of the Danube's banks in the bridge zone, as well as a huge number of statical penetrations from the city area, especially from the zone of the Sloboda bridge, were used. On the basis of geotechnical model of the terrain and familiar procedures the bearing capacity has been calculated for the individual piles of diameters ∅600 and

∅1200. It is poissibe to perform piles ∅600 by use of Simplex method, while a HW in piles ∅1200. Adopted diameters are real for performing in the coastal part of the Danube and in the Danubes's riverbed. The number of piles and their distribution within the group will be analyzed later, when the bridge's construction is finally adopted. Considered lengths of piles are adopted based on experiences of building bridges on the Danube, as well as on basis of real geotechnical model of terrain in the said bridge zone, and it is proposed that length of piles beneath the Danube's riverbed bottom be 20m. For the traffic structures which are on the left Danube bank the 15m long piles could be used. For the safety factor 2 the calculated permetted bearing capacity of individual pile ∅1200 is 3180kN. For the safety factor 2 the calculated permitted bearing capacity of the individual pile ∅600 is 1280kN.

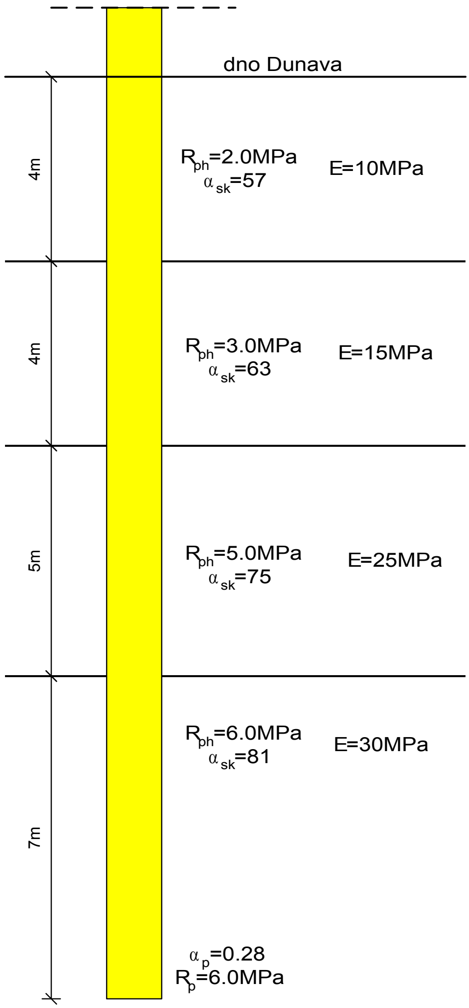

Fig. 2. Geotechnical terrain model, HW pile ∅1200 in the Danube's river bed (Geotehnički model terena, HW šip ∅1200 u koritu Dunava)

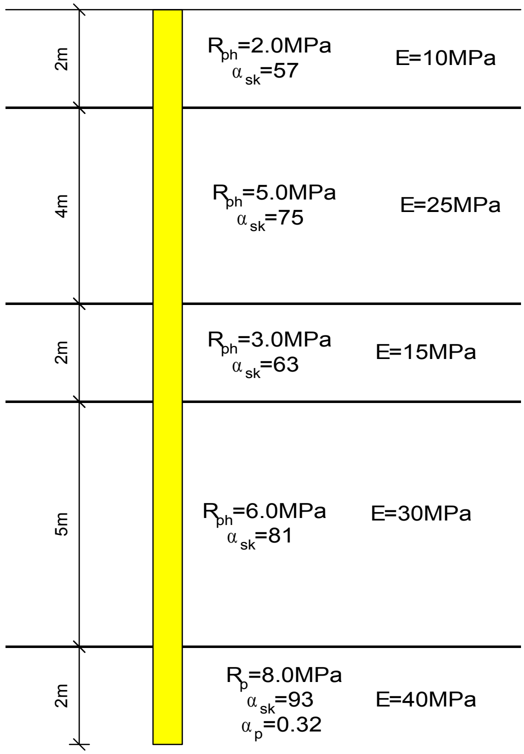

Fig. 3. Geotechnical terrain model, SIMPLEX pile ∅600 in the bank area (Geotehnički model terena, SIMPLEX šip ∅600 u priobalnom delu)

10. CONCLUSION

The paper is a concised survey of previous geotechnical analyses done for needs to start production of the technical documentation of a traffic communication which includes also the bridge across the Danube below the Fortress. In the Danube riverbed there are diabases and sediments. The foundation of the first pier next to Petrovaradin is founded on the hard diabases, while all others are foundeded in the alluvial sandy-pebbly sediments of Danube. It is certain that the foundations of old destroyed bridge have been preserved, but also that the new bridge will probably be on new piers and foundations, what will be analyzed in the subsequent designing phases. Two possible ways of founding, for that, have been analyzed: shallow on diabases and on piles in the alluvion.

11. LITERATURE

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STRUCTURAL REHABILITATION OF INDUSTRIAL CONCRETE CHIMNEYS USING FIBRE REINFORCED POLYMERIC COMPOSITES

Summary: *The paper presents the principal aspects relating to the structural strengthening solutions for industrial concrete chimneys, using fibre reinforced polymer (FRP) composites. The main advantages of FRP are based on many specific qualities of these materials, such as: high strength to density ratio, high stiffness to density ratios in case of some composites tailor ability of mechanical and thermal properties in the needed directions, good behaviour to aggressive environments, compatibility with base materials and ease of application. The objective of this paper is to investigate the issues encountered by the behaviour of industrial concrete chimneys under seismic and wind actions, technologic and climatic temperature variations. In time, normal usage and environmental influences the industrial chimney may affect the strength and the functionality of these slender structures. The paper presents modern methods for structural rehabilitation of chimneys with their related advantages and disadvantages. The details regarding the types of polymeric composite material constituents, reinforcing products, their properties, location on the industrial chimney height and application procedures of the strengthening systems are also given in the paper. A numerical analysis using finite elements method has also been performed. Various models of the chimneys according to the expected behaviour and the type of action have been utilized.*

Key words: *structural rehabilitation, industrial chimney, polymeric composites, numerical analysis.*

KONSTRUKTIVNA REHABILITACIJA BETONSKIH INDUSTRIJSKIH DIMNJAKA KORIŠĆENJEM VLAKNASTIH FRP KOMPOZITA

Rezime: *Prikazan je pristup sanciji dimnjaka pomoću vlknima ojačanog polimera (FRP) koji imaju visoke čvrstoće i njihov odnos prema gustini i dobre mehaničke i termičke karakteristike. Analiziran je dimnjak na seizmička dejstva i vetar, kao i na tehnološke i temperaturne promene. Opisani su detalji oko izbora komponenti materijala i postupak pojačavanja. Prikazana je i numerička analiza korišćenjem metode konačnih elemenata uz korišćenje različitih modela za prognozu ponašanja dimnjaka pod određenim dejstvima.*

Key words: *konstrukcijska sanacija, industrijski dinmnjaci, polimerni kompoziti, numerička analiza*

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1. INTRODUCTION

Due to various causes (disfunctionalities occurring during its life time as results of the effect of external actions specific to industrial chimneys but also of the ageing phenomenon for materials), a continuous research regarding structural rehabilitation is required.

The decay process in case of industrial chimneys begins with its erection and continues during its service life. The significant height of these structures as well as their slenderness make them a distinct category of structures, for which the deterioration process is an accelerated one.

There are no visible signs in the first service years, later on the following years, the deteriorations rapidly accelerate. Major repair works are often needed when minor maintenance procedures were not properly performed. Repair and maintenance works in case of industrial chimney stand themselves as complex procedures requiring specific technologies and equipments

2. PRINCIPLES IN STRUCTURAL REABILITATION OF REINFORCED CONCRETE CHIMNEYS USING COMPOSITE MATERIALS

Structural rehabilitation methods based on the use of FRP composites may be realized using various combinations of reinforcing fibres and polymeric matrices. The most utilized constituents of FRP used in structural rehabilitation are epoxy, polyester and vinylester resins as matrices and carbon, glass or aramid fibres as reinforcing components, respectively.

The efficiency of reinforced concrete industrial chimneys structural rehabilitation using modern materials relies on the composite-concrete area ability to undertake and transfer the stresses between components.

The make-up and the application of composite manner influence the efficiency of the rehabilitated system:

- fibre reinforced polymer (CFRP) composite plate attachments (plates obtained by pultrusion by contact or vacuum methods) on concrete surfaces using adhesive materials with proper physical mechanical properties;
- composite membrane attachment performed in situ using ”lay-up” procedures of obtaining multi-layered composites (successively laid fibre reinforcement sheets onto resin layers, thus simultaneously obtaining the composite material and bonding with the concrete support surface)
- fibre reinforcement and resin disposal in an air-tight environment consists in placement of fabric shaped fibre reinforcement over the targeted element surface and resin infusion under same air-tightening conditions. The process is performed in a closed environment; the resin infusion wets the reinforcement fibres and fills in concrete cracks.

Best results are obtained when prefabricated composite plates and membranes are attached to the concrete support by a strong adhesive material. The adhesive material must present physical mechanical properties that are both compatible with the concrete support and the composite final product.

3. STRENGTHENING METHODS FOR REINFORCED CONCRETE INDUSTRIAL CHIMNEYS USING FRP

Reinforced concrete industrial chimneys may be strengthened using FRP confinement devices applied on specific damaged areas or on their entire height. [8,9,10]. Bearing capacity enhancement can be obtained by chimney jacketing with multi-layered glass or carbon fibre sheets, as required (figure 1a). The bond between the concrete surface and the sheet is realized by epoxy resin impregnation of the later.

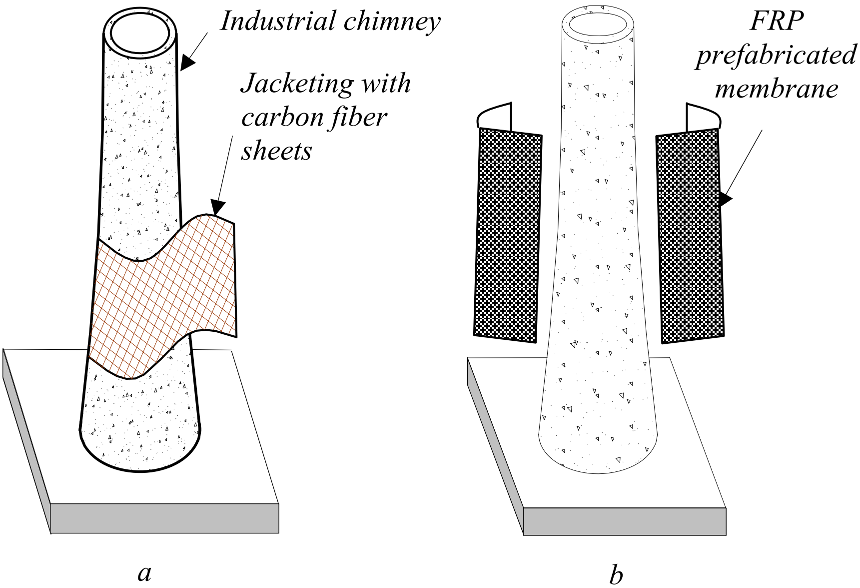


Figure 1 Concrete chimney confined with: a) FRP sheets; b) FRP membranes

In figure 1b a confining method using FRP prefabricated membranes. The rehabilitation method consists in chimney surface impregnation with epoxy resin and application of the membranes on the concrete chimney. The possible air holes are eliminated using a metallic roll. The FRP membranes present the advantage for a fast execution without occasional finishing operation.

Another strengthening alternative is based on the use of carbon or Kevlar CFRP strips. The composite strips are attached to the concrete surface by an adhesive. These strips can act as substitutes for the reinforcing transverse steel which can assure a efficiently confining effect. The distance between the composite strips, figure 2a, are previously calculated.

The solution from figure 2b is a complex method that utilizes carbon or Kevlar prestressed strands or bunches. This procedure is applied if the big transversal deformations occur and a special anchorage system is required.

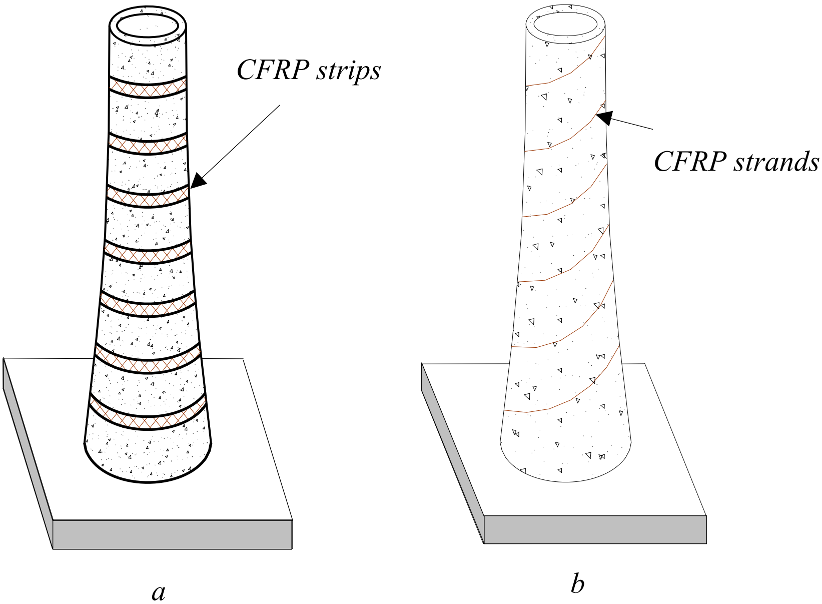


Figure 2 Concrete chimneys' strengthening: a) FRP strips; b) FRP strands

A fast strengthening method using FRP composites can be achieved using an automatic confining method. The impregnated or unimpregnated fibres are rolled under the specified slope around the chimneys' structure. To perform this procedure the operator must get trough followings steps: the elimination of the chimney accessory (stairs, platforms, signalling installations); if is necessary the repair of the concrete surface; the disposal of a resin layer for fibres impregnation; the application of the reinforcing fibres by automatically confining method.[2, 3]

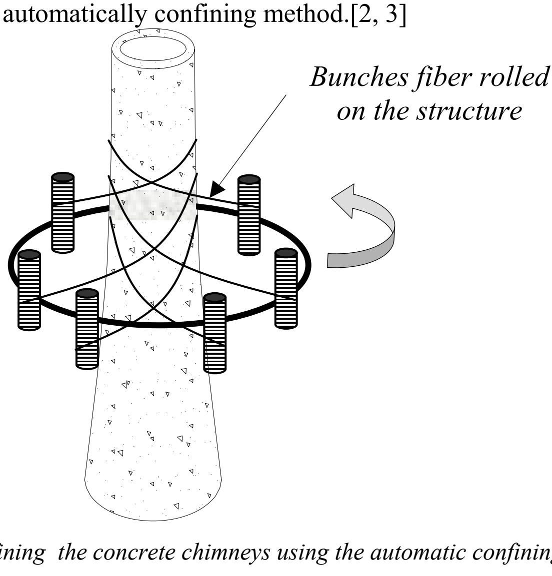


Figure 3 Confining the concrete chimneys using the automatic confining method

If the reinforcing steel is deteriorated that it can be replaced by cutting the corroded parts and overlapping them with other reinforcing steel bars or glass or carbon bars. The advantages of the composite bars are: strength to corrosion, a convenient value for the elasticity modulus and hence a ductility and strength increased of the chimney structure.

In figure 4a, 4b it is presented the replacing procedure for a deteriorated reinforced concrete chimney followed by a total concrete chimney confining, using glass or carbon sheets (figure 4c). This method is very efficient because the strengthened structure works very well with the foundation [4]. The composite bars are disposed in the special grooves cut near concrete surface and are bonded by resins.

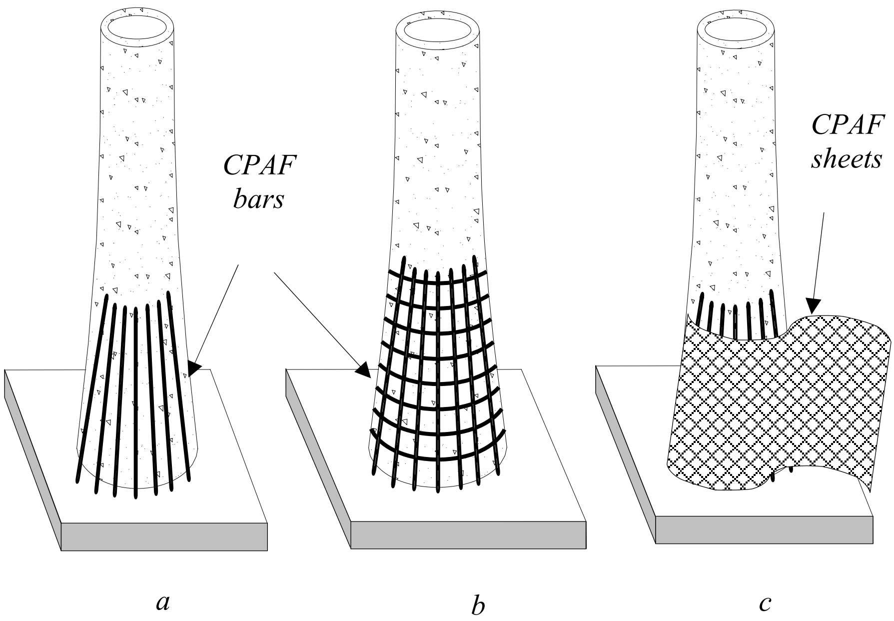
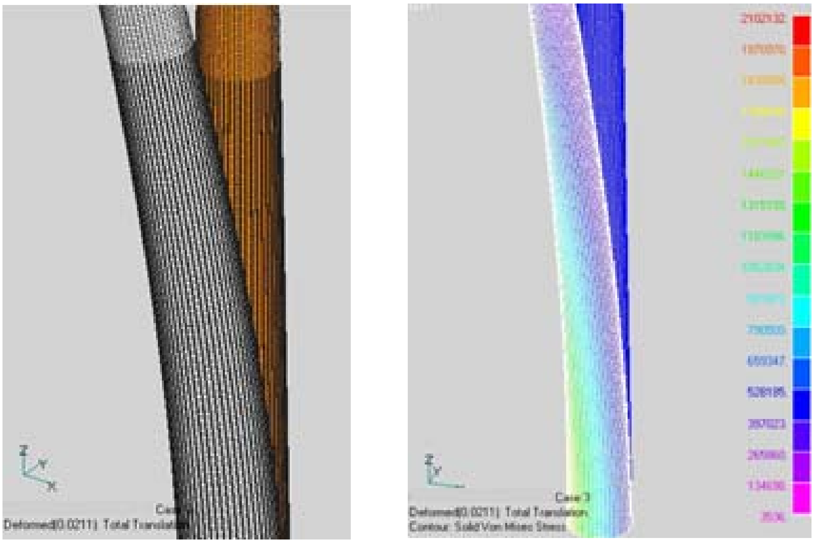


Figure 4 Strengthening of the reinforced concrete chimneys using: a) longitudinally mounted composite bars; b) composite bars arranged in longitudinal and transverse directions; c) confining with glass or carbon fabrics

The analysis of the strengthening of a 80m high concrete chimney with composite materials (longitudinal carbon reinforcing elements and carbon fibre sheets disposed to 45˚ with its vertical axis, figure 5.a) as compared to the traditional strengthening solution with 10cm concrete jacket on the 30m height and entire height of the chimney, showed the stopping of the failure process and the increasing of the strength and the stiffness of the initial chimney.

The equivalent stresses obtained by numerical analysis lead to the following results: in the chimney rehabilitated with carbon reinforce and composite sheets the stresses are 93,3% of the original chimneys and 77,2% of the system rehabilitated with the concrete coating on the 30m height and finally 88,2% of the stresses obtained for chimney strengthened by coating on the entire height [2].



a *b*

Figure 5 The industrial concrete chimney rehabilitated with composite materials. a) Disposal of the longitudinal composite reinforcement; b)Equivalent stresses in the rehabilitated chimney (compressive zone) with composite reinforcing bars and sheets [2]

The analysed structure under horizontal actions	σ_e [MPa]
Concrete chimney 80 m high	2,2489
Rehabilitated structure with 10 cm concrete coating on the first 30 m from the base	1,736
Rehabilitated structure with 10 cm concrete jacket on the entire height	1,984
Rehabilitated structure with composite materials (carbon longitudinal reinforcements and carbon sheet disposed at 45˚ with respect to longitudinal axis)	2,102

Tabel 1 The effect of chimneys strengthening solution on the equivalent stresses calculated at the bottom of the chimney

4. CONCLUSION

The traditional rehabilitation methods specified previously increase the bearing capacity and the stiffness of the structure, improve the chimneys ductility, so they improve the behaviour of the chimneys' structure but they have some disadvantages such as:

- These solutions require the interruption of chimney operation during rehabilitation works;
- Traditional systems and solution are time and labour consuming
- The concrete coating applied on the chimney increases its self weight and consequently the permanent loads which must be transmitted to the foundation and then to the foundation soil are also magnified;
- The increase of the chimney wall thickness modifies the dynamic characteristics of the structures and its dynamic and seismic behaviour;

The utilization of the fibre reinforced polymer composite materials to the rehabilitation solutions of concrete chimney gives the following advantages:

- required transversal and longitudinal reinforcing steel bars can be supplemented by using composite materials under various shapes (sheets, membranes, strips, strands);
- FRP reinforcing products can increase the strength of the concrete chimney and enhance its ductility;
- The use of the FRP composites to industrial chimney rehabilitation maintains a equally apparent section with the initial section; the structure weight is increased with insignificant values;
- The manipulation of the materials is easy and the transport is not expensive;
- The execution of the rehabilitation procedures based on FRP elements are realized in a short time;
- The FRP materials are more corrosion resistant to aggressive action than the steel reinforcing bars;
- The maintenance costs after the FRP strengthening are much reduced.

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STUDIES REGARDING THE SOLAR ILLUMINATION AND THE AIR CIRCULATION IN DENSELY BUILT URBAN SPACES

Summary: *The sunshine and the air circulation are defining elements of the urban microclimate and significantly step into the valorisation potential of solar energy (natural illumination, passive heating, photovoltaic), as well as into the natural ventilation of buildings, which has implications on the quality of the interior and exterior environment and on the buildings' energy consumption.*
The densely built urban spaces present some peculiarities from this point of view, and this fact determined a major interest from all the actors involved in the conception, the design and the execution of the urban units and of the buildings, research being a major part of the process.
A synthesis of the most important and topical studies in the field have allowed the formulation of some indicators through which any site can be characterized from the points of view of the maximum available volume in which one can build without violating the right to solar illumination and natural ventilation of the existing buildings.
The analysis of real situations existing in the large cities of Romania has shown the necessity of such an approach in urban design.
Key words: *solar illumination, air circulation, densely built urban spaces, energy consumption.*

STUDIJE VEZANE ZA SOLARNO OSVETLJENJE I CIRKULACIJU VAZDUHA U GUSTO GRAĐENOM URBANOM PROSTORU

Rezime: *Sunčeva svetlost i cirkulacija vazduha su elementi za definisanje urbane mikroklime i predstavljaju značajan korak u valorizaciji potencijala solarne energije (prirodno osvetljenje, pasivno grejanje, fotovoltaža), kao i u prirodnoj ventilaciji zgrada, što ima određene implikacije na kvalitet unutrašnjeg i unutrašnjeg okruženja i na potrošnju energije u zgradama*
Ključne reči: *solarno osvetljenje,cirkulacija vazduha, gusto građeni urbani proctor, potrošnja energije.*

1. INTRODUCTION

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The accelerated urbanization of the last decades has determined a dramatic growth of the dimensions of urban agglomerations. According to a UNO report (UNCHS, 2001), our planet has 19 cities with more than 10 million inhabitants, 22 cities having between 5 and 10 million inhabitants, 370 cities with 1 to 5 million inhabitants and 433 cities with 0.5- 1 million inhabitants. In the developed countries, the main issues created by the urban explosion are related to the excessive resource consumption, especially the energy consumption and its consequence, the pollution of the environment.

The energy consumption is one of the essential aspects which define the quality of urban life and is mainly meant for the transportation system and for the use of the buildings, meaning heating, warm water, illumination, air conditioning, and the functioning of equipments. The data referring to the energy consumption in the residential sector varies from city to city, for example from 48% in Copenhagen, to 28% in Hanover. In the same time, the buildings of the commercial sector are responsible for 20-30% from the final energy consumption in the cities.

The continuous increase of the prices for fuel, as well as the diminishing of the resources and the dependence the majority of the developed countries have towards the few countries which have big fuel resources is encouraging the preoccupation for valorising every possibility to reduce the energy consumption and to develop alternative sources.

At the urban level, the measures for reducing the energy consumption involve:

- valorising the sunshine potential with positive effects on the length of the time period for using the natural illumination and the solar energy for warming/air conditioning and water heating through the greenhouse effect;
- valorising the air circulation in order to insure natural ventilation, air conditioning and urban comfort.

The occupation density of the land, the height, the shape and the orientation of the buildings are decisively influencing the solar illumination and the ventilation potential.

The legislation regarding urban planning refers differently to these aspects from country to country, but there are three main modalities of addressing the issue:

- Prescriptive and descriptive standards, recommending the suited physical solutions for each situation. For example, in San Francisco there is a regulation limiting the height of the buildings according to the slope of the site.
- Standards based on meeting certain criteria of performance which specify the levels to be met through design. For example, the regulations regarding the zoning of Boston do not admit the reciprocal shadowing of the buildings between 8.00- 14.30. In San Francisco, a standard of performance limits the wind speed determined by the buildings in the following way: a building which shape is causing wind speeds going over 11 miles/ hour in the promenade areas and 7 miles/ hour in the stationing ones do not receive the approval to be built. Obviously, the application of these standards involves specific studies, through numerical simulation or aerodynamic tunnel in order to determine if the performance criteria are met.
- Studies elaborated as a part of the impact research. For example, in New York, the supporters of an important project of urban development finance the studies regarding the action of the wind. Their purpose is to make sure that the wind speed in the new designed open spaces does not exceed the medium speed

registered in similar urban spaces, which are considered satisfactory from the point of view of urban comfort.

As it results from the above, there are no signs referring to regulations regarding the valorisation of the natural factors specific to a certain urban context in order to reduce the energetic consumption in the exploitation of buildings.

In Romania, the existing regulations fit in the first category; they have a prescriptive- descriptive character, and the criteria they recommend to be respected regard exclusively insuring the intimacy and some minimal values for natural illumination and sunshine specified in norms elaborated by the Department of Health (Order no. 536/1997 regarding the approval of the norms of hygiene and the recommendations regarding the life environment of the population). These criteria have been adequate to the planning of the building blocks specific to the modern period, in which the shape and the arrangement of buildings was quasi standardized. The actual dynamic of urban development, uncontrolled in certain situations, as well as the imperatives of sustainable development, calls for another way of addressing the issue, based on integrative principles that would loyally reflect the relation between the natural environment, the building and its user.

2. SOLAR ILLUMINATION: THE SUNSHINE POTENTIAL AND THE VALORIZATION OF SOLAR RADIATION

In order to contribute to the meeting of the objectives of sustainable development, an intensive exploitation of the solar radiation incident on the facades of the buildings built in urban areas is not only desirable, but also feasible. Modern technologies, as well as the simulation instruments allow the valorisation not only of the direct solar radiation, but also of the diffuse one. The indicators through which a site can be characterized are defined as the potential of passive solar warming, the potential for the photovoltaic system integrated into the building, the potential for natural illumination and the value of the annual irradiation of the façade.

The orientation and the reciprocal shadowing can be analyzed through solar diagrams. For complex situations, determined by the placing of certain high buildings in urban areas of reduced height, the solution is to resort to building a solar envelope, through numerical simulation. It indicates the maximum available volume for building without crossing the right to solar illumination to the neighbouring residential areas.

The solar envelope has been used to provide the first information for the design of a complex business district developed on the vertical in Tel Aviv, in a densely built area, in the purpose of insuring the right to solar illumination. The maximum admissible height of the future buildings was determined, so that the solar illumination of the neighbouring buildings would be possible throughout the winter, between 8 a.m. and 3 p.m.[10] Besides this, another request was also formulated, so that the two main streets orientated east-west, which insure the access to the inhabitants to the railway station would benefit from sunshine in the same time period.

Respecting the solar envelope as a starting point in design, some building have been modified , as well as shape, height or have even been displaced to another location.

Regarding the evaluation of the natural illumination potential, the existing simulation instruments allow taking into consideration the reflected component, which is very important in very dense urban spaces (the RADIANCE program), as well as the multiple transparencies that appear in the case of the atrium, of the greenhouse type of space of in the double façade cases (the SOLENE program) [10].

3. AIR CIRCULATION

3.1. The natural ventilation potential

The use of natural ventilation has ceased to be exclusively the characteristic of buildings with a reduced number of levels, so that lately the system imposed itself at an increasing number of high rise buildings- office buildings or other, in the purpose of reducing the energetic consumption and the contamination risks with various bacteria (Legionella) which are associated with the air conditioning installations.

Valorising the natural ventilation potential specific to the different urban districts, according to [1], [2], [3], is based on the following principle: for a building situated in a specific urban ensemble, the differences of transversal pressure on the building have to be insured and known, and on their basis the function of the flux of exterior air can be optimized by the design and the positioning of the openings, so that the optimal rate of ventilation is realized, respecting the conditions of comfort linked to the moving speed of the air.

It must be mentioned that using the natural ventilation involves some important constraints regarding the quality of the exterior air (the content of polluting substances), the noise level, but also the lack of methodologies and design instruments. The URBVENT project (2001- 2004), coordinated by the La Rochelle University, France, in which have been involved five university laboratories, two technical centers and industrial company, proposed itself to fill this void. The methodology developed in the framework of the project associated with a calculus program, focuses on insuring the quality of the interior air, while also taking into account the other objectives of natural ventilation, related to comfort and the energy economy. [1]

According to [1], the natural ventilation potential depends on an important number of variable parameters, some of them related to the general climatic characteristics of the site, some depending on the local conditions, like the urban structure, which determines the speed and the wind direction, the air quality, the noise level, etc. (fig.1). On the other side, the ventilation potential depends on the constructive characteristics of the building, like permeability, the position and the dimension of the openings, the orientation in relation to the dominant wind (fig.2). The proposed methodology integrates the two categories of constraints, defining a potential for the natural ventilation of a certain building, placed in a specific place. A case study realized for Seville could serve as an example:

- the ventilation potential of the site is defined by the medium wind speed, the medium difference of temperature between interior and exterior (the thermic drawdown), the time fraction in which the natural cooling of the air is possible, during day and night time.
- the possible flow of air, conditioned by the position and the dimensions of the openings.

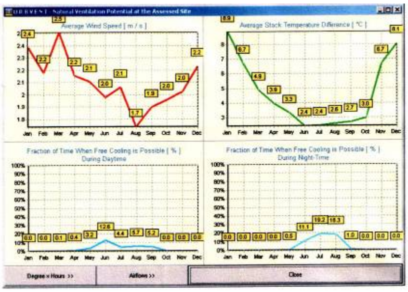


Fig.1. Graphic of the potential of a site (Seville)

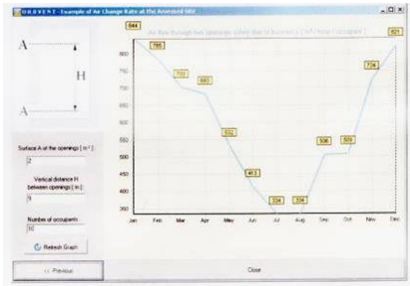


Fig.2. Potential air debits

3.2. The transport of the polluting agents

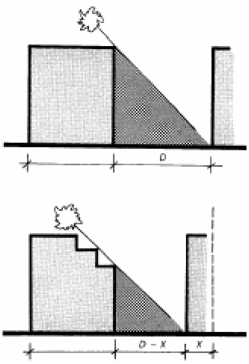
Knowing the directions of the wind currents inside the densely built urban areas is as important as identifying the polluting sources and their position, in order to optimize the urban planning, and more specifically to protect the residential districts. In this context, a new approach is imposed. If until not so long ago, the building was considered individually or as part of a small group, sustainable urban planning needs to consider larger ensembles of buildings, going up to an entire town, with the reciprocal relations between them and their environment. It is necessary to adopt complex models, based on statistical studies, capable to reflect the interrelation between buildings, transportation and industry, as well as the potential of valorising the unconventional energies. A model of this kind was elaborated by the School of Architecture of Cardiff, UK, serving as an instrument for the sustainable urban development in the sense of reducing the energy consumption and the CO_2 emissions, the proposed objective being a 30% reduction for the year 2007. The model initially applied for the city of Cardiff is now used by the local authorities from the big cities in the UK as a mean of prediction of the effects of the urban development decisions at the level of the whole city and implicitly at a local level, from the neighbourhood to the building. [9]

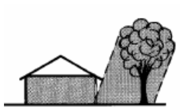
4. THE IDENTIFICATION OF SPECIFIC URBAN STRUCTURES FROM THE PERSPECTIVE OF THE SOLAR ILLUMINATION AND THE AIR FLOW, FREQUENTLY MET IN THE CITIES OF ROMANIA

The length of the present paper does not allow a detailed analysis from the point of view of solar radiation and ventilation, of organisation types in the big ensembles of collective housing. Bringing forth the premises of various analysis, aiming to optimize the design of the new districts of housing, e it individual or collective, is mandatory. The cardinal orientation, the sunshine-illumination, the direction of the dominant winds, the volumetric configuration of the buildings, the economy of energy, all these can be main criteria for such an analysis.

1. 2. 3.

1.Solar radiation-control of the section of the buildings in densely built areas;
2.The position towards the neighboring structures- natural obstacles;





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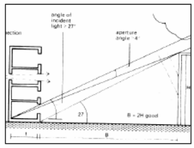

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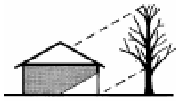
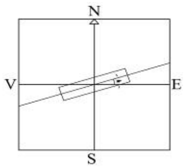
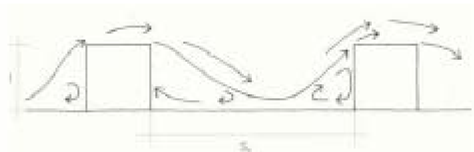




Iarna

Iarna

1.



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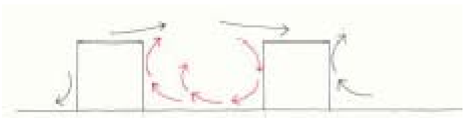

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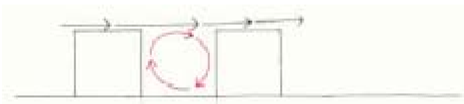


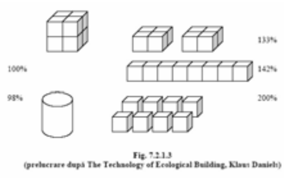
Main variations of the air flux depending of the distance between buildings.

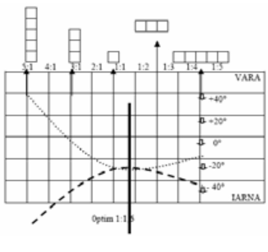
1.S > 2,4 H   -best ventilation

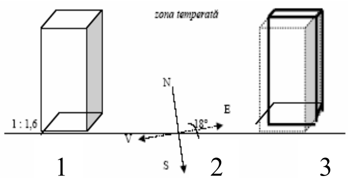
2. 1.4 < S < 2.4 H

3.S < 1,4 H

The correct use of the wind and of the pressure exercised over the envelope can lead to natural ventilation, even for very high buildings.



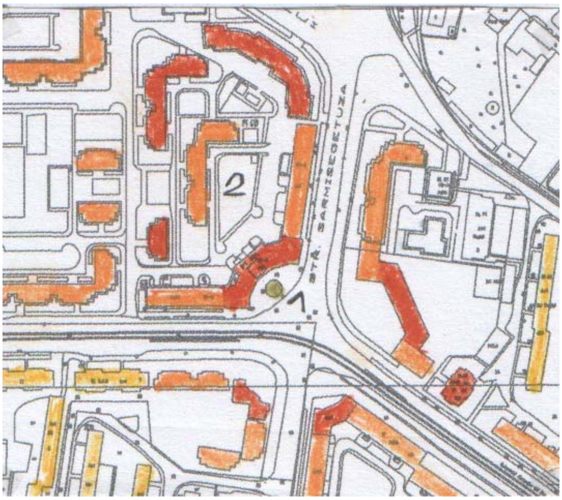




The rapport between the volumetric configuration / the necessary energy consumed to maintain the comfortable temperature in a building.

Optimal rapport between the dimensions in plan of the built surface : +40 degrees +40 grade-heat earning 10000 BTU/day, - 40 degrees- loss

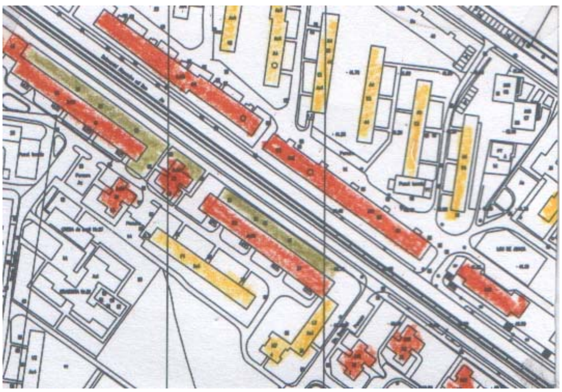
1.The shape of the building, the optimum rapport between length/width 1/1,6
2.The optimum orientation of the functions, the optimizing of the relation between the sunshine-the heat necessity.
3.Optimum repartition of the tampon spaces.



1. Semi closed plazas
-insufficient solar radiation
-circular currents
-insufficient ventilation
(winter- summer)

2. Closed precinct – excessive shadowing
-strong temperature differences compared to (1)

T-shaped plazas
-Unequal sunshine- on the longitudinal side on the surface of the plazas
-strong currents in the winter
-air stagnation in the summer



Tunnel streets
-strong shadowing >1/2 of the width of the street in the summer – total in the winter
-strong air currents
-in the summer – air stagnation

Examples of types of urban organisation, Iasi, 60 thousand inhabitants

5. CONCLUSIONS

The accelerated development of the urban localities determines important microclimate changes, associated with the increase of the energetic consumption and the pollution degree. In these conditions, a new approach of the urban planning is imposed, one that would take into consideration the valorizing of the solar illumination potential and natural ventilation potential in the purpose of reducing the energy consumption and the pollution degree.

Numerical simulation offers to planning useful instruments for the analysis of real situations, specific to every site, in the perspective of selecting the optimal solution for both new urban complexes, as well as in matters of urban rehabilitation problems.

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NOVI SAD, NOVEMBER 22-24,2006.

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Anka Starčev-Ćurčin[2]

BEHAVIOUR OF REINFORCED CONCRETE GIRDERS WITH CRACKS – APPLICATION OF FRACTURE MECHANICS

Summary: *The appearance of cracks in reinforced concrete (RC) elements significantly influences on durability and serviceability of these structures. Extending of durability and increase in serviceability are realised, beside application of other measures, also by limiting the width of the crack opening. Following the appearance of cracks, the state of ultimate equilibrium occurs in statically determinate structures; the formation of the first plastic hinge causes collapse of the structures, whereas in statically indeterminate girders, a redistribution of stresses occurs, caused by the stiffness changing, until attaining the ultimate equilibrium state. It is possible to introduce the stiffness changing with propagation of the cracks by the use of Fracture mechanics. In this paper a design model for analysis of stress-strain section of the beam in case of complex bending by application of fracture mechanics is presented. The described example of determination the crack width, points to the significance of providing the bond between the reinforcement and concrete.*

Key words: Reinforced concrete element, cracks, bond stresses, mode of cracks, stiffness changing

PONAŠANJE ARMIRANOBETONSKIH NOSAČA SA PRSLINAMA - PRIMENA MEHANIKE LOMA

Rezime: *Pojava prslina u armiranobetonskim (AB) elementima bitno utiče na trajnost i upotrebljivost ovih konstrukcija. Produženje trajnosti i povećanje upotrebljivosti se ostvaruje, pored upotrebe drugih mera i ograničavanjem širine otvora prsline. Nakon nastajanja prslina javlja se kod statički određenih konstrukcija stanje granične ravnoteže, pojava prvog plastičnog zgloba izaziva lom konstrukcije, dok se u statički neodređenim nosačima, usled promene krutosti, javlja preraspodela statičkih uticaja sve do postizanja granične ravnoteže. Uvođenje promene krutosti sa propagacijom prslina moguće je korišćenjem Mehanike loma. U radu je prikazan proračunski model za analizu naponsko-deformacijskog stanja preseka grede u slučaju složenog savijanja primenom Mehanike loma. Prikazani primer određivanja širine prsline ukazuje na značaj obezbeđenja prianjanja između armature i betona.*

Ključne reči: Armiranobetonski element, prsline, napon prianjanja, oblik prslina, promena krutosti, preraspopodela uticaja

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1. INTRODUCTION

It is difficult to comprise the exact scope of specificities in deformation of reinforcement and concrete during designing the reinforcement-concrete (RC) structures. The most often used is the design on the section model, most often the crack design, which introduces the equilibrium conditions of the cut-off parts, the assumption of stress and strain distribution and the criteria of reaching the bearing capacity. In order to use an object free of risk, it is necessary to provide not only its safety but also durability which is most often controlled by the crack design [8]. The crack opening has to be maintained within the determined limits which depends on the environment of the structure. In elements and structures, not only the section but also arrangement and number of cracks have been observed which all influence on their behaviour. The appearance of multiple cracks of lesser openings has always been more convenient than the appearance of a single larger size crack-fracture. The cracks are influenced by correct designing of details (reinforcement diameter and bar distance in the tensile zone of concrete). The linear stress-strain bond of steel and concrete has been used for section analysis together with additional contribution to strengthening of the concrete in tensile zone and the effect of shrinkage and creep of concrete.

The application of Fracture mechanics in concrete structures has not been developed as it is in steel structures. It is of interest to present its application by simple examples and to point to the possibility of its use, which was the aim of this paper [7]. It presents a short overview concerning the application of Fracture mechanics in the analysis of RC structures, starting with the papers of Grift published in 1920, which allowed the application of Fracture mechanics in the analysis of brittle materials. Its application dated in the sixties years of the last century with constructing concrete structures (CS). The example is the model of smeared cracks, applicable in small concentration of cracks. The work of Technical Committees of American Concrete Institute resulted in publications [2] and [6], whereas the control of cracks in RC and previously reinforced structures as well as the analysis of frame structures for seismic influences was presented in [3].

As far as RC structures in service state and the state proceeding the fracture have been exposed to cracks of different size and width, a large number of papers deals with to the problem behaviour analysis in these states [4], [5], [8] and [11]. The paper [8] deals with idealized conditions of cracks (vertical to the axis of element, spreading all over the height of the tensile zone), at constant bending moment and normal force. Also, the assumption is that all cracks are of equal width and equally distributed along the length of the element, whereas only width of cracks changes during the time. The behaviour of RC elements following the appearance of cracks has been analysed in paper [5]. However, the most serious study of this behaviour is presented in the book [12] with particular accent on influence of the properties of strain on possible redistribution of forces in RC statically indeterminate girders. In statically indeterminate structures, the distribution of stress and strain significantly depends on the stiffness of each section and the levels of load they have been exposed to. Both in the phase of service state and proceeding fracture the cracks cause a redistribution of the bending moment and consequently the stresses in some RC sections. The study [12] presents further analysis of the Theory of ultimate equilibrium which is of significance for analysis of redistribution in the phase proceeding the collapse (formation of plastic hinges).

The theory of the design of RC linear structures with cracks, by application of Fracture mechanics is given in the study [10]. A valid contribution to this domain is the paper [1] which uses the concept of Fracture mechanics in estimation of ultimate bearing capacity of RC beams. The paper [7] gives a proposal of improvement of this method, because the paper [1] did not comprise the behaviour of concrete and reinforcement at the condition of fracture. The ultimate analysis of RC structures is based on a assumption that concrete does not transfer the stresses in tensile zone, whereas in the compressed zone it is described by elastoplastic behaviour. This analysis does not take into account the change in stiffness following the propagation of the crack and the singularity of stress on the tip of the crack. These effects can only be taken into account while applying the concept of Fracture mechanics, [7] and [10]. In order to improve the model, the fact that elastic strain of reinforcement occurs at the site of cracks was taken into account.

This paper gives a concise presentation of the design of cracks using the Fracture mechanics according to [10], illustrated through a single example. Some possibilities of application of this theory in practical analyses of behaviour of elements after the appearance of cracks, have been presented. In this scope a comment is given on the possibilities of estimation of redistribution of bending moments proposed by the standards [4] and [8].

2. ESTIMATION OF CRACKS BY THE USE OF FRACTURE MECHANICS

When the stresses in concrete reach the tensile strength of the concrete, the cracks occur in RC elements. The minimal percentage of reinforcement μ_{min} of the section is determined on the condition that there was no appearance of premature formation of cracks [11], i.e. brittle fracture of section - when the ultimate moment M_u is higher than the moment at the appearance of the first cracks M_p. Usually, the crack design is performed at the level of section [4] and [8]. By the appearance of cracks and their development, the stiffness of RC section exposed to bending decreases, which caused increases deformations in statically determinate structures, whereas in statically indeterminate structures, a redistribution of static influences occurs. The behaviour of RC element before and after the appearance of cracks is presented in *Fig 1*. In case with no crack, deformations in the beam are insignificant. With appearance of cracks (state 2) deformations increase, until the plastic hinge is formed (state 3), and the determinate beam transfers to the mechanism. On the contrary, in statically indeterminate beams, due to the redistribution of bending moments, only the formation of a larger number of plastic hinges leads to the collapse mechanism (either local or complete).

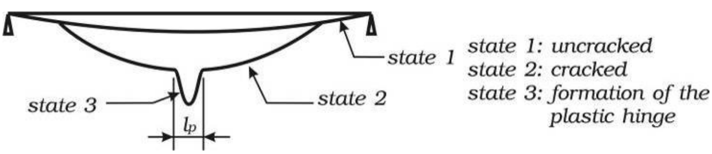

Fig 1. Behaviour of RC beam under different levels of load

In estimation of real behaviour of RC elements, a limited redistribution of influences has been used, under the conditions defined by a limiting percentage of reinforcement μ_{lim}=0.405f_B/σ_v. According to [8] the allowable redistribution of bending moments is: 10% for μ_1-μ_2=0.5μ_{lim}, 15% for μ_1-μ_2=0.25μ_{lim}, 20% for μ_1-μ_2=0, where μ_1 and μ_2 are the coefficients of reinforcing by tensile and compressed reinforcement. According to [4], for the continuous beams and slabs which are dominantly exposed to bending, under the condition that the ratio of adjacent spans is not higher that 2, the value of redistribution of the moments is estimated under the following condition: $\delta \geq k_1+k_2x_u/d$, for $f_{ck} \leq MPa$; $\delta \geq k_3+ k_4x_u/d$, for $f_{ck}>MPa$, $\delta \geq k_5$ in case of the use of the reinforcement class B and class C (Anex C [4]), and $\delta \geq k_6$ when the reinforcement class A (Anex C [4]) has been used. The value δ presents the ratio of redistributed moment versus elastic bending moment , x_u is the position of neutral axis within the section following the redistribution, d is static height of the section, k_i are the coefficients given in national standards.

During the process of development of cracks [10] in the broad zone of RC linear element, the two stages can appear: the initial stage comprising the formation of cracks and the final stage which presents the complete opening of the wide mouth cracks. Particular cracks in concrete appear because of its unhomogenous composition, presence of manufacturing defects, initial stress and temperature deformations already during the low levels of stress. Therefore the stage of formation of the cracks is the state of structure at which the most highly stressed areas undergo the process of redistribution of micro cracks into macro cracks.

The basic assumption of the Leonov-Panasyuk's design model has been used for classification of the formation of cracks according to [10], after which the category of micro cracks comprises the cracks - the openings of which do not exceed the size of δ_k, whereas the stresses can be equal to the concrete tensile strength of R_{bt}. The paper [10] introduces following the design assumptions for the crack model of RC element:

- distribution of deformations for the block between the two cracks, by section height corresponds to the hypothesis of plane sections,
- the compressed concrete and tensile reinforcements are described by nonlinear graphs, stress-strain (σ-ε) obtained experimentally, whereas the compressed reinforcement follows the elasto-plastic model,
- concrete shows linear-elastic deformations up to the stress corresponding the concrete tensile strength of R_{bt};
- the zone before the fracture, depth r_y, which is by surface stress parallel with the level of the crack, acting at the tip of the crack presented by the force $N_{crc}=K_I^2/(\pi R_{bt})$ at the crack opening displacement of $y_c=r_y/3$ from the crack tip, is appearing when the stress in the concrete exceeds the concrete tensile strength R_{bt} (*Fig 2*).

In case the spacing exceeds the value of δ_k and if there is no interaction between them, the cracks are treated as macroscopic ones, developing the micro openings at the tips.

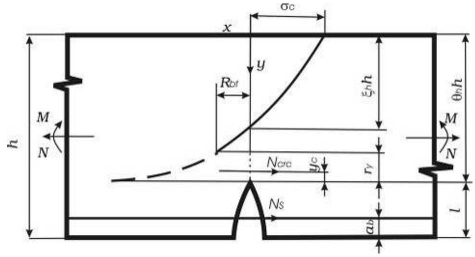

Fig 2. Presentation of participation of tensile stress in concrete above the crack

The stress-strain state of the RC beam has been presented, which, under the effect of the bending moment *M* and normal force *N*, formed the system of cracks h_{crc} of depth, at the distance of l_{crc}. The cross-section of the beam is rectangular, $2y_0$ high and *b* wide, reinforced by longitudinal reinforcement placed within the lower and upper zone of the section (*Fig 3*).

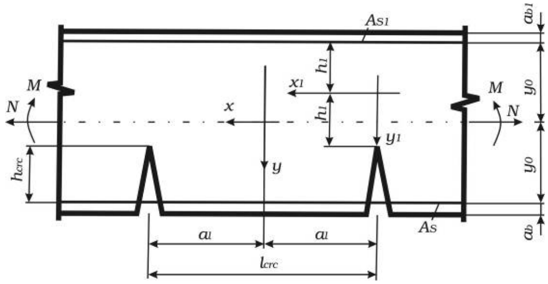

Fig 3. The scheme of reinforcement-concrete element with cracks

The reinforcement area has been distributed across the section width, by ignoring depth of the protective layer of reinforcement, so that it is considered that the reinforcement has been placed at the margin fibres of the section. The stresses of the beam at the crack can be presented by the following equation:

$$\sigma_{x1}\left(k \cdot l_{crc}, y\right) + \sigma_{x2}\left(k \cdot l_{crc}, y\right) + \sigma_{x3}\left(k \cdot l_{crc}, y\right) = R_{bt},\ y \le h_{crc}, k = 0, \pm 1, \pm 2, \mathrm{K} \qquad (1)$$

in which the first member σ_{x1} of the equation represents the stress within the cross-section of the beam under the influence of external load with no influence of cracks (the quantity of stress in tensile zone can exceed the R_{bt}); the second member σ_{x2} defines the stress caused by the phenomenon of adhesive forces between the reinforcement and concrete during the formation of cracks: the third member σ_{x3} is the stress at the point of the crack caused by the distributed loads, which compensates (through R_{bt}) the excessive cumulative tension of the the first two members. It has been analysed as a plane state of the stress.

The condition of limiting the crack openings by the value δ_k has the following form:

$$u\left(k \cdot l_{crc} + 0, y\right) - u\left(k \cdot l_{crc} - 0, y\right) = 2 \int_{y}^{h_{crc}} \mu\left(t\right) dt \le \delta_k \qquad (2)$$

in which the $\mu(t)$=-$d\lambda(t)/dt$ is the function of crack formation (*Fig 4*), according to [10].

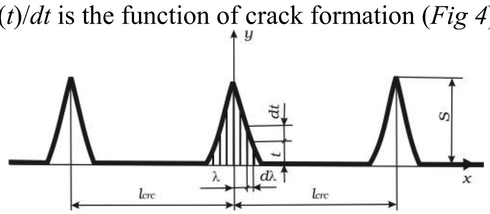

Fig 4. Mode of cracks

The model of Leonov-Panasyuk describes the relation between the opening and tip of the crack:

$$\left(\partial u(k \cdot l_{crc}, y)/\partial y\right)\Big|_{y=h_{crc}} = \mu\left(h_{crc}\right) = 0 \qquad (3)$$

The systems of (1), (2) and (3) equations describe the stress-strain condition of a beam at the site of crack formation.

The mode of cracks depends on bonding of the reinforcement and concrete, which has been presented in the paper [10] as a distributed stress between the reinforcement and concrete along the length of plane element, described by linear dependence along the length y=0:

$$\tau\left(a_0\right)=\begin{cases}-\tau_c\left[\left(kl_{crc}+l_a\right)-a_0\right]/l_a\,, kl_{crc}\le a_0\le kl_{crc}+l_a\\ \tau_c\left[a_0-\left(kl_{crc}-l_a\right)\right]/l_a\,, kl_{crc}-l_a\le a_0\le kl_{crc}\,, k=0,\pm1,\pm2,\mathrm{K}\end{cases} \tag{4}$$

in which τ_c is a maximum value of bond stress; kl_{crc} are distances of points at which maximal values of bond stress appear; l_a is the length of participation of active section of reinforcement and concrete (the length at which the bond stress appears); a_0 is the current value of the coordinate (x-line) [10].

2.1. Design equations at the stage of service state

The ratio of stress and strain is linear during the development of cracks with an increase of load without dynamic influences. By approaching the tip of the crack it becomes less linear, although this region is rather small when compared with the whole length of the crack, at least several *cm*, which can therefore be classified as a macro crack. The element is exposed to the effect of bending moment M and axial tensile force N. If the full section height of the element is marked by $2y_0$, the height of un cracked area of the cracked section by $2h_1$, then the height of the crack is $h_{crc}=2y_0-2h_1$ (*Fig 3*). The following relative coordinates can be introduced with $\xi=x/l$, $\eta=y/y_0$, $\xi_1=x_1/a_l$, $\eta_1=y_1/h_1$ and marks $\gamma_{lh}=a_l/y_0$, $\lambda_0=h_1/y_0$, $e_l=h_{crc}/2y_0=1-\lambda_0$. Following the development of a crack, the boundary conditions for stress-strain state of the element are such that, at extension of the crack during the whole process of loading, there are only linear loads during the interval $-y_0\le y\le y_0-h_{crc}$ which remain linear, so that $\tau_{xy}=0$, $u(x)=c_1+c_2\eta$, whereas during the interval $y_0-h_{crc}<y\le h_{crc}$, $\tau_{xy}=0$, $\sigma_x=0$.

If all the sections $x=\pm a_l(2n+1)$, n=0, 1, 2,... are under the same strain-stress conditions, the equilibrium conditions are considered only for the section $x=a_l$. They are presented in the following form:

$$\begin{aligned}&b\int_{-h_1}^{h_1}\sigma\left(y_1\right)dy_1+N_S+N_{S_1}=N\\&b\int_{-h_1}^{h_1}\sigma\left(y_1\right)y_1dy_1+N_S\left(2y_0-h_1\right)-N_{S_1}h_1=M+N\left(y_0-h_1\right)\end{aligned} \tag{5}$$

at which the N_S and N_{S1} are the forces in tensile and compressed reinforcement in the section with the crack; b is the cross-sectional width.

After introduction of Lagrange's functions $L_{\sigma u}$ for normal load and $L_{\tau u}$ for tangential load, the expression for displacement of the rectangular section point $(1,\eta)$ with the crack, satisfying boundary conditions of the stress-strain state, is:

$$\begin{aligned}&\lambda_0y_0\int_{-1}^{1}L_{\sigma u}\left(\gamma_{lh};1,\lambda_0\eta_1-e_l,\lambda_0\xi-e_l\right)\sigma\left(\lambda_0\xi-e_l\right)d\xi+\\&+a_l\tau_c\int_{0}^{1}L_{\tau u}\left(\gamma_{lh};1,\lambda_0\eta_1-e_l,\xi\right)f\left(\xi,1\right)d\xi=c_1+c_2\left(\lambda_0\eta_1-e_l\right)\end{aligned} \tag{6}$$

in which the $\sigma(\lambda_0\xi\text{-}e_l)$ is normal stress at extension of the crack; $f(\xi,1)$ is the function of distribution of shearing stresses along the line η=1; τ_c is the maximum value of the bond stress; c_1, c_2 are normal components of point displacement (1,0) on geometrical axis of the element and normal component of point displacement (1,1) at the level of end fibre of the relative point on the axis of the element.

The system of equations (5) and (6) defines the stress-strain state of the RC element with cracks. The following re

Relationships have been used for estimation of the forces in compressed and tensile reinforcement:

$$N_{S_1} = \frac{u_{S_1}}{a_l} E_S A_{S_1} = \frac{c_1 - c_2}{\gamma_{lh} y_0} E_S 2\mu_1 b y_0 \,,\; N_S = 2\frac{c_1 + c_2}{\gamma_{lh} y_0} E_S \mu b y_0 + \frac{\gamma_{lh} b y_0 \tau_c}{3} \,,$$

in which u_{S1} is total deformation of the fibre $\eta=\eta_1$=-1 in case the protection layer has not been taken into account; μ_1 is the coefficient of reinforcing of the compressed reinforcement; E_S is the elasticity modulus of the reinforcement; A_{S1} is the cross-sectional area of the compressed reinforcement.

The distribution of normal stresses on extension of the crack can be described, in accordance with linear Fracture mechanics, as a function which becomes smooth and continuous all over the interval $-h_1 \le y_1 < h_1 (-1 \le \eta_1 < 1)$ by not taking into account the point $y_1=h_1$, i.e. (η_1=1), as $\sigma\left(\lambda_0\eta_1 - e_l\right) = \varphi\left(\eta_1\right) \Big/ \sqrt{1-\eta_1}$, [10], in which $\varphi(\eta_1)$ is a smooth and continuous function limited in the [-1,1] interval and presented in the form of order $\varphi\left(\eta_1\right) = \sum\limits_{n=0}^{\infty} b_n P_n^{(-1/2,0)}\left(\eta_1\right)$ where b_n are coefficients which have been defined during the process of solving the problem, and $P_n^{(-1/2,0)}$ is the Jacobie's polynome [9].

From the system of equations formed on the basis of expressions (5) and (6), the coefficients b_0, b_1, $b_2 ... b_n$, c_1 i c_2 have been determined. The system of equations is presented in the following form:

$$\left(12\lambda_0 + \theta_\sigma \gamma_{lh}\right) b_0 + \theta_\sigma \gamma_{lh} \sum_{n=0}^{p} \left(-1\right)^n b_n + 6\sqrt{2}\frac{1+n_\mu}{\gamma_{lh}} \mu\alpha \frac{E}{y_0} c_1 + 6\sqrt{2}\frac{1+n_\mu}{\gamma_{lh}} \mu\alpha \frac{E}{y_0} c_2 = 3\sqrt{2}\frac{E}{b y_0}$$

$$\left(\frac{2\sqrt{2}}{3}\lambda_0^2 + \frac{2-\lambda_0}{3\sqrt{2}}\theta_\sigma \gamma_{lh}\right) b_0 + \left(\frac{8\sqrt{2}}{15}\lambda_0^2 - \frac{2-\lambda_0}{3\sqrt{2}}\theta_\sigma \gamma_{lh}\right) b_1 + \frac{2-\lambda_0}{3\sqrt{2}}\theta_\sigma \gamma_{lh} \sum_{n=0}^{p}\left(-1\right)^n b_n +$$

$$+2\frac{2-\lambda_0-\lambda_0\eta_\mu}{\gamma_{lh}}\mu\alpha\frac{E}{y_0}c_1 + 2\frac{2-\lambda_0+\lambda_0\eta_\mu}{\gamma_{lh}}\mu\alpha\frac{E}{y_0}c_2 = \begin{cases} \left(e_0 + 1 - \lambda_0\right) N \Big/ \left(b y_0\right), & N \ne 0 \\ M \Big/ \left(b y_0^2\right), & N = 0; \end{cases} \qquad (7)$$

$$\sum_{n=0}^{p}\left\{\frac{2\sqrt{2}}{4n+1}\left[\frac{\pi E}{2} B_{ou}^{(n)}\left(\gamma_{lh};1,\lambda_0\eta_{1j} - e_l\right) - C_n\left(\eta_{1j}\right)\right]\right\} b_n + \sum_{n=0}^{p}\left\{\left(-1\right)^n \frac{\pi\theta_\sigma}{2\sqrt{2}\lambda_0} u_\tau\left(\gamma_{lh};1,\lambda_0\eta_{1j} - e_l\right)\right\} b_n -$$

$$-\frac{\pi E}{2\lambda_0 y_0} c_1 - \frac{\pi E}{2\lambda_0 y_0}\left(\lambda_0\eta_{1j} - e_l\right) c_2 = 0,\; j = 1, 2, \ldots, p+1;\; P_{p+1}^{(-1/2,0)}\left(\eta_{1j}\right) = 0,\; e_0 = M \big/ N y_0,\; \alpha = E_S \big/ E,\; \eta_\mu = \mu_1 \big/ \mu$$

The coefficients $B_{ou}^{(n)}\left(\gamma_{lh};1,\lambda_0\eta_1 - e_l\right)$ are obtained from

$$B_{ou}^{(n)}\left(\gamma_{lh};1,\lambda_0\eta_1-e_l\right)=\frac{4n+1}{2\sqrt{2}}\sum_{k=1}^{r}A_k^{(r)}P_n^{(-1/2,0)}\left(\xi_k\right)L_{ou}\left(\gamma_{lh};1,\lambda_0\eta_1-e_l,\lambda_0\xi_k-e_l\right),$$

the values $A_k^{(r)}$ are calculated according to expression $A_k^{(r)}=\dfrac{\sqrt{2}\left(4r-1\right)^2\left(1-\xi_k^2\right)}{4r^2\left(2r-1\right)^2\left[P_{r-1}^{(-1/2,0)}\left(\xi_k\right)\right]^2}$,

where the ξ_k is the root of equation $P_{r-1}^{(-1/2,0)}\left(\xi_k\right)=0$, whereas the values of Lagrange's regular function of influences for normal displacement of rectangular area margin under the influence of normal load $L_{\sigma u}(\gamma_{lh};\ 1,\ \lambda_0\eta_1\text{-}e_l,\ \lambda_0\xi_k\text{-}e_l)$, are obtained from the tables of Pashenko and Trapeznikov [10, p86].

The coefficients $C_n(\eta_{1j})$ are obtained on the basis of the following expressions:

$C_n\left(\eta_1\right)=\dfrac{4n+1}{2\sqrt{2}}\sum\limits_{k=1}^{r}A_k^{(r)}P_n^{(-1/2,0)}\left(\xi_k\right)\ln\left|\eta_1-\xi_k\right|$, for $r\neq n$.

The coefficient θ_σ depends on the bond stress between reinforcement and concrete and it is within the limit of 1/6 to 1/3. The values of integral $u_\tau\left(\gamma_{lh};1,\eta\right)=\int\limits_0^1 L_{\tau u}\left(\gamma_{lh};1,\eta,\xi\right)f\left(\xi,1\right)d\xi$ in which $L_{\tau u}(\gamma_{lh};\ 1,\eta,\xi)$ is the Lagrange's function of influence for displacement of the rectangular area margin under the influence of tangential load, are taken from the tables of Trapeznikov presented in the paper [10, p79].

After finding the coefficients b_0, b_1, $b_2 \ldots b_n$, c_1 and c_2, the stress-strain values are determined: the stress intensity coefficient at the tip of the crack: $K_I=\sqrt{2\pi\lambda_0 y_0}\left[b_0+\sum\limits_{n=1}^{p}b_n\dfrac{\left(2n-1\right)!!}{n!2^n}\right]$; the stress in concrete in the end compressed fibre $\sigma_b=\dfrac{1}{\sqrt{2}}\sum\limits_{n=0}^{p}\left(-1\right)^n b_n$; stresses in tensile and compressed reinforcement $\sigma_S=\dfrac{\alpha E}{\gamma_{lh}y_0}\left(c_1+c_2\right)+\dfrac{\theta_\sigma\gamma_{lh}}{6\mu}\left|\sigma_b\right|$, $\sigma_{S_1}=\dfrac{\alpha E}{\gamma_{lh}y_0}\left(c_1-c_2\right)$; displacement of the crack margin at $\eta_1>1$ is

$$u\left(\eta_1\right)=\frac{2\lambda_0 y_0}{\pi E}\sum_{n=0}^{p}\left\{\frac{2\sqrt{2}}{4n+1}\left[\frac{\pi E}{2}B_{ou}^{(n)}\left(\gamma_{lh};1,\lambda_0\eta_1-e_l\right)-C_n\left(\eta_1\right)\right]\right\}b_n+$$

$$+\frac{2\lambda_0 y_0}{\pi E}\sum_{n=0}^{p}\left\{\left(-1\right)^n\frac{\pi\theta_\sigma\gamma_{lh}}{2\sqrt{2}\lambda_0}u_\tau\left(\gamma_{lh};1,\lambda_0\eta_1-e_l\right)\right\}b_n,\quad 1<\eta_1\le\left(2/\lambda_0\right)-1;$$

the corresponding value of the crack width at $\eta_1>1$ will be $a_{crc}\left(\eta_1\right)=2\left[c_1+c_2\left(\lambda_0\eta_1-e_l\right)-u\left(\eta_1\right)\right]$. Any parameter X of the stress-strain state of an element can be obtained by superposition of the solution $X=\left(M/by_0^2\right)X_M+\left(N/by_0\right)X_N$, where X_M and X_N are the values of this parameter at $M/(by_0^2)=1$, $N=0$ i $N/(by_0)=1$, $M=0$.

2.2. The example of calculation of the crack width

To illustrate the method of calculation of the crack width in case of pure bending N=0 and M=0, the results of example given in [10] have been presented. Using the system of equations (7), the analysis has been performed for the values of M=1, b=y_0=1, E=1, the solutions of which give the arbitrary values M, b, y_0, and E which are used to obtain the relevant coefficients and parameters of stress-strain condition of the element.

The mode of the crack in case of lack of bonding (θ_σ =0, the line marked by 1 in *Fig 5*) and in case of good bonding of the reinforcement and concrete (θ_σ =1/3, the line marked by 2 in *Fig 5*) during the service state, at pure bending (N=0, M≠0) of RC beams for $h_{crc}/(2y_0)$=0.6, (≥0.1), $\gamma_{lh}=a_l/y_0$=2, and the coefficient of reinforcement of $\mu\alpha$=0.1, has been presented in *Fig 5*.

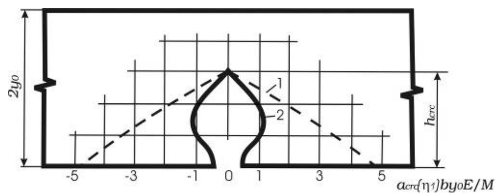

Fig 5. Graph of the function $a_{crc}(\eta_1)by_0E/M$ for γ_{lh}=2

The mode and size of the crack opening has been shown in *Fig 5*, which significantly depend on the bonding coefficient θ_σ. If there is no bonding, the largest opening of the crack occurs at the site of tensile section margin, whereas the width of the crack has been gradually decreased towards the tip of the crack (line 1 in *Fig 5*.) In case of undisturbed bonding, the crack 's maximum width of the opening occurs above the tensile margin at the corresponding distance (line 2 in *Fig 5*). Comparing the results of the cases 1 and 2 line in *Fig 5*, it can be noted that the size of crack opening is 5 times lesser relative to the case with no bonding. Therefore, this fact has to be taken into account in structural elements, in order to avoid this negative phenomenon. The example for this is the formation of elements exposed to tensile forces (tie-beams). In order to avoid the problems of large opening cracks formation, in construction of the RC tie-beams, a two-phase concreting has to be applied [11]. The concrete work in the first phase comprises concreting with no protection layer. When the first phase concrete gets hardened, the tie is tensioned by maximum force, after which the second phase concrete work follows, i.e. the protection layer has been formed which presents with no cracks due to the tensile force.

With more dense distribution of cracks the value of the bonding coefficient has no essential influence on the shape and absolute value of the crack openings (*Fig 6*).

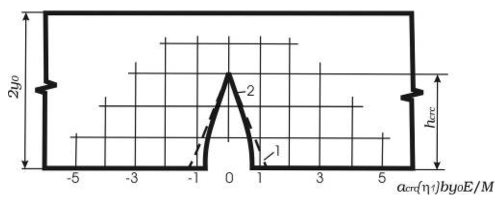

Fig 6. Graph of the function $a_{crc}(\eta_1)by_0E/M$ for γ_{lh}=2/3

The bonding coefficient value does not essentially influence on maximum distance between the margins a_{crc} distributed along the tensile area, at the low ratio of distances between the cracks and height of the girders ($\gamma_{l/h}\leq 1/2$). *Fig 6* shows that the maximum opening of the crack for θ_σ=0, goes beyond the corresponding value of the crack opening at θ_σ=1/3, only for 30%.

3. CONCLUSION

Further investigations are needed for developing the methods of designing, which certainly comprise the analysis of the shape of fracture, capacity of ductility and broader application of Fracture mechanics which has not yet been adequately denoted in concern to its significance. Its application is of particular significance in analysis of the crack development and changes of local condition of the stress, for higher phases of load at which the changes in behaviour of structures are more explicit [7]. Current studies, here presented are the investigations of American Concrete Institute (ACI), create conditions for its broader application in practice.

This study showed that it is necessary to supply complete bonding between the reinforcement and concrete in order to restrain the crack width in RC element. This is of particular significance in case of remote cracks. The deficiency of presented calculation lies in the complexity of solving the problem which imposes the use of PC.

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AN ANALYSE OF THE NECESSITY AND THE POSSIBILITIES CONCERNING THE THERMAL RETROFITTING OF THE EXISTING BUILDINGS

Summary: *The hygrothermal rehabilitation of existing older buildings, particularly of those erected in the last few decades, is a real necessity nowadays in our country, confronted with very serious comfort and hygiene problems, in the current circumstances of energy crisis.*

In order to solve this important problem, now there are numerous preoccupations concerning the best hygrothermal protection and retrofitting of existing dwellings; a rich technical and economic legislation in this field, elaborated in the last few years, represents an important support of these kind of technical interventions.

But, the great number of the sick old buildings, as well as the high costs of the general retrofiting, in comparison with the low financial possibilities of the owners, require a rational unfolding of hygrothermal rehabilitations, begining with the most important dwellings and the most damaged closing elements.

This paper presents some reasons concerning the necessity of the thermal rehabilitation of existing blocks of flats, having in view the economic circomstances existing now in our country.

Key words: *dwellings, comfort, energy, thermal rehabilitation, retrofitting, legislation, solutions, owners, costs.*

ANALIZA MOGUĆNOSTI I POTREBA VEZANIH ZA TERMIČKO POJAČAVANJE POSTOJEĆIH ZGRADA

Rezime: *Higrotermička rehabilitacija postojećih starijih zgrada, posebno onih podignutih tokom nekoliko poslednjih dekada, predstavlja stvarnu potrebu danas u suprotnosti sa veoma ozbiljnim problemom komfora i higijenskih problema u okolnostima kriza. Da bi se rešio ovaj veoma važan problem, danas postoje brojne preokupacije vezane za bolju higrotermičku zaštitu i pojačanje postojećih zgrada; bogata tehnička i ekonomska zakonska regulativa u ovom polju, razrađena u toku nekoliko poslednjih godina, predstavlja važnu podršku ove vrste tehničkih intervencija. U ovom radu su prikazani neki od razloga vezanih za neophodnost termičke rehabilitacije postojećih stambenih blokova, imajući u vidu ekonomske okolnosti u Rumuniji.*

Key words: *boravište, komfor, energija, termička rehabilitacija, osposobljavanje, regulativa, rešenja, koštanje.*

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1. THE MODERNIZATION OF EXISTING BUILDINGS – AN URGENT NECESSITY FOR OUR COUNTRY, ROMANIA

Important bases of buildings, with different destinations – but mainly for dwelling, are in service now in our country. Thus, in 1997 year the number of existing dwellings was about 8 millions. From these, over 3 millions are placed in towns, in bocks of flats.

These buildings have been constructed in different periods, so that nowadays there are in use are in use many old buildings, dating before 1950, and new buildings, made in the last years. The main part is represented by the buildings made between 1960…1980.

According to levels of exigency imposed by the regulations and technical as well as economical possibilities from the construction period, these buildings presents specific aspects regarding the functional layout, structural conception, envelope elements, finishing, installation, utilities, etc.

It is evident that according as the buildings are older, their qualities are lower, and their performances are more different than the actual exigencies regarding the functional, reliability, hygiene, comfort and other technical requirements.

In this critical situation there are also more recent buildings made of large reinforced concrete prefabricate panels with flat roofs, that presents important lacks in comparison with the actual functional requirements, due to the quality of the construction materials used, negligence in the treatments of the joints, normative requirements situated at the lower limits of the rational values, imposed by a erroneous economic politics etc.

To all these deficiencies, that can explain largely the unconformity between the actual requirements and low functional performances of the existing buildings, must be added the harsh climatic conditions characteristic for our country and some inadequate utilization factors, having important negative influence over the constructive elements that provides the hygrothermal protection of the building.

The low functional performances of a large number of buildings rise acutely the necessity of retrofitting and upgrading by constructive actions regarding the functional, mechanical, thermophysical, aesthetic aspects of the building, made according to superior quality requirements practiced practised at present.

In that concern the building modernization, in order to fulfil the comfort and health requirements, the emergency is evident in the actual circumstances of energetic crisis, environmental protection needs, users' higher requests and economic difficulties.

The need for the retrofitting of the existing building is important for all types of buildings, but especially for dwellings and usually civil buildings, designated for the daily use of the population on a longer periods of time than other buildings. The actual regulation elaborated in the last years in our country, stipulate specially the necessity of getting this workmanship (O.G. 29/2000).

To ensure the optimal quality of services, according to the requirements of the modern life and in maximal security, represents a major objective for all the existing buildings, no meter how old there are. This can be achieved just with adequate interventions based on efficient and rational technical constructive solutions, that could eliminate the differences between the old buildings and the new ones, made according to the high exigency requests.

2. PRIORITY IN RETROFITTING ARE THE INADEQUATE BUILDINGS

The civil buildings are preponderant in the existing range of buildings from our country. Among these, the dwellings are the majority and their number expands due to their social importance.

The large diversity of types, structural solutions, envelope and dividing systems that characterize these buildings, together with the differences of their maintenance and usage, make practically impossible to propose a general technical retrofitting solution that can be applied in all the cases. However can be recommended some general principles that could permit the elaboration of specific details for particular projects.

Due to the dimensions of these civil buildings and the large surfaces of glassed elements, imposed by natural illumination requirements, significant surfaces of the envelope presents reduced thermal insulating capacities. This is another disadvantage of this type of building in comparison with other buildings with few openings.

Must be noticed that while in the developed countries the increase of the thermal protection of the building is an usual achievement, that determine the reduction of the specific heat consumptions with 30…50%, in our country these procedures have been made occasionally especially in order to test the efficiency of some new solutions, to be generalised. However in the last 5…6 years have been retrofitted from hygrothermal aspect some civil buildings, especially students hostels and blocks of flats.

At the same time, the seismic protection, very important for our country, imposes that each hygrothermal retrofitting be correlated with a mechanical reconsolidation and conversely. This condition determine some particular technical solutions of retrofitting, specific to our country, in comparison with those currently used in other countries.

3. THE REHABILITATION OF THE BUILDINGS – STRINGENT NECESSITY FOR OUR COUNTRY

All over the world a great part of the energy is used for consumption in buildings, respective for heating, cooling, hot water, artificial light, home devices etc. In the European Union this consumption represent about 40% from the total primary energy consumption. In Romania, according to 1998 Statistics yearbook, the primary energy consumption in household field represents about 33% from total. This is explained by:

- reduced level of comfort in numerous buildings,
- the existence of great consumer in industries,
- the omission of the statistics of heat resulted from the burn of wood.

The importance of energy conservation measures in the building area is obvious being determined mainly of:

- major economic interests (remission of fuel imports, diminution of the maintenance costs of lodgements etc.);
- the improvement of the life conditions in buildings (hygiene, comfort, health);
- environmental protection by the reduction of the carbon dioxide emissions, the main reason for the greenhouse effect;
- natural resource saving (fissile fuel, wood etc.).

For the existing buildings the energy conservation supposes the necessity of hygrothermal retrofitting, in order to reduce the heat losses in the cold season.

The main component of the hygrothermal retrofitting is the thermal rehabilitation, with the aim to improve the thermal resistance of the closing elements of the building.

Concerning the main causes that impose the thermal retrofitting of the envelope elements after a period of service can be enumerate as follows:

a. The diminution of the thermal insulating qualities of the materials, due to the cyclic influence of outdoor and indoor actions.

Between the climatic actions, most dangerous are: • variations of the temperature, • periodic frozen, • infiltrations of meteoric water, • solar radiations.

Between the indoor agents, the major damages of the thermal insulations are produced due to the water vapour condensation in the structure of the materials. The water accumulation determine the increment of the thermal conductivity and associated with the low temperature and recurrent frozen can produce cracks in the material.

b. The increase of the thermal exigencies regarding the thermal comfort, especially the values of the indoor temperature due to the modification of life habitudes, lodgers' age evolution etc.

According to this aspect, the indoor temperature of +20 °C, used in heat engineering and hygrothermal regulations from our country for dwellings and similar buildings (STAS 1907; STAS 6472), tends to grow to the value of +22 °C or using the value T_i = +20 °C as a resultant temperature (T_r), considering the radiant temperature of the perimetric elements of the rooms.

Also, the admissible temperature difference ($\Delta T_{i,\ max}$) between the indoor air and the internal face of the envelope elements who was $\Delta T_{i,max}$ = 6 °C according STAS 6472/73 regulation become $\Delta T_{i,\ max}$ = 4,5 °C after 1989 regulation.

c. The increase of the exigency level regarding the thermal insulating capacity of the envelope elements became acute in the last few years due to the global energetic crisis and the interest for the pollution reduction.

In the developed countries, due to the tendency of energy consumption reduction the envelope elements must present a very high thermal resistance. For example, in U.E. the standardized specific resistances are about 2,20…2,86 m^2K/W for walls and about 3,33…4,00 m^2K/W for the different types of roofs.

In Romania the main part of existent buildings have been constructed between 1960…1980 according to very low requirements regarding the value of the thermal resistance of the envelope elements. Thus, the STAS 6472 regulations imposed in 1960-1970 the values of 0,67…0,83 m^2K/W for walls and 0,89…1,25 m^2K/W for flat-roofs.

Just after 1984, beginning with NP15 regulations, the R_0 values imposed by energy economy reasons was 1,16…1,25 m^2K/W for walls and 1,46…1,63 m^2K/W for roofs and after 1998 become 1,40 m^2K/W for walls and 3,00 m^2K/W for terrace roofs.

The tendency of providing comfort conditions with reduced energy consumption will determine the required resistance to grow up to the values used in UE. Therefore, are expected values of 2,5…3 m^2K/W for the massive part of the wall and 3,5…5 m^2K/W for terrace roofs.

d. The necessity of rehabilitation of an existing building based on other motivations (mechanic, functional, architectural) can be a good opportunity to make the thermal retrofitting of the building together with the other interventions.

Taking into account the economical particularities of the actual period in our country, respectively the reduced economic power of the apartment owners it is preferable a step by step strategy of retrofitting with a scale of priorities according the importance of the building, the degree of deterioration, costs, etc. The following steps are proposed:

- the thermal retrofitting of the depreciated or low insulated terrace roofs;

- the thermal improvement of the windows and other glassed elements;
- the increase of the thermal insulating capacity of the external walls;
- the supplementary thermal insulation of the building base.

For each rehabilitation step considered, the thermal proposed protection level must be at least equals to that recommended by the last regulation in use. It is also rational to accompany the thermal protection with the improvement of the heating installations.

The main technical solutions of thermal retrofitting consists in a supplementary thermal insulating layers placed on the opaque surfaces of the walls, associated with captive air layers between the glasses, or to use thermal efficient glasses.

Proceeding in this manner, we consider that after approximately 10…15 years, the main part of the existent building could provide a good level of thermal protection.

4. PRIORITIES IN THE RETROFITTING OF THE BLOCKS OF FLATS

The number of buildings with problems of insulation and comfort is very important, thus a national action of thermal retrofitting is not possible on large scale in present or in the next future, in our country, without an external economic aid.

Because the reduction of the energy losses and the increase of the comfort conditions must be as near as possible to the actual requirements, it is important to make at list a partial rehabilitation, beginning with the most important constructive elements with predominance in the envelope surface.

According to the scale of importance presented above, the building roofs are situated on the first place considering the importance of these construction part, their reduced hygrothermal performances and the technical facilities offered in case of general retrofitting.

Another argument for the high necessity of roof retrofitting is the advanced stage of deterioration due to pluvial water infiltration, the reduced degree of thermal insulation that can encourage the water condensation.

For the thermal retrofitting of the terrace roofs the flowing steps are rational:

- the assessment of the actual deteriorations and their cause;
- the selection of the adequate thermal insulation materials;
- to identify the most efficient system of retrofitting;
- the disproof of vapour condensation and accumulation in the structure.

Considering the dynamics of roof deterioration results the following principles of repair by retrofitting:

- the increase of the thermal resistance of the whole roof system;
- to impede the of water vapour condensation in the roof structure;
- possibility for the water vapour elimination;
- to provide a good and durable waterproof membrane after retrofitting.

The principles mentioned above must be concretized using accessible means, like:

- for the supplementary thermal insulation must use quality materials;
- to stop the water valour infiltration by vapour barier;
- the water vapour evacuation by special means.

5. ABOUT THE ACTUAL POSSIBILITIES OF THERMAL RETROFITTING

Nowadays we have a good experience regarding the evaluation of the real thermophysical performances of the existing buildings. In this direction a lot of theoretical and experimental studies have been made in the Department of Civil Engineering of Iasi.

The experience on this field of building retrofitting from other countries action adapted to our conditions and also from some works made in our country supported by new and efficient materials and technologies make possible to solve this problem.

Also there are numerous technical regulations and standards on this field, that offer the adequate legislation to sustain the actions of thermal retrofitting of the buildings.

The online impediment consists in the reduced economic power of the apartment owners and the absence of any other financial support in this direction. But any action is better than no action because the existent deteriorations tend to grow and must be stopped.

6. CONCLUSIONS

The modernisation by thermophysical retrofitting of the existing buildings is very important, because regards directly the owners of the buildings. This aspect must be considered together and with the same importance as the mechanical rehabilitation of the existing buildings.

In the action of thermophysical retrofitting of the old buildings it is important to begin with buildings of first degree of priority, as blocks of flats used for long periods by great numbers of people. In this buildings the order of priority is the building roof, the glassed elements and the external walls, as well as the installations.

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TENTH NATIONAL AND
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OBSERVATIONS AFTER THE HYGROTHERMAL REHABILITATION OF EXISTING BUILDINGS

Summary: *The hygrothermal rehabilitation of existing old buildings, specially of those erected in the last few decade, is a real necessity in our country, confronted nowadays with economic and energy problems. In order to solve this very important requirement, in Romania there are now numerous preoccupations concerning the hygrothermal protection and retrofitting, with some remarkable results. One of the most recent is the rehabilitation, with supplementary thermal insulation layer, of a large panel hostel, high energy consumer in winter, selected as a model between the lots of such buildings existing in Iasi and in all country. But, after the retrofitting, with favorable effects on the comfort and the energy consumption, some shortcomings have been pointed out by the users and confirmed by measurements. This paper presents the most important such aspects, inadequate for the normal conditions, in order to avoid them in other similar works who will be used for the future modernization of the old buildings.*

Key words: *buildings, heating, comfort, hygiene, energy, consumption, rehabilitation, modernization, effects, shortcomings, conditions.*

OPSERVACIJE NAKON HIGROTERMIČKE REHABILITACIJE POSTOJEĆIH ZGRADA

Rezime: *Higrotermička rehabilitacija postojećih starijih zgrada, posebno onih podignutih tokom nekoliko poslednjih dekada, predstavlja stvarnu potrebu danas u suprotnosti sa problemom utroška energije i komfora. Da bi se rešio ovaj veoma važan problem, u Rumuniji danas postoje brojne preokupacije vezane za bolju higrotermičku zaštitu uz neke značajne rezultate. Jedan od poslednjih je rehabilitacija sa dodatnom termičkom izolacijom velikih panela. Međutim, nakon pojačavanja, pored mnogobrojnih povoljnih efekata sa stanovišta komfora i energije, nekoliko nedostataka je uočeno od strane korisnika koja su potvrđena merenjem. U radu su prikazani najvažniji takvi aspekti, koji su nepodesni u normalnim okolnostima, da bi se izbegli u budućoj modernizaciji starih zgrada.*

Ključne reči: *zgrade, grejanje, komfor, higijena, energija, konzumiranje, rehabilitacija, modernizacija, efekti, nedostaci, uslovi.*

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1. GENERAL CONSIDERATIONS

The buildings are long life and high value objects, destined for more generation of users, so that some radical interventions are possible only in special situations, like restoration after earthquake, or with a view to fully modernization. One of the most important reason for such interventions is the thermal improvement, necessary having in view the decrease of thermal protection and of the efficiency of existing installations.

In order to ensure the normal hygiene and comfort conditions within the rooms and, at the same time, to stop the energy losses during the exploitation of existing buildings, in the last few years the interest for the researches regarding these aspects had taken a high and justified scope, taking into account the very important implications of this problem on the life quality of the lodgers and of the whole society.

A research group of our Department has participated at the establishment of adequate technical measures for the hygrothermal rehabilitation of a hostel T14, selected like representative from among the five-floor hostels existing in Iasi.

The structural system of this building, characteristic for the majority of living buildings erected in the 60…80-is years in our country, is made of large reinforced concrete prefab panels and covered with flat-roof.

2. PROPOSED AND APPLIED REHABILITATION SOLUTIONS

In principle, the thermal improvement of the hostel consist of increasing the thermal isolation of existing envelope components, using: supplementary thermal insulation layers for the blind portions, like walls and roofs, and supplementary simple new window with 3 mm glass pane, as well as simple glazed bow windows (Fig.1).

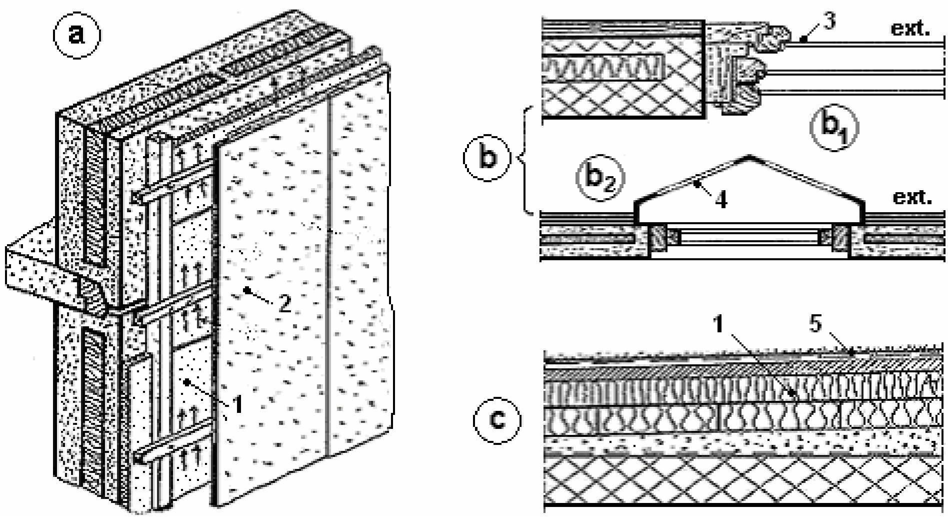

Figure 1 – The hygrothermal rehabilitation systems adopted to improve the student hostel T14: ***a. - External walls****: additional hermal insulation layer of polystyrene (1), protected by Polyalpan system (2);* ***b. - Windows****: b_1 - additional simple window (3), b_2 - glazed aluminium bow window (4);* ***c. Flat-roof****: supplementary polystyrene layer (1), protected with new water-proof membrane (5).*

3. MEASUREMENTS OF MICROCLIMATE PARAMETERS

The mentioned building has been tested in point of view of inside hygrothermal conditions, by means of measurements of microclimate parameters, in cold winter periods, both before and after the rehabilitation works.

The furnished energy for heating have been maintained approximately the same in both situations during the measurements period, in order to find out the indoor climatic differences, therefore the benefits of thermal rehabilitation given the initial situation.

The results of these measurements, presented in the Table 1, point out the positive influence of the thermal rehabilitation on the indoor air and surfaces temperature, respectively on the thermal comfort, what have permitted ulterior a significant reduction of necessary heat consumption, therefore some important economies in exploitation.

Nr.	Microclimate parameter	Before rehabilitation	After rehabilitation	
		Current room with double windows	Current room with treble windows	Current room with double windows + bow window
1.	Outdoor air temperature T_e (°C)	- 8.0…+ 9.2	-15,1…+ 10.4	
2.	Indoor air temperature T_i (°C)	17.3…21.1	27…29.5	27.5…30.0
3.	Surface temperature of external wall - T_{si} (°C)	12.8…15.5	23.3…23.9	24.2…24.5
4.	Outdoor air humidity - φ_i (%)	44.0…73.0	30.0 … 90.0	
5.	Indoor air humidity - φ_e (%)	47.0…81.0	14.0…47.0	17.0…42.0
6.	Indoor air velocity - v_i (m/s)	0.05…0.32	0.0…0.035	0.0…0.021

Table 1. Effective values of hygrothermal parameters measured in wintertime in T14 student hostel, before and after the rehabilitation works

4. FAVOURABLE EFFECTS OF THE THERMAL REHABILITATION

The results of the effective measurements, as well as the findings in exploitation conditions, prove some very important benefits of the applied hygrothermal rehabilitation solutions on the indoor comfort and hygiene conditions, namely:

- the increase of indoor air temperature within all the rooms with 7…12 °C, what is an essential gain, because the normal comfort temperature values can be ensured with a significant cut of furnished energy for heating (about with 30…40%);

- the increase up to double the temperatures on the inside surface of external walls, having as result in the normal exploitation conditions the total avoidance of the condensation, as well as of negative comfort and health effects;
- an important diminution of indoor air relative humidity (3…5 times less), very important also for the avoidance of condensation phenomenon, having at the same time some positive implications on the health of the lodgers.

The adopted solutions for thermal improvement of envelope component elements of existing student hostel allowed some important favourable effects on the room comfort and hygiene conditions, established in the new situation, after the thermal rehabilitation operations, as demonstrate the effective measured values of inside microclimate parameters, as well as the subjective opinions of the lodgers of T14 hostel, generelly satisfied with the new conditions, like:

- The better, healthier microclimate is contributing to rise the students working capacity, and to their new attitude concerning the better built environment, with less energy consumption in exploitation.
- The thermal rehabilitation works presents, at the same time, an important diminution of polluant emissions due to the necessary heating with classic methods.
- The new treatment of external walls, including the glazed bow windows, present a very favourable architectural effect, comparatively with the initial monoton appearance.

These findings constitute important arguments to extend, if is possible on large scale in future, the proposed retrofitting technical modalities, as well as other such solutions, for the hygrothermal improvement of existing old buildings in our country.

The conclusions recommend the application of the hygrothermal rehabilitation for all the existing old constructions, specially for the dwellings in blocks of flats.

5. SOME NEGATIVE ASPECTS FOUNDED AFTER REHABILITATION

Though the results after these hygrothermal retrofitting works have indisputable real favorable effects on the comfort, hygiene and health conditions, as well as regarding the rational energy consumption in exploitation, some shortcomings have been pointed out by users, and confirmed by means of some ulterior tests and observations.

Because the analyzed hostel is modernized enough recently, the surveillance will be continued, in order to find all the bad situations who can occur in time, with a view to be possible afterwards the elaboration of most rational and efficient systems.

That's why in this paper we present only the most important preliminary such aspects, who have been useful for the ulterior rehabilitation of other two student hostels:

- The main negative effect pointed out after the functioning resumption of improved hostel is the very important decrease of air change rate of rooms by natural ventilation, from the higt rate $n = 1.2…1.8\ h^{-1}$, corresponding to the initial situation, to the very low air change rate $n = 0.25…0.6\ h^{-1}$.

This is the consequence of thermal improvement interventions on the windows, who have diminished the air permeability of untight joints between the moving and fixed components of these old windows, who constitute the usual way for the air circulation, without to ensure in stead some adequate ventilation devices.

As a result, the fast pithy diminution of air quality in rooms necessitate frequent and prolonged openings of new windows, which bring about important heat losses and the cooling of indoor air, or to accept the unhealthy conditions in rooms.

▪ An other negative effect, owing to the same cause, is the very feeble air movement in rooms, only with v = 1…2 cm/s (Table 1), inferior given the normal air speed necessary for the normal comfort sensation and for health; the total air stagnation is a frequent situation, both in the corners and in the rest of lot of rooms, what determine thus abnormal hygiene and health conditions.

▪ Particularly in the rooms with supplementary new bow windows the anterior mentioned effects are more increased, and the possibility to open the windows is too hard because of absence of adequate opening accessories in these elements.

▪ Inside the bow windows the air temperature is with 8…10 °C higher than the outdoor temperature, and with 10…18 °C less than the room temperature, what determine the hard cooling of external glass pane of bow window, favorable for the abundant condensation of vapor penetrated in this space out of the room, as well as the frost of produced water, embarrassing the normal visual comfort

▪ The natural lighting of rooms is considerably affected because of decrease with 3…5% of effective glazed surface of windows, as a result of new frames and jambs necessary for the supplementary windows, broader than the existing wooden elements; in the case of bow windows this effect is higher, what necessitate supplementary electrical lighting, therefore additional energy consumption.

▪ The vertical wooden laths break off the thermal isolation layer, who determine a thermal bridge effect, with possible hygrothermal shortcomings in time course, as: plus thermal losses, risk of condensation, moistening and to rot of lathes etc.

▪ The air circulation in the special space reserved in this aim behind of protection panels is practically non-existent, even during the wind action, because the ventilation holes practiced down and on upper part of panels are probably too little, and the hydraulic resistance in the air space, among the lathes is too high.

▪ Owing to the wind action, as well as to the temperature differences, who determine dimensional variations of protection panels, some noises are generated as well as the enlargement of joins of panels, favorable for rain water penetration.

▪ The costs of this kind of thermal rehabilitation is still high for the financial possibilities of lodgers in the actual period, the elaboration of more inexpensive solutions being necessary with a view to can generalize the thermal rehabilitation.

6. IMPROVEMENT OF REHABILITATION SOLUTIONS

In order to remove the presented shortcomings of applied hygrothermal rehabilitation works, and to make more performant the proposed solutions for other such operations, the next improvement solutions are necessary:

▪ To ensure the normal room ventilation, by means of adequate devices, or by means of electric automated fans, mounted in windows or in the external walls.

▪ To reduce the opaque portions of supplementary windows or bow windows, using new, modern glazed elements.

▪ To ensure the continuity of isolation layer, fixing the lathes over this layer.

▪ To apply more inexpensive protections panels, covered with nonmetallic sheets.

▪ To adopt a more efficient system to ensure a feeble air circulation inside the space between the panels and the isolation, necessary to eliminate the vapor.

Applying these measures by means of adequate technical solutions, is possible to obtain more efficient results with the hygrothermal rehabilitation of existing buildings, and more accessible in point of the prices, for the majority of lodgers of these houses.

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TENTH NATIONAL AND
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IMPORTANCE OF GLAZED ELEMENTS FOR THE HYGROTHERMAL REHABILITATION OF THE BUILDINGS

Summary: *Owing to the actual world-wide energy crisis, the heat conservation in buildings is a very important problem. That is why the thermal improvement of existing buildings is subject of numerous studies concerning especially the external walls and roofs, as well as, in a less measure, the glazed elements, even their surfaces in the external walls is enough important and their thermal qualities are very low. Due to their specific structure, these elements cannot be rehabilitated in the same manner like the others: the supplementary thermal insulation layer is inadequate, and a radical thermal improvement of glazed elements in walls is too difficult and enough expensive for the majority of the owners of existing old houses. Some simple and economic solutions with this aim as regard the windows equipping the most habitual blocks of flats in our country, are presented in this paper, based on some preoccupations of the author in this domain. Also, some of these solutions can be applied easily in the existing buildings, even by the lodgers, with low material consumption and costs, but having remarkable results for the interior comfort in winter.*
Key words: *buildings, heat losses, windows, rehabilitation, solutions, inexpensive, lodgers, comfort.*

VAŽNOST GLAZIRANANIH ELEMENATA ZA HIGROTERMIČKU REHABILITACIJU ZGRADA

Summary: *Zahvaljujući aktuelnoj svetskoj energetskoj krizi, održavanje energije kod zgrada je veoma važan problem. To je razlog što je poboljšanje termičkih osobina postojećih zgrada predmet brojnih studija vezanih posebno za spoljašnje zidove i krovove. U manjoj meri studije se odnose i na glazirane elemente pošto je njihovpoložaj u spoljašnjim zidovima dovoljno važan, a termička svojstva veoma niska. Zbog specifične strukture ovi elementi ne mogu biti rehabilitovani na isti način kao ostali: dodatni slojevi termičke izolacije su neadekvatni, a radikalno poboljšanje glaziranih elemenata u zidovima je isuviše teško i previše skupo za većinu vlasnika starih kuća. Neka jeftina i ekonomična rešenja koja se odnose na prozore koji se najviše primenjuju u stambenoj gradnji u našoj zemlji, zasnovana na autorovim preokupacijama u ovom području, prikazanu su. Neka od tih rešenja mogu biti veoma lako primenjena kod postojećih zgrada, pri čemu se dobijaju značajni rezultati sa malim utroškom materijala i niskom cenom.*
Key words: *zgrade, toplotni gubici, prozori, rehabilitacija, rešenja, jeftina, vile, komfor.*

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1. ABOUT THE WINDOWS OF EXISTING BUILDINGS

The general thermal balance of the habitual buildings point out that the heat losses through the glazed elements in external walls, like windows and doors, represent about 20 % of the overall heat losses, what is very much taking into account that theirs total surface not exceeds habitually 15...20 % of the envelope surface, and that 50 % about of the heat transferred in exterior is due to the ventilation.

This situation is caused by the specific structure of these elements, having large transparent portions and with untight joins between the mobile and other elements.

Therefore, the glazed elements of external walls constitute the most feeble zones of thermal point of view, so that the preoccupations for theirs improvement are alike justified and necessary, as part of general thermal rehabilitation of the old buildings, or separately, for diminish the energy consumption of the rooms.

Numerous buildings in our country, especially blocks of flats erected in the 1950...1980 period, have now some hygrothermal and comfort problems due to the long cyclical action of environmental physical factors, as well as of indoor bad exploitation, what necessitate either a general hygrothermal rehabilitation, or at least some partial interventions, like the improvements and repairs, first of all for the glazed elements.

Owing to theirs positions, as well as the specific of their use, the windows are the most exposed to damages. That is why, after a long utilization, the windows present some deformations and wears, who determine the increase of air tightness of joints.

This situation can be found frequently in the case of wood-windows, equipping the habitual blocks. The supplementary air change determined by the enlargement of the joints reach currently at twice and more given the normal, especially in the windy days. As a result, in very cold periods, with the existing heating conditions, the lodgers must to supplement the energy consumption for heating, using also other sources, like the electric or gas heating, or to plug the joins of windows, often excessively.

The usual double wood-windows of the existing buildings are equipped with 3 mm thickness ordinary glass panes, at 3…12 cm distance between. Their current thermal transmitance is 2,5 W/m^{2}K about, therefore the resistance is R_w = 0,4 m^{2}K/W, at least three times less than the resistance of the opaque portions of existing external walls.

The high heat losses through these zones are intensely felt on the rooms conditions, so that the lodgers are compelled to consume supplementary energy for heating, or to use some improvisations in order to preserve the available heat.

All these deficiencies justify the necessity of the hygrothermal rehabilitation of windows, if possible using the most efficient technical solutions.

2. HEAT TRANSFER THROUGH THE WINDOWS

Unlike the opaque portions of external walls, the heat transfer through the glazed elements ($Q_{w,}$) take place both by direct transmission ($Q_{w,t}$), as well as by air change through the joints among the component elements ($Q_{w,a}$), therefore:

$$Q_W = Q_{W,t} + Q_{W,a} \tag{1}$$

The explicit form of this equality is obtained taking into account of (S_g) surface of the glazed portions and (S_o) of the opaque elements (frames, jambs), of theirs thermal transmitances (K_g and K_o), as well as the air flow through the joints (D_a) and the temperature difference between the inside and outside air (T_i and T_e), therefore:

$$Q_W = \left[\left(K_g \cdot S_g + K_0 \cdot S_0\right) + c_a \cdot D_a\right] \cdot \left(T_i - T_e\right) \tag{2}$$

Results that the decrease of heat flow through the glazed elements of walls is possible by some interventions, just on the thermal transmitances and on the air flow. Because the surfaces of the windows opaque elements, also theirs thermal improvement are reduced, in order to limit the heat losses remain to be analyzed: the transmitance of the glassy area (K_g), and the air flow (D_a).

The thermal transmitance can be written:

$$K_g = \frac{1}{R_g} = \left(\frac{1}{\alpha_i} + \sum\frac{d_g}{\lambda_g} + R_a + \frac{1}{\alpha_e}\right)^{-1} \tag{3}$$

where:

- R_g **-** is the oerall thermal resistance of glazed zones (m^2K/W);
- α_i and α_e **-** the surface thermal coefficients of windows (W/m^2K);
- d_g **-** the thickness of window panes (m);
- λ_g **-** the thermal conductivity of the glass (W/mK);
- R_a **-** the thermal resistance of air closed between the panes (m^2K/W).

The air flow (D_a) can be expressed in accordance with the minimum flow necessary by hygiene conditions (D_{min}), and the air surplus (ΔD_a), who determines useless heat flow, respectively:

$$D_a = D_{min} + \Delta D_a ,$$

therefore:

$$Q_a = c_a \cdot D_a \cdot \Delta T + c_a \cdot \Delta D_a \cdot \Delta T \tag{4}$$

3. POSSIBILITIES TO REDUCE THE HEAT LOSSES OF THE WINDOWS

Analysing the previous relations of the heat transfer through the glazed elements, results that in the case of habitual buildings, ventilated naturally, the intervention possibilities for modify this heat transfe are reduced, only in two directions: **1-** modifying the thermal transmitance of glassy portions; **2-** to act on the supplementary air change through the joints between the components of the windows.

a. To improve the thermal transmitance of windows they are some possibilities:

1 **-** using special window panes with high thermal capacity;

2 **-** modifying the thickness of habitual panes on existing frames;

3 **-** modifying the thermal resistance of the air layers.

The first variant consist to substitute the ordinary panes with high thermal quality window-panes (thermopan). Because of high price, this solution is now still prohibitory to use on large scale for habitual buildings.

In the second case, the obtained results are unimportant, owing to the high thermal conductivity of the glass; therefore, the thick glass panes can not constitute a valid solution for such improvement.

The third possibility is based on the thermal insulation properties of the thin air blades closed between the glases. But, having in view that the thermal resistance of these is maximum $R_a = 0{,}18\ m^2\ K/W$ for 5...7 cm of thickness, so that the increase of distance between panes beyond these values is useless.

Taking into account that the very thin air blades (d = 1...3 cm) present the thermal resistance near to the maximum ($R_a = 0{,}14...0{,}16\ m^2K/W$), result the possibility to improve the thermal isolation capacity of the habitual glassy zones multiplying the number of window panes, consequently the closed air blades, practically using more thin panes on the same frames of glazed elements.

b. The supplementary air change of rooms is favorable from the hygiene point of view, but is undesirable energetically. Because the most of buildings are ventilated naturally, by air change through the joints, the minimum value (D_{min}) must be realized increasing the windows air-tightness.

It must to remark that the control of windows air permeability is very difficult, so that the air flow limitation by usual means is uncertain. Therefore we recommend to close up all the joints, if possible, using afterwards fans or transom windows to obtain the necessary air change, who can ensure a controlled ventilation.

4. SOLUTIONS FOR IMPROVEMENT OF THE GLAZED ELEMENTS

Based on the exposed principles, some practical solutions have been elaborated:

a - In order to increase the thermal resistance of the glazed zones of the windows:

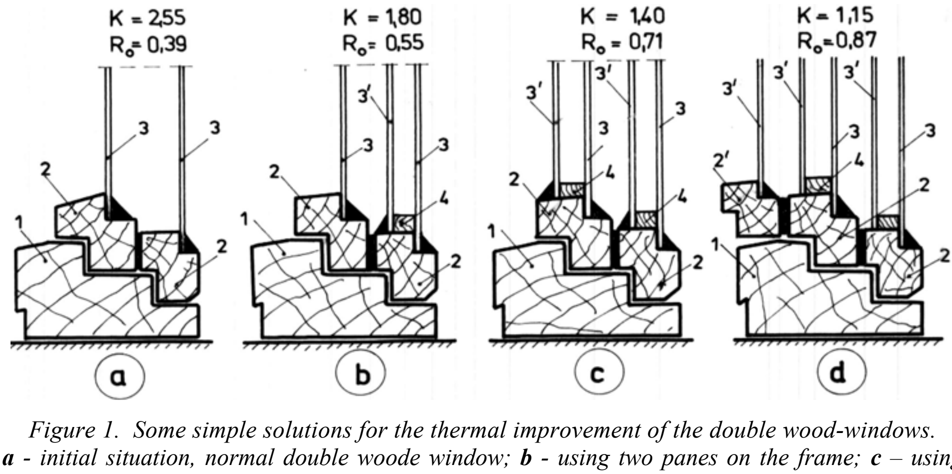

Figure 1. Some simple solutions for the thermal improvement of the double wood-windows. ***a*** *- initial situation, normal double woode window;* ***b*** *- using two panes on the frame;* ***c*** *– using double pane on both frames;* ***d*** *– with double panes on existing frames and a new simple window; 1 - jamb; 2 – frames; 2'– new jamb; 3 – existing glasses; 3'– additional new glasses; 4 - wood-slat.*

- The most simple solution is to attach a thin ordinary pane, 2...3 mm thickness, on the one of existing frames, using a distance-slat (Fig.1b), to close thus a vertical air layer, 1..3 cm thick. Because is not possible to ensure a perfect tightness of this new pane, some holes in the frame can ensure an useful air circulation in the space between the glasses. This solution can increase the thermal resistance of transparent zones with 40 % about.

- If a supplementary pane is applied on the both frames of the window (Fig.1c), the increase of thermal resistance can reach 80...85 %. This solution is easy to apply, but the difficult access between the panes for periodical cleaning make it less preferable.

- With another additional pane, fixed on new frame, and double glasses on the both existing frames the thermal performance of window increases at 125% given the initial double window (Fig.1d), but this solution have some disadvantages

- Using high quality window-panes on existing frames instead of usual glasses, or a new high thermal quality modern window, the thermal resistance increases to $R_0 = 0{,}96$ m^2K/W, therefore with 146 %, this being the most performant solution, even if their actually cost is still high comparativelly with other analized solutions.

After many years of service with thus improvement solutions, results that the most preferred variant are the first and the third, possibly together, because the new simple window is independent and permits the facile cleaning, also the double glas improve the hygrothermal behaviour of window.

b - The decrease of the air flow through the windows can be realized thus:

- in the case of old buildings, an overhauling of windows is necessary, with a view to replace the degraded components, as well as to eliminate the deformations.

- in order to ensure a supplementary air-tightness of the windows, who can stop the supplementary air change of the rooms, it can be used: garnishes made of foamy or spongy materials, like the polyurethane, the polystyrene, the rubber etc (Fig. 2a), also garnishes of thick synthetic stuff (Fig. 2b), as well as some special systems made of rubber, neoprene or other elastic materials, fixed in the joints of windows (Fig. 2 c).

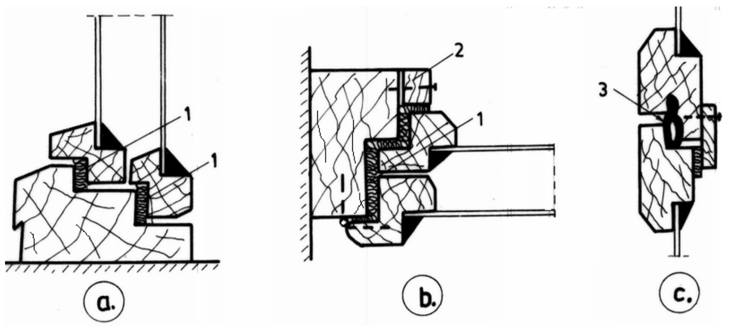

Figure 2. Some simple, usual solutions for diminish the air tightness of the window elements.
***a** - using band fittings (1); **b** - using wood laths (2); **c** – using tube fittings (3).*

In order to eliminate the risk of the excessive vapour accumulation inside the rooms, as a result of too high air tightness obtained applying these measures, the mobile transoms, as well as the fans constitute the best solution.

5. CONCLUSIONS

The thermal improvement of existing windows can be realized using some radical but expensive solutions, like the high thermal quality new windows, or by means of simple, accessible and cheap technical measures, like the multiple glases on the existing windows, as well as to seal the nontight joints, which practically can ensure similar heating effects if their achievement and maintenance are adecuate.

In the vast action of thermal rehabilitation of existing old buildings, especially of the numerous habitual old blocks of flats, also in the improvement of the glazed elements too, the mentioned simple solutions are often preferable, by economical and technical point of view. The obtained results in the actually conditions in our country confirm the suitableness of these solutions.

Certainly, from point of view of thermal qualities, the modern solution based on windows with high thermal resistance are the best, being recommended to replace the old glazed elements, even if the financial aspect is still non attractive for many lodgers.

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BEHAVIOUR SOME RC ELEMENTS STRENTHGENED WITH FRP SYSTEMS

Summary: *Extensive research on the use of fibre reinforced polymer (FRP) composites and its behaviour in the strengthening of reinforced concrete (RC) structures has been carried out in recent years. One of the main aspects considered is ductility and, in case of dynamic loading, fatigue. These characteristics are imperative in order to prevent any possibility of brittle failure modes in structure's service life, especially since FRP materials are very brittle. To achieve this, besides being familiar with failure modes, it is necessary to provide adequate ductility especially in columns. This paper considers RC elements behaviour of strengthened with FRP as a composite structure. Analysed within are methods providing ductile behaviour of RC columns strengthened with FRP systems and recommended guidelines given by ACI, CSC and FIB.*

Key words: *RC elements, FRP elements, ductility, failure modes, fatigue, confinement, columns*

PONAŠANJE NEKIH ARMIRANOBETONSKIH ELEMENATA POJAČANIH FRP KOMPOZITIMA

Rezime: *Poslednjih godina se intenzivno istražuje ponašanje vlaknima ojačanih polimera (FRP) primenjenih za pojačavanje armiranobetonskih (AB) konstrukcija. Jedno od bitnih svojstava pri primeni ovog postupka pojačavanja je duktilnost, a u slučaju dinamičkih opterećenja i zamor materijala. Ova svojstva su veoma bitna da bi sprečila neželjene oblike otkaza (krti lom), u toku eksploatacionog veka, jer su materijali za pojačavanje krti. Da bi se ovo postiglo potrebno je poznavati mehanizme loma i obezbediti adekvatnu duktilnost, naročito u stubovima. U radu se razmatraju AB elementi pojačani FRP materijalima i njihovo ponašanje. Analizirane su smernice Američkog instituta za beton, Britanskog društva za beton i Internacionalne federacije za beton koje se odnose na pojačavanje stubova.*

Ključne reči: *AB elementi, FRP elementi, duktilnost, mehanizmi loma, zamor, utezanje, stubovi*

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1. INTRODUCTION

Fibre reinforced polymers (FRP) systems are primarily used to strengthen existing structures. Structures may need strengthening due to deterioration, design or construction errors, a change in use or loading, or for a seismic upgrade [8]. These fairly new systems are being used in various forms (plates, sheets, laminates, bars, shells etc.) and essentially work as reinforcement in concrete and provide strength where concrete is weakest- in tension. Among many possible applications, their use as externally bonded systems of reinforced concrete structures is the most common one [16]. Advantages of these materials to steel plates, and other traditional methods of repair, in their excellent corrosion resistance and high stiffness-to-weight ratios are evident, but high cost and lack of long-term performance is still considered to be an unfavourable restraint when considering usage of FRP systems. It is very important estimate strength and deformability strengthened reinforced concrete (RC) elements. These are investigated experimental and numerical, but mostly bearing capacity and failure mode, and ductility and fatigue, in aim to formulated adequate mathematical models [3], [13] i [17].

The use of carbon, aramid or glass fibres combined with epoxy, vinyl ester, phenolic or isophthalic resin to form an FRP composite. They may be applied to concrete structures in order to strengthen them and increase their load-carrying capacity. The increase in FRP application has be a result of growing confidence in the materials, technique and methods of strengthening. FRP may be used on beam or slab soffits to provide additional flexural strength, or wrapped around columns to provide confinement and additional ductility, which is a primary concern in seismic upgrades. Practical application of techniques began in the mid-1990s for typical structural are [6]:

- Bridges with strengthening bridge decks, with the application: of FRP plates on both the soffits and top surfaces; edge beams have been strengthened using proprietary shear strap; columns have been wrapped, or encased in shells, to improve impact resistance and for improve columns ductility and their seismic performance.
- Buildings for strengthening floor slabs to carry additional imposed loads and strengthened columns to carry additional loads. This strengthening is very effective because of the speed of installation and the minimal increase in the column dimensions. This material used to replace missing hoop reinforcement in columns or to improve seismic performance.
- Industrial structures included strengthening power station cooling towers to resist wind loading, pier and jetties.

For increase of wide application of fibre composites to strengthening RC structures it is very important understanding of the behaviour between them (the FRP composites and the RC elements), especially under extreme forms of loading and different laminate configuration [3] and [4]. Rison for that are actions where ultimate strength of FRP governs failure. Also new material and technique such as the use near surface – mounted (NSM) reinforcement and prestressing of FRP and anchorage system, which developed recently is important to know [5] and [6].

There are three main areas where strengthening using FRPs is performed: flexural, shear and compression strengthening. RC Beams and slabs can strengthened in flexure by addition of FRP reinforcement to the tension force of the member. Although analysis for flexural and shear capacities draw on many of the same assumptions used for steel reinforcement, significant differences between the material properties and mechanical properties and mechanical behaviour of FRP and steel necessitate a shift away from conventional concrete design philosophy. The linear elastic stress-strain characteristic of most FRP composites are 1-3% ultimate strain, implies that FRP reinforced concrete design procedures must account for inherently less ductility than that exhibited by conventionally reinforced concrete [2].

1.1. Design issue

FRP reinforcement concrete is designed using limit states principles to ensure sufficient strength, to determine the governing failure mode, and to verify adequate bond strength. Serviceability limit states (deflection and crack with, stress level under fatigue or sustained load), and relaxation losses for prestressed concrete are then checked. Two main issues must be addressed: 1) the ultimate moment capacity (described in [10]), and 2) the possibility of premature separation of FRP from the concrete surface ([11]). In [5] practical approach is suggested based upon sectional analysis easier for application in practice. In this method are following steps:

- Calculate the ultimate capacity (UC) of the unstrengthened section and ensure that the critical section is at least capable of withstanding the unfactored loading condition, so that collapse should not occur if FRP failure occurs.
- Estimate the amount of FRP required assuming the neutral axis remains in the same place as for the unstrengthened case and limiting the strain in the FRP to 0.008.
- Assume the maximum compressive strain in the concrete is less than 0.0035 and assume a neutral axis position.
- The neutral axis is than adjusted until horizontal equilibrium is obtained.
- The moment capacity can be calculated by taking moments of the forces about a suitable position, and compared with the required ultimate moment.
- If the moment of resistance is less than that required, the initial estimate of the compressive strain is increased, up to 0.0035, and the process repeated until the moment of resistance is $\geq$ the required ultimate moment (UM).
- Should be required UM not be achievable for a given quantity of FRP, an incrementally higher quantity of FRP is assumed. The calculated stress and strains must be assessed against the separation failure criteria.
- Should be calculated moment of resistance be substantially grater than that required, a lower quantity of FRP could be chosen, and process repeated.

The existing approach for RC beams can be followed with the FRP treated as additional reinforcement, provided the brittle nature of RFP is taken into account. For either FRP rupture of concrete crushing, the steel reinforcement generally has already yielded at failure. If failure is by FRP rupture, the compressive concrete cannot in general be assumed to have reached failure in strength calculation [18]. Because FRPs only have a high strength in the direction of the main fibres, these may be so orientated as to best reinforce the beam [1] and [3].

If the limit of shear stresses satisfied, the ultimate shear capacity of FRP strengthened beam can be expressed as the sum of the shear resistance of the concrete, the steel reinforcement and the additional FRP strengthening. The contribution from FRP is dependent upon the effective strain in the FRP at failure and effective depth of the FRP at failure and the FRP shear reinforcement. Calculation of the effective strain, ε_{fe}, is the key to evaluating the amount of additional shear strengthening given by FRP [6].

Experimental result showed that the distribution of the FRP to the shear strength of the beam is less than its ultimate tensile strength. According to [18] Triantafillou proposed the use of an effective strain at shear failure instead of the ultimate strain of FRP. Shear forces in a beam may be reversed under revised cyclic loading. Fibres may be arranged in two different directions to satisfy the requirement of shear strengthening in both directions. Strengthening for revised shear forces FRP sheets with fibres in three directions (0, 60 or 120^0) may used [4].

1.2. Mechanisms and failure modes

All three types of FRP's is that their stress-strain behaviour is linearly elastic [2] until rupture (Fig. 1). These materials do not possess the ductility that steels have and their brittleness may limit the ductile behaviour of RC member strengthened with FRP composites in RC beams bonded with an FRP soffit plate, failure by concrete crushing is permissible, as the FRP rupture mode is also brittle. This contrasts with normal RC beams design where steel yielding should be ensured to precede concrete crushing. Brittle behaviour of FRPs caused limited redistribution.

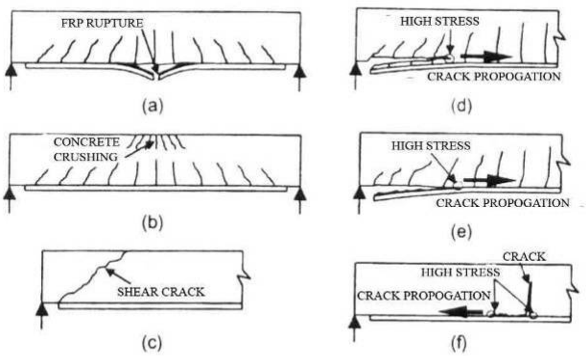

Figure 1. Failure modes of FRP plated RC beams, after [18]

A number of failure modes are possible. If the ends of the plate are properly anchored, then failure occurs when the ultimate flexural capacity of the beam is reached, by either tensile rapture of the FRP element (Fig. 1.a) or crushing of concrete (Fig. 1.b). A shear brittle failure of an RC element is also possible if flexural capacity of the strengthened beam exceeds the shear capacity of the RC beam alone (Fig. 1.c). Also, a variety of de-bonding failure modes have been observed. These have been roughly categorized as: a/ those associated with high interfacial stresses near the ends of the bonded plate (Fig. 1.d and Fig. 1.e), and b/ those induced by a flexural or flexural-shear crack (intermediate crack) away from plate ends (Fig. 1.f).

The three distinct failure characteristics of FRP strengthened flexural elements: ductile, near ductile and brittle failure modes are inextricably linked to well defined ductility and deformability indices. Ductility formulation for FRP strengthened elements are given, throw total (E_{tot}) and elastic energy (E_{el}), in [18]:

$$\phi_{du} = \frac{1}{2}\left(\frac{E_{tot}}{E_{el}} + 1\right) \tag{1}$$

1.3. Fatigue

Many tests have been conducted worldwide over the past few decades to analyse and improve short term behaviour of under-reinforced concrete elements strengthened with FRP systems, but researchers have paid little attention to the problem of ductility and fatigue [7]. FRP elements significantly increase strength of reinforced concrete elements and structures, but since FRP material does not exhibit plastic strains as steel does, its brittle behaviour must be accounted for in structural design. Also, fatigue response of FRP material, as a result of long term behaviour, has to be considered. Tests conducted on RC beams strengthened with FRP systems have shown improvement not only in short term behaviour, but also in fatigue.

Fatigue is often described as a physical phenomenon that can lead (often as a result of multiple reasons) to falling of the structure. Concrete structures housing large vibrating machinery are submitted to cyclic loading with complex stresses. Therefore, fatigue behaviour of such structures is a big concern. RC railway bridges also exhibit fatigue behaviour, although they can endure millions of cycles of repeated axial loading during their service life. Cracking as a result of material's fatigue, happens either from tension in steel reinforcement, in concrete from compression, or along the concrete and the embedded steel. Further propagation of deflection and cracking can result to diminished service life of the structure. In general, fatigue is considered to be a process of damage accumulation in material submitted to live loading.

Fatigue behaviour can lead to crushing even if loading is far below yielding. Classical fatigue behaviour analysis is based on fracture mechanics and S-N (stress versus number of cycles) approach. FRP are considered to be resistant to fatigue to the extent that fatigue may be neglected at a material's level. By characterizing the fatigue behaviour of FRP elements as S-N diagram, it is noticeable that slope of the curve will substantially be reduced at low stress level, but the apparent endurance level for most FRP materials will not be obtained. Therefore, it is common design practice to specify the fatigue strength as endurance limit of the FRP material at very high number of cycles (10^6 or even 10^7 cycles). Unlike steel, FRP composites subjected to cyclic loads can exhibit gradual softening or loss in stiffness due to microscopic damage. Damage and cracking resulting from fatigue is one of the main reasons for distress when using FRP materials on building and bridge substructures.

1.4. Ductility

Failure in RC beams is initiated by yielding of the steel reinforcement and followed by concrete crushing at ultimate state, after considerable deformations and forming of plastic hinges. This failure mode is ductile and favourable. Ductility is ability of post-elastic deformations. It is a desirable structural property since it allows stress redistribution and provides failure warning.

FRP material, unlike steel, shows only elastic behaviour and does not have plastic deformations. Hence, its failure modes are brittle and sudden, and load capacity is reached with little or no inelastic deformations. At the structural level, components fabricated from FRP composites can be designed to exhibit a sequence of damage mechanisms which ensures a gradual failure with extensive deformation, leading to a progressive and safe mode of failure.

Since design of FRP elements on structures in flexure is based on Bernoulli's hypothesis of strain compatibility, if ultimate limit state (ULS) is considered, the amount of FRP material used will be considerably higher than what is needed for the serviceability limit state (SLS), and ductility demands of strengthened RC sections might not be fulfilled. Existing guidelines recommend that, no matter how large the ultimate moment is, the service moment of the strengthened RC cross section should not exceed the value reaching yielding of steel reinforcement, in order to provide plastic deformations in the member at service. Thus, predicted failure mode for the cross section is always concrete crushing.

FRP composites can contribute to increase the ductility of other structural systems (i.e. rehabilitation of columns by wrapping which improves shear strength, confinement and ductility). If structure express ductility, it will also possess rotation capacity. However, if a structure displays rotation capacity, it does not necessarily displays ductility, which is the case of RC elements strengthened with FRP. Moment redistribution relies on rotation capacity of critical sections, and in case of FRP strengthened zones, it is limited to 30%. Out of FRP strengthened zones, moment redistribution is not recommended [5]. Confinement of concrete columns with the use of external FRP jacketing has been proven to result in enhanced system ductility and hence performance under seismic loading. This is the case because FRP confinement provides constraint to lateral strain.

2. COLUMNS RETROFITTING AND DUCTILITY ENHANCEMENT

Compressive strength and strain capacity of columns can be increased by confining the concrete (FRP jackets wrapping of columns). This can be achieved by wrapping the column circumferentially with FRP. For designing columns for concentric and eccentric axial load used stress-strain model of Lam and Teng which described the main aspects of the behaviour of FRP – confined circular concrete columns in a simple form [6]. It is recommended that the maximum usable compressive strain should be limited to 0.01. Model may be used for concentric and eccentric load except in cases where confinement is low or when eccentricity is high. In these situations a full confinement model is inappropriate and the use of an extended strain unconfined stress-strain model is suggested.

2.1. ACI Guidance

Bonding FRP to the column surface enhances axial load capacity and ductility of the columns [1] and [12]. For a circular column confined with FRP elements, axial compressive strength can be evaluated from:

$$\Phi \cdot P_n = 0{,}85 \cdot \Phi\left[0{,}85\Psi_f f'_{cc}\left(A_g - A_{st}\right) + f_y A_{st}\right] \text{ (column with spiral stirrups)} \tag{2}$$

$$\Phi \cdot P_n = 0{,}80 \cdot \Phi\left[0{,}85\Psi_f f'_{cc}\left(A_g - A_{st}\right) + f_y A_{st}\right] \text{ (column with hoop stirrups)} \tag{3}$$

where a reduction factor ψ_f=0.95 accounts for lower reliability of FRP elements in comparison to steel reinforcement, and strength reduction factor Φ is given by ACI 318 standard depending on whether spiral or hoop stirrups have been used.

At load levels close to ultimate, radial cracks will appear in concrete. FRP jacket is then activated to ensure the structural integrity of the column. ULS is to be avoided by preventing lateral strain. According to ACI, this can be achieved by limiting stress in concrete to 0.65 $f_c^{'}$, and in steel to 0.6 f_y. Thus, FRP confinement remains passive, and will only act during overloads.

2.1.1. Square and rectangular cross section columns

FRP confinement is most effective with circular columns, because it provides uniform confinement pressure and hence prevents lateral strain, so with circular sections, efficiency factor κ_a=1.0. With rectangular cross section columns limited strength improvement has been noted after wrapping them with FRP elements. ACI does not give guidelines for confining rectangular section columns but suggests further testing. Although confining FRP elements with rectangular columns does not improve its compression strength significantly, it does enhance ductility. The reinforcement ratio for rectangular sections can be found from:

$$\rho_f = \frac{2n \cdot t_f (b+h)}{b \cdot h} \tag{4}$$

The confining efficiency factor for rectangular sections, with r being radius of the edges, should be determined as follows:

$$k_a = 1 - \frac{(b-2r)^2 + (h-2r)^2}{3b \cdot h(1-\rho_g)} \tag{5}$$

2.1.2. Ductility

Enhancement of ductile behaviour enables grater lateral strain of the column's section. According to ACI, increased ductility of section results from the ability to develop greater compressive strains in the concrete prior to compressive failure (Seible et al. 1997). The FRP jacket can also serve to prevent buckling of a longitudinal reinforcement in compression and to clamp lap splices of longitudinal steel reinforcement. For seismic applications, jackets should be designed to provide a confining stress sufficient to develop concrete compression strains associated with the desired plastic displacements. Although, confining square and rectangular sections is not as effective in increasing axial capacity, it is proven to be effective in improving the ductility of compression members. Also, rectangular sections with aspect ratios b/h>1.5 or with one dimension exceeding 90cm (36”), should not be confined with FRP jackets unless testing demonstrates their effectiveness.

In design, maximum compressive strain in concrete for FRP confined circular RC column, according to ACI, is limited to (Mander et al. 1988):

$$\varepsilon_{cc}^{'} = \frac{1.71(5 \cdot f_{cc}^{'} - 4 \cdot f_c^{'})}{E_c} \tag{6}$$

Shear forces should also be evaluated to prevent brittle shear failure.

2.2. CSC guidelines

Although steel confinement is in use much longer, its disadvantages are obvious. Steel is very corrosive, diminishes construction's aesthetic value and incompatibility of Poisson's ratios between concrete and steel is present. Those were the main reasons to start using FRP material for confinement with concrete instead of steel.

Experimental results on cylindrical specimens exposed to hydrostatic pressure (Richart et al. 1929.) have shown that increase of lateral confinement pressure increases also concrete compressive strength and section's ductility. Increase in compressive strength of concrete is given by:

$$f_{ccd} = f_{c0} + 4{,}1 \cdot f_r \tag{7}$$

with
f_{ccd} confined concrete compressive strength,
f_{c0} unconfined concrete compressive strength, and
f_r confining pressure.

2.2.3. Square and rectangular cross section columns

As stated earlier, confining square columns is generally acknowledged to be less efficient than confining circular ones. This is contributed to the fact that confinement with square and rectangular columns is concentrated mainly at the corners and not over the entire perimeter. Current research on small columns has shown that the maximum achievable increase in compressive stress for FRP-confined square columns with reasonable levels of rounding of corners is about 50%, compared with up to 200% for circular columns. For larger columns, the level in increase may be even less than this. The efficiency decreases further with columns of rectangular cross-section and/or large side dimensions.

The generally accepted theoretical approach is to develop an area of effective confinement defined by four parabolas within which the concrete is fully confined, and outside of which negligible confinement occurs.

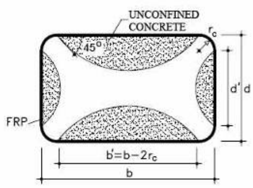

Figure 2 – Assumed confined region for rectangular column (white- effective confined region)

Confinement efficiency can be improved by rounding the corners. Where it is not possible to shape the corners, to a radius of at least 15mm, significant stress concentrations may occur resulting in FRP failure. In theses cases, it is beneficial to use additional localised FRP reinforcement at the corners prior to the application of the confinement.

2.2.4. Ductility

Some columns designed according to older design codes may be incapable to withstand large horizontal displacements that occur between member ends during earthquakes. Therefore, they require ductility enhancement in order to hold the cover concrete in place and prevent buckling of longitudinal reinforcement under axial load. Upgrading normally involves confining the concrete with FRP jackets at column ends where bending moments are greatest. Ductility enhancement may increase the risk of shear failure, so shear strength must also be considered in any proposed column upgrading. This is especially important with slender columns where $M/VD>4$ (M being the maximum bending moment, V shear force and D column diameter). Since seismic loading is not a major loading case for United Kingdom, Technical Report 55 does not analyse further this aspect.

2.3. FIB guidelines

Experimental evidence shows that the ultimate strength of confined concrete is closely related to the failure strain of the FRP confinement. This circumferential strain mostly occurs at strains lower than the ultimate strain ε_{fu} of the FRP material. This reduction is due to several reasons:

- the three axial state of stress of the wrapping reinforcement ,
- strains in concrete are not homogenous due to creeps,
- the quality of application,
- size effects when applying several layers.

Considering all of the above, FRP confinement stiffness of a circular column is given by:

$$K_{conf.} = \frac{\rho_f \cdot E_f}{2} \cdot k_e \tag{8}$$

If the concrete is partially wrapped, less efficiency is obtained as both confined and unconfined zones exist (Fig. 3). In this case, the effective lateral confining pressure is obtained by introducing a confinement effectiveness coefficient. For a fully wrapped circular column, this coefficient is k_e=1.0, but in case of existing confined and unconfined zones, then k_e≤1.0. As illustrated on figure 3, the confining arching effect between two subsequent wraps, is assumed to be a parabola with an initial slope of 45^{o}.

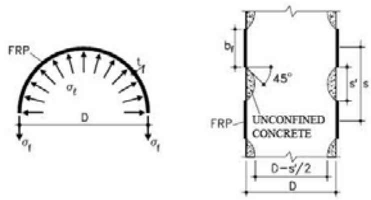

Figure 3 - Confining pressure exerted by the FRP

3. CONCLUSIONS

Over-strengthening due to used of large area of FRP composites is express high deformability bat low ductility. Consequently the strengthened members are likely to fail in a brittle way at ultimate limit state. Elements with light internal steel reinforcement (slab) exhibit more ductile behaviour. For ductile failure, the deformability and ductility indices will converge to similar values. As the ratio of deformability to ductility increases the failure mode becomes more brittle [18].

For the ductile failure mode in an FRP strengthened flexural element minimum ductility index should be 2.0, whereas the desirable value is 2.5 or greater. New results of investigation effectively influenced on improving the application FRP for repair and strengthening RC structural elements, and for innovation of Guidelines for rehabilitation design and executed.

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Margarita Nikolova Hamova[1]

GEOTECHNICAL AND ECOLOGICAL PROBLEMS RELATED TO THE BUILDING AND FACILITIES CONSTRUCTION

Summary: *Earth crust and relief formation has taken place for millions of years. This process also takes place today, but it is slow and imperceptible. It is considered that in the nature a relative equilibrium has been reached, on which the anthropogenic construction activity has crude and aggressive impact. The basic factors leading to changes in the environment, related to the construction of buildings and facilities, are examined and classified in this paper. The measures for construction, which is synergic with the environment, and the environmentally correct terrain exploitation are outlined.*
Key words*: geotechnical and ecological problems, construction, terrain exploitation*

GEOTEHNIČKI I EKOLOŠKI PROBLEMI PRI GRAĐENJU ZGRADA I INDUSTRIJSKIH OBJEKATA

Rezime: *Formiranje zemljine kore i reljefa se odvijalo milionima godina. Ovaj proces se odvija i danas,ali je spor i nevidljiv. Razmatrano je da je u prirodi dostignuta relativna ravnoteža, na koju proces građenja ima grub i agresivan uticaj. Osnovni faktori koji vode ka promenama u okolini povezani su sa procesom građenja objekata, su ispitani i klasifikovani u ovom radu. Mere za građenje, koje su u skladu sa okolinom i ekološki ispravna eksploatacija tla su ukratko prikazane.*
Ključne reči*: geotehnički i ekološki problemi, građenje, eksploatacija zemljišta*

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1. CONSTRUCTION AS A FACTOR FOR CHANGES IN THE ENVIRONMENT

The formation of the Earth's crust and its relief had proceeded in the course of millions of years. During this long period mountains, hills, water basins, rivers, etc., were formed. As a result of the weathering processes, a great part of the basic rocks was covered with easily transported and deformed disperse soils. These processes of replacing and deforming occurred at a slow rate during the entire period of relief formation and seem to be completed when referred to the present moment. Hence an apparent long-term natural equilibrium has been established. However, construction anthropogenic activities provoke severe disturbances in this equilibrium.

Human life is impossible without construction. Man started to build as early in the remotest past – after leaving the cave dwellings primitive men had to build themselves shelters for protection against natural disasters and wild animals. Today people build residential, industrial and administrative buildings, roads, tunnels, dams, etc. It could be stated that in its construction activity mankind reaches greater heights and greater depths. As a result of this process, which is considered to be unlimited and difficult to predict, serious and irreversible consequences for the environment are in progress.

1.1 Classification of the geotechnical and ecological changes due to anthropogenic construction activities

Nowadays construction may be considered as the most large scale factor introducing changes in the environment in the following directions:

- **Changes in the stressed and strained state of the soil massif**

The construction of buildings and facilities is connected with the execution of large scale excavations and embankments. As a result some terrains are deprived of their natural loading and become loose while others are additionally loaded and deformed. Except for the embankments, the loading is increased by the newly constructed buildings and facilities. The significantly increased loading on the soil base may provoke not only exceeding of the admissible deformations but also exceeding of the soil shear strength, when plastic zones and sliding surfaces are formed. This leads to loss of bearing capacity and catastrophic consequences for the building or the equipment.

Two schemes are possible here, which are related with the changes in the environment and the loading on the soil massif (Fig. 1).

The performance of excavation and embankment works is connected with translation and movement of natural material – the soil, from one place to another (Fig. 1a), while the output and processing of natural materials for producing building materials is connected with its incorporation in buildings and facilities (Fig. 1b). In both cases the natural loading on the terrains at the sites of natural material extraction is decreased, while the loading on the terrains of the construction sites is increased.

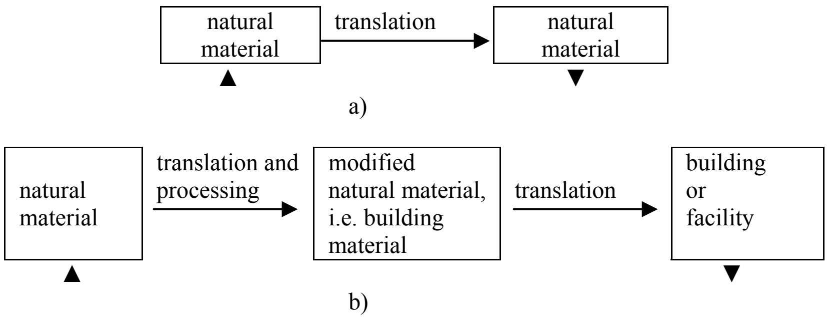


Fig. 1. Possible schemes for the changes in the environment and loading during construction activities (▲,▼- decreasing or increasing the load on the soil massif)

• Changes in the temperature-humidity regime of soil and air

It has been proved by experience that construction on a given territory is connected with rising of the groundwater table and increasing of the temperature of soil, air and water. Very often a building that has been founded above the groundwater level turns to be under water after construction on its surrounding territory. This fact has to be known, so that appropriate hydraulic insulation of foundations could be envisaged.

The construction of large artificial water reservoirs leads to permeation of water to great depths and hence to increasing the groundwater table of the surrounding terrain. It is considered that this is one of the reasons for provoking earthquakes.

• Exhaustion and modification of natural resources

Modern construction requires vast amounts of concrete and steel. The production of concrete needs natural aggregate materials and cement, which is actually processed natural rock material. All these resources transformed into concrete, represent modification of one type of materials to another, in combination of transferring these materials from the quarries to the building sites. The same refers to steel and the rest of the most frequently applied building materials – bricks, mortar, etc., which represent processed and transferred from one place to another natural materials. In this way construction in its essence represents huge modification and translation of natural resources.

At the same time the buildings and facilities grow old and are transformed in the course of time into depots of processed and useless wastes occupying still wider territories. To maintain the living standard of people, new buildings are constructed that usually occupy new territories, previously used for agriculture and food production for the population. This continuous process threatens the balance of agricultural areas on world scale.

• Pollution of soil and water with toxic chemical substances

This type of ecological pollution is the direct consequence from the construction of chemical enterprises, output of materials by means of chemical methods, soil strengthening using toxic waste materials, dumping of industrial waste products.

Some of the chemical substances alter to a significant extent the physical and mechanical parameters of the soil and the conditions for buildings and facilities foundation.

• **Increase of the dynamic impact on the soil base**

The increased dynamic impact on the soil base is connected with the development of machine building, transport, military industry, etc. The dynamic effects make the bonds between soil particles weaker and lead to reducing the soil shear strength and hence to loss of stability. The weak soils as fine-grained and silty water saturated sands, water-saturated silts and clays, poorly compacted embankments, etc., are especially susceptible to dynamic impacts. Sometimes even slight dynamic effects lead to catastrophic consequences for the soil base and the relief of the territory.

As seen from the above mentioned considerations, construction represents the most large-scale factor for changes in the environment from ancient times till the present days. When observed from above, the settlements with their facilities look like tumor formations on the Earth's crust. However, nature strives to restore the equilibrium in the relief forms again destroying a part of the already built facilities by landslides, rack-falls, earthquakes and other natural disasters, that have been provoked to a certain degree by human construction activities.

1.2. Measures for protecting the natural environment and the relief of the Earth

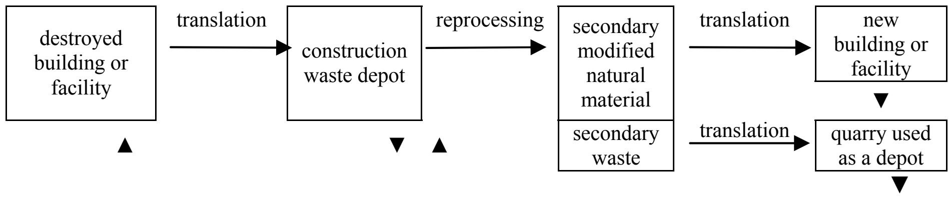
The construction of buildings and facilities should be performed in harmony with the environment in contrast to the harsh interference within the existing relief forms [1]. However, contemporary construction has acquired such dimensions that it is impossible to leave the environment undisturbed. The aspiration is to keep the changes due to construction activities at minimum levels.

The following rules have to be adhered to for the protection of the natural environment:

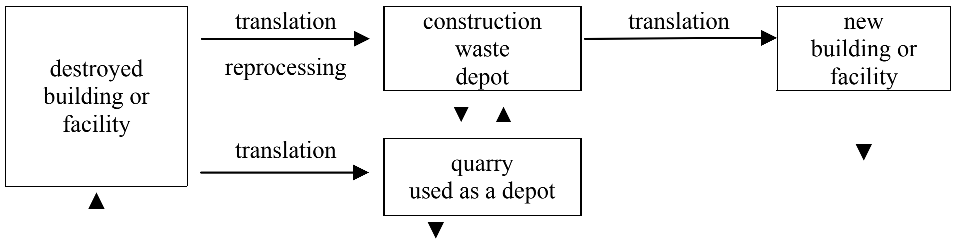
• The already used waste construction materials have to be recycled and used again. The construction wastes that have not been utilized have to be applied for filling up exhausted quarries, restoring in this way a part of the loading on the soil massif. The processes of environmental friendly reprocessing and storing of construction waste materials may be described by the scheme presented in Fig. 2.

The construction wastes from destroyed buildings and facilities may be reprocessed entirely (Fig. 2a) or partially (Fig. 2b) into recycled building materials, which represent secondary modified natural products. Both schemes ensure economizing of natural resources and restoring the soil massif loading on terrains, where output of natural resources has taken place either for direct use or reprocessing.

• Terrains that are not suitable for agriculture have to be used for construction purposes [2]. In the case of existing weak soils, deep foundation has to be applied or the soils have to be strengthened by natural and non-toxic materials.



a)



b)

Fig. 2. Possible schemes for reprocessing, utilizing and storing construction waste materials (▲, ▼ – decreasing or increasing the load on the soil massif)

• To avoid provoking of landslide processes, several basic rules have to be obeyed when construction takes place on inclined terrains – the buildings have to be made along the inclination without cutting the slope, the construction on the terrain has to start from the foot of the slope with higher and heavier buildings and the storey-number should be decreased with the slope height. Construction on inclined terrains requires obligatory drainage with simultaneous construction of the water supply and sewerage system.

In conclusion it has to be noted that the above mentioned measures for ecological protection during the construction of buildings and facilities do not claim to be exhaustive. But the greater is the awareness for protection of nature during construction and exploitation of the terrains, the better will be preserved the natural equilibrium and the relief of the Earth.

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POSSIBILITIES FOR CONSTRUCTION ON POTENTIAL LANDSLIDE TERRAINS

Summary: The natural relief forms are usually stable and are in equilibrium even when situated with steep inclination. Under conditions of incorrect construction and exploitation this equilibrium is disturbed and the hazard of sliding emerges for the terrain together with the buildings and facilities on it. This is the case with many terrains along the Bulgarian Black Sea coast. In order to avoid this risk construction on such terrains is often forbidden. Instead of construction prohibition, suitable measures are proposed here allowing the construction on terrains, potentially endangered by landslides. Except the activities for construction, operation measures are also proposed for protecting the buildings and equipment against sliding. World experience has proved that successful and reliable construction is possible even under much more unfavourable engineering geological conditions than these in Bulgaria.
Key words: landslide terrains, activities for construction and operation

MOGUĆNOSTI ZA GRAĐENJE NA POTENCIJALNIM KLIZIŠTIMA

Rezime: Prirodni oblici reljefa su najčešće stabilni i u ravnoteži čak i kada se nalaze na stepenastim kosinama. Pri uslovima nepravilne gradnje i eksploatacije ravnoteža se narušava i rizik od klizanja se pojavljuje u zemljištu zajedno sa objektima i postrojenjima na njemu. Ovo je slučaj sa mnogim terenima duž Bugarske obale Crnog mora. U cilju izbegavanja ovog rizika na takvim terenima građenje je često zabranjeno. Umesto zabrane građenja, ovde su predložene odgovarajuće mere kojima je dozvoljeno građenje na terenima koji koji su potencijalno ugroženi klizištima. Osim aktivnosti za građenje, mere upravljanja su takođe predložene radi zaštite objekata i opreme od klizanja. Svetsko iskustvo potvrđuje da je uspešna i pouzdana gradnja moguća čak i pod mnogo nepovoljnijim inženjersko geološkim uslovima nego u Bugarskoj.
***Ključne reči**: klizišta, aktivnosti za građenje i upravljanje*

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1. CLASSIFICATION OF MEASURES FOR CONSTRUCTION ON POTENTIAL LANDSLIDE TERRAINS

A significant part of the relief in Bulgaria is hilly and mountainous, consisting mainly of disperse soils obtained from the weathering of the basic rock species. Equilibrium had been attained in the course of millennia and stable relief forms were formed, many of them being endangered by the expansion of construction activities.

The urbanization of territories usually started on plain terrains around water basins and with settlement enlargement construction spread gradually on the surrounding hilly areas. The initial low-storey and light construction is not fatal for slope stability - the problems arise with increasing the height and density of the buildings. Nowadays there are many data for the origin of landslides in settlement areas on hilly terrains. On the other side, there are vast low-productive inclined terrains in the country, mainly along the seacoast, which are private ownership and have been used for years as vineyards, orchards, etc., the predominant construction on them consisting mainly of bungalows and ancillary buildings. They are especially valuable construction terrains now with their panorama view to the sea but are subjected to landslide processes due to the specific features of the terrain and the soil as well as due to anthropogenic impacts. Prohibition of construction has been imposed for some of these terrains [1], which is indispensable for most of them because of the existing deep sliding surfaces and abundant groundwater, making the stabilization and operation of the terrains inexpedient. In some cases however, the problems ensue from indiscriminate construction and lack of drainage facilities and canalization, leading to super-moistening of the terrain in within short time periods. For the case of landslide terrains along the seacoast this happens during the summer season, when water from the life activity of many people is accumulated in a short time.

It is possible to carry out construction activities on potentially endangered by landslides terrains due to incorrect exploitation, if some preliminary measures are taken and certain rules for construction and operation are observed:

- Performing detailed engineering geological and hydrological investigations on the whole hazardous terrain with the view of exploring the soil base, finding eventual sliding surfaces, establishing the level and regime of groundwater. The strength characteristics of the soil, which are necessary for investigating the stability, should be determined under static and dynamic conditions with peak and residual values.
- Geodetic survey of the terrain, without which it is impossible to investigate the terrain stability, the vertical planning and design of drainage facilities, as well as communication equipment above and under the ground.
- Detailed investigation of the terrain stability for all possible cases – without construction, with construction – in stages or simultaneously along the whole slope, under static and dynamic conditions taking under consideration the corresponding earthquake degree. These calculations show also the necessity of new shaping of the terrain slopes – the stability is increased after removing soil from the top and depositing it in the foot of the slope.
- Developing detailed plan for arrangement and construction of the territory [1]. Except for the usual attributes, the construction plan should contain also the routes of the underground communications and drainage facilities, as well as the storey number and the sequence of the construction stages for the

territory. The highest and the heaviest buildings should be situated in the foot of the slope and are the first to be built – in this way they will serve as an abutment against eventual sliding of the terrain. The construction should be performed in stages in upward direction with gradual decrease of the storey number (Fig. 1).

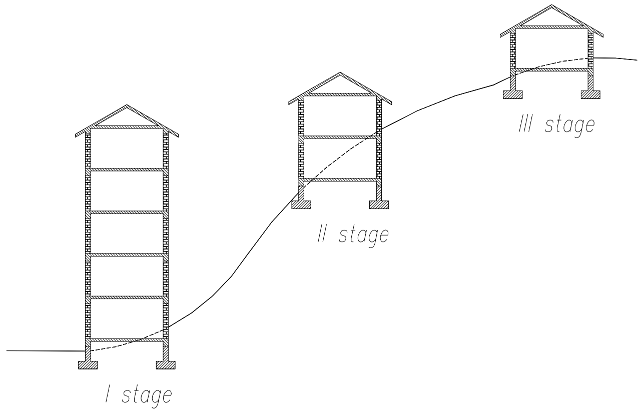

Fig. 1. *Storey number and stages of the construction on landslide terrains*

- Design and construction on the landslide endangered terrain. Here it is necessary to change the usual order of construction – first the buildings have to be erected and after that – the engineering facilities. In order to be effective and safe, the construction should be carried out in the following order: new shaping of slopes (if necessary), simultaneous laying of water supply and sewerage systems, execution of drainage facilities, construction in stages of buildings and equipment. The existing practice of constructing buildings with water supply without sewerage should be forgotten when working on landslide endangered terrains. The situation of the buildings should enhance the free runoff of surface water. The runoff backwatering by dense barriers should be forbidden. The slope undercutting is undesirable and in case of necessity the terrain should be stabilized by retaining walls (free standing or connected with the building bearing elements), anchor structures, piles, etc. The basic principle in any interference in the natural stressed and strained state is to replace the action of the missing part of the massif by stabilization, weight addition, anchoring, etc.

Except for all other activities, the landslide endangered terrains have to be afforested with suitable vegetation – tree, shrub and herbaceous species that do not require abundant watering.

The attention and measures against sliding do not end with the completion of construction on the terrain – they continue during the exploitation period too by regular revision and maintenance of the water supply, sewerage and drainage facilities.

The performance of the above mentioned activities connected with the construction and exploitation of endangered by landslides terrains can be ensured if the terrains have an owner – state or private one or an association of owners. The small holders of single real estates cannot afford the consistent realization of all measures and the construction according to the rules applied so far will only aggravate the problem, compromise the construction and lead to resource losses. The investments for the measures preceding the construction will be returned many times, especially in the case of resort construction along the seacoast, if terrains potentially endangered by landslides are included in the building.

The world experience shows that construction on inclined terrains with correct urban situation of buildings encounters fewer problems. The construction activity of the Bulgarian James Velkov, who has built two towns on the extremely steep terrain of the Tenerife Island, Spain, represents a good example in this respect. He purchased the terrain, planned and built the territory, observing all rules and regulations for construction on inclined terrains. There were no trends towards situating high and heavy buildings on top of the slope, all buildings being incorporated in the environment. The streets are with constant inclination and have an asphalt cover, the water supply and sewerage systems being preliminarily laid. High construction culture is typical for the exploitation of the buildings – there are no rearrangements and super structures in the existing buildings, there are no changes in the initial conception of the architect and the civil engineer. This is the only way for successful construction and exploitation of inclined terrains without the hazard of landslides. Very useful knowledge may be derived in this respect from world experience.

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USING THE VIRTUAL INSTRUMENTATION IN EXPERIMENTAL TESTING OF STRUCTURES

Summary: *The paper describes the main components of an virtual instrument, and the specific parameters which are used. The virtual instrument is based on a computer (hardware and software) which enables the user to simulate the most sophisticated test and measurement equipments. Development systems, with great flexibility, allow the user to create specific instruments. A lot of hardware components are available, that extend almost unlimited the applications. Complex signals can be software generated using output capabilities and computer databases can be created to store and analyze the measurements results.*
The main advantages of using Virtual Instrumentation are presented: flexibility, reliability, great precision of measurement, more complex offline signal analysis on the computer, low cost and great performance. The solutions and opportunities given by virtual instrumentation are summarized. Specific virtual instruments and measurement chains are used in experimental testing of structures.
Key words: *data acquisition system, virtual instrumentation.*

PRIMENA VIRTUALNIH INSTRUMENATA U EKSPERIMENTALNOM TESTIRANJU KONSTRUKCIJA

Rezime: Rad opisuje glavne komponente virtualnih instrumenata i određene parametre koji su korišćeni. Virtualni instrumenti su zasnovani na primeni kompjutera (hardvera i softvera) koji korisnicima omogućuju da simuliraju veoma sofisticirane testove i opremu za merenje. Razvijeni sistemi sa velikom fleksibilnošću omogućuju korisniku da stvore specifični instrument. Veliki broj hardverskih komponenti je dostupan, što pruža skoro neograničenu primenu. Složeni signali mogu biti softverski generisani koristeći mogućnosti izlaznih podataka, a kompjuterske baze podataka mogu se stvarati kroz skladištenje i analizu rezultata merenja.
Prikazane su glavne predosti primene Virtualnih Instrumenata: fleksibilnost, pouzdanost, velika preciznost merenja, , niska cena i visoke performanse. Rešenja i mogućnosti primenom virtualnih instrumenata su rezimirana. Specifični virtualni instrumenti i merenja su iskorišćeni u eksperimentalnom testiranju konstrukcija.
Ključne reči: *data acquisition system, virtual instrumentation.*

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1. INTRODUCTION

The experimental data acquisition and analysis in structural evaluation tests became easier with generalized use of personal computers in different domains, once dedicated to industrial use. More factors contributed to this evolution. In structures monitoring, used frequencies are relative low and don't require special sampling performances. Input/output interfaces with increased transfer rates have been added to PC-s, and operating systems has evolved in stability, control and performance. The development cost of measurement chain have decreased due to exponential growth of computers numbers, software applications and hardware interfaces available on the market.

The improvement of performance stability of personal computers, and also the decrease of prices made possible the use of PC-s in domains like military activity and supervising control of processes. Also, specialized software applications emerged at a reasonable cost and very good performances. The domain of structural tests in construction fits very well to these technologies, due to specific characteristics like the need of in-site evaluation, and the existence of low costs hardware and software.

Identification and modeling techniques became indispensable means of structure evaluation and testing, in the process of new solution development. Reduced scale models exposed to usual conditions are used. The measured and evaluated answer is than utilized in the design process. Using virtual instrumentation, more measurement or testing specialized devices can be replaced by only one computerized hardware measurement system, completed with different software applications. A computer with input-output specialized peripheral equipments, simulating the characteristic and functioning of a measurement instrument or system for testing or data recording, represent a virtual instrument (VI). In order to measure physical signals, virtual instruments are using signal conditioning systems, digital analog and analog digital converters, transducers and sensors.

Unlike the classic measurement systems, all acquisition, processing and analysis of measured values are realized by the computer and not by dedicated devices. Software applications replace components representing 80% of a classic measurement or testing device's circuits.

The new generation of measurement, analysis and control devices, using specialized software (LabView, TestPoint, etc.) allow the development of specific applications in the domain of the evaluation of structural behavior in constructions.

2. DATA ACQUISITION SYSTEM; VIRTUAL INSTRUMENTATION

2.1. Hardware

Measurement transducers, signal adapters, acquisition interfaces and a computer with software components are the hardware components of the acquisition system (fig.1). In VI acquisition systems, the type of measured physical signals dictates only the configuration of transducers and in some cases of signal adapters. There are specific transducers for many different applications, such as measuring temperature, velocities, accelerations, or displacements.

Because transducers could have different requirements for converting phenomena into a measurable signal, some may require excitation in the form of voltage or current. Others may require additional components and even resistive networks to produce a signal. The appropriate transducer converts the physical phenomena into electrical measurable signals. Sometimes transducers generate signals too difficult or too dangerous to measure directly with a digital acquisition device. For instance, when dealing with high voltages, noisy environments, extreme high and low signals, or simultaneous signal measurement, signal conditioning is essential for an effective digital acquisition system. Signal conditioning maximizes the accuracy of a system, allows sensors to operate properly, and guarantees safety.

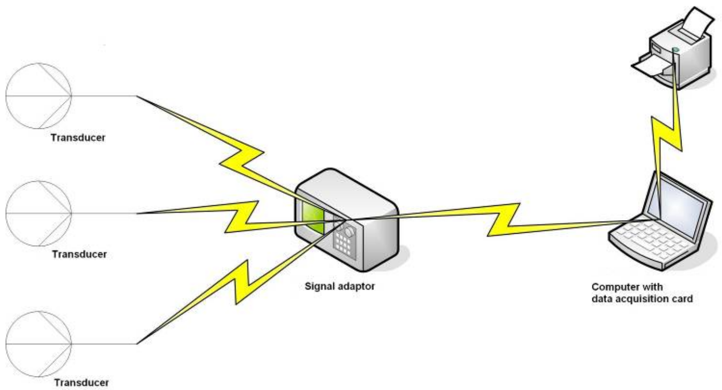

Figure 1. Data acquisition system configuration

In many systems multiplexers and converters switched from the computer zone to the signal conditioners zone, with a serial interface. The computer process a measurement points network, much more than the analogical form would have been permitted.

The open system has the following functions:

- Acquires data from the measurement transducers
- Converts analogical signal in digital signal, and processing them
- Show on-line the value of the measurement which it can store and analyze later

After the analog digital converting, the information referring to the value of measured signal is processed only with the software components.

2.2. Software

Development software running on personal computer (LabView, TestPoint) offers the user significant advantages of graphic programming. These are accessible to specialists less familiarized with the classical high language programming and more specialized in structure testing problems, working with the instruments the user are familiar to (fig.2).

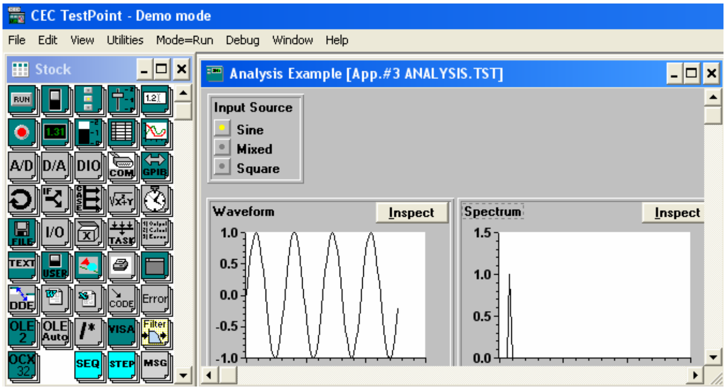

Figure 2. TestPoint user interface

The data acquisition and control system functions allow creation of simple monitoring systems of experimental tests, by using dedicated acquisition cards (ex. National Instruments). Data transmission control is being done by the mean of computer ports control specific function, or GPIB and VXI devices. Networking communication functions facilitate exchange of data with other applications or computing systems.

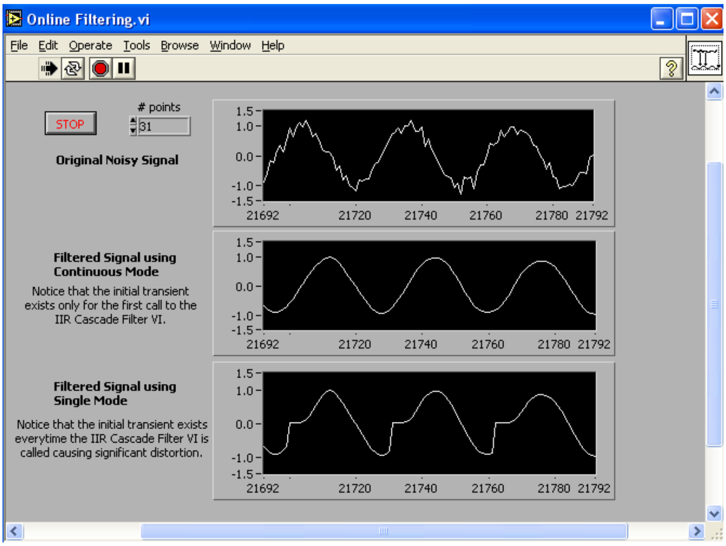

Figure 3. Signal filtering module (LabView)

In data analysis domain (statistic evaluations, algebra elements, functions for time and frequency domains, numeric filters (fig.3)), advanced directly acquired data processing modules allow real time display of monitored parameters in suggestive formats. The creation of a new database witch can be later processed, is also possible (fig.4).

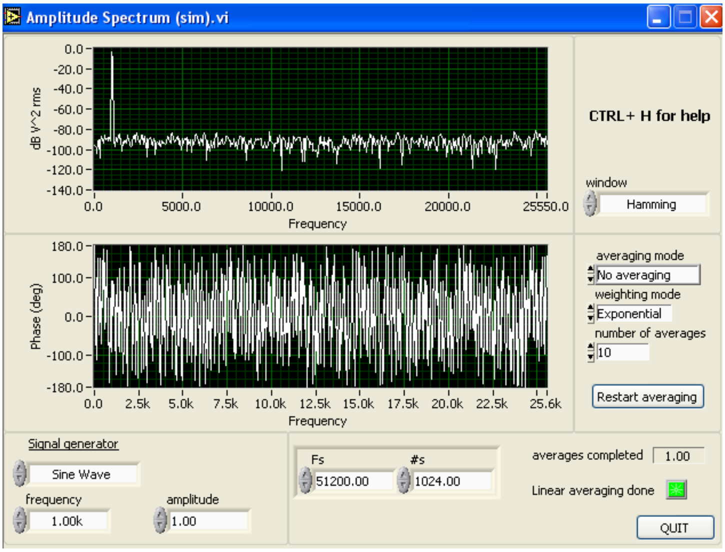

Figure 4. Signal processing application

Measurement precision is much better comparing to classical measurement systems. Further more, measured data can be saved and analyzed, compared, and post-processed. Thus, simple instruments or real portable test stands, that save measurement results directly in databases, can be created. Dedicated graphical programming media allow that the same measurement system, but with different software application, to be able to have multiple destinations, for example to be used as a voltmeter, as a data storage instrument, or as a warning system.

The acquisition system remarks itself by flexibility, because it can be easily modified and completed without needing extra costs. The functionality of the implementation is software defined by graphic programming (LabView, TestPoint), on a modular and reliable architecture. For analysis, data storage and network communication, there are available extended libraries of functions, among with PID and fuzzy control algorithms (LabView Realtime). Assembled correctly, acquisition and processing systems of the experimental data realized with virtual instrumentation can run on different configurations.

3. CONCLUSIONS

Design of the great structures, complex technical achievements, built in the last years, requires enormous resources. Exposed to a variety of environmental condition, structures have to be tested on reduced models or/and using in situ monitoring. The catastrophic events which took place recently to some structures proved the utility of testing and monitoring them, in order to reduce the unwanted effects.

Acquisition and processing data systems using virtual instrumentation can represent powerful monitoring / control tools in structures experimental tests. The acquisition system using virtual instrumentation is composed of :

- transducers for different measured signals

- conditioning blocks of signals from the measurement transducers
- data acquisition card
- computer with specialized software for signal analysis

The acquisition system structure offers the following advantages:

- enhanced flexibility of measurement system configuration
- components increased flexibility
- increased precision of information processing
- the capacity to make high complexity programming
- much lower cost compared to the usage of classic measurement and control devices

The number of measurement equipments and the programming time are substantially reduced. The acquired data and even the partial results already processed can be displayed during the tests. Thus virtual instrumentation offers the possibility to measure, overlook or control any type of parameters.

Graphical programming functions make possible a quality jump towards the use of virtual laboratories, in order to monitoring data coming from experimental tests. In conclusion, we can appreciate that software application developed in graphical programming media (Lab view, Test Point), for data acquisition systems using virtual instrumentation have a huge potential for those who work in the education, science and technology domain, not regarding their level of specialization and qualification.

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TENTH NATIONAL AND
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IT INTEGRATION IN URBAN DESIGN PROCESS

Summary: *In urban modeling activity* ***the n****ew digital methods are provided by Information technology (IT). Hardware and software development changed urban planning methods, improving the available tools for the design process.*
This article presents the project work of the students of Architecture from Iasi, designed by means of digital technology. Virtual images – both static and animated ones – describe in detail the project at the ensemble volumes level, exterior and interior spaces planning. The project emphasize relations between build or equipped volumes, relations with unoccupied spaces, valuating interior and exterior perspectives images, sunlit and darken zones. The virtual reality was simulated in exterior and interior paths, placing VR linked cameras.
It was demonstrated that in an integrated computer environment the urban design process was enhanced. The potential of planning and design process was considerably improved by using computers in representing, modeling and evaluating the changes to the built environment. Using the Internet, exchange of information between participants at design process was also possible.

Key words: information technology, urban design, virtual reality.

IT INTEGRACIJA U PROCESU URBANISTIČKOG PROJEKTOVANJA

Rezime*: Za modelitranje urbanih aktivnosti obezbeđen je novi brojni metod pomoću informacione tehnologije (IT). Razvoj hardvera i softvera menja metode urbanističkog planiranja, poboljšavajući raspoložive alate za projektovanje.*
U radu su prikazani projekti studenata arhitekture iz Iasija, pomoću digitalne tehnologije. Virtualne slike i animacije daju detaljan opis projekta na združenim prostornim nivoima, spoljašnjeg i unutrašnjeg prostornog planiranja. Projekat ističe međusobnu zavisnost između građenih ili opremljenih prostora, u relaciji sa nezauzetim prostorima, vrednujući unutrašnje i spoljašnje vizure, svetle i tamne zone. Virtualna realnost je simulirana na spoljašnjem i unutrašnjem putu, postavljajući VR povezane kamere.
Pokazano je da je u integrisanom računarskom okruženju povećan proces urbanističkog projektovanja. Potencijal planiranja i procesa projektovanja je značajno poboljšan primenom računara pri prezentovanju, modeliranju i proceni promena u građenom okruženju. Korišćenjem interneta postoji mogućnost razmene informacija između učesnika u procesu projektovanja.

Ključne reči*: informacione tehnologije, urbanističko projektovanje, virtualna realnost.*

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1. INTRODUCTION

As we all know, urban planning is a complex process with spatial, physical, social and economic aspects, which interact one with other within the urban systems. Traditional planning techniques are time consuming, focusing on experts judgment and often addressing superficial rather than fundamental issues, due to lack of resources and the poor methods used in conveying the relevant information.

The new era of information technology (IT) provided wide arrays of new digital tools that can support the urban modeling activity. Developments in both hardware and software changed urban planning methods, the tools available for the design process being considerable improved.

The computers were traditionally used in urban planning for analysis, forecasting and evaluation, from a data or modeling point of view. The advantages of computer-aided design packages in students daily practices can not be denied, computer graphics and animation playing an important role in visual presentation of students work. Particularly powerful in visualizing urban and built environments, 3D facility presents the relevant information in an intuitively comprehensive form.

Within contemporary digital environments, there are increasing opportunities for students to explore and evaluate design proposals which integrate both architectural and urban aspects. Integrated design solutions exploring buildings and their surrounding context is possible through the shared virtual environments. The complete model may be viewed in a more meaningful way either through animation, or through a total simulation using virtual reality.

2. THE CASE STUDY

In the past ten years, students in architecture and urbanism at *Technical University of Iasi* have made significant progress in applying *computer aided design* (CAD) in their work. Nowadays more then 75 % of them are able to do their final work using CAD technology without assistance from computing staff.

Developing students ability to interactively visualize projects is an important advantage of a VR approach to planning. Different stages of the urban designing process involve rapid and effective storage and retrieval of information, various kinds of visualizations. Thus the final design - evaluation process can be significantly shortened since digital information is much more easily manipulated, edited and presented.

Urban design process is essentially a creative modeling process, where students use physical models to represent the urban reality. This process may not be systematically linear, so the students must often use analogue models – hand-drawing sketches and building massive models to express their ideas. The advantage of hardware/software development – computer-based drawings, sketches and technical drawings, 3D static or animated models, - offer new ways and possibilities. Autodesk, Bentley and Graphisoft have already developed “object-oriented” CAD packages capable of performing model-based design.

Urban planning projects the students deal with, involves a lot of spatial design work. Generally speaking, there are four primary steps.

1. The first step is to create a base map from available maps using scanner, raster-vector transformation and digitizer.
2. Step two can be described as “classical” design – urban design guidelines, sketches and diagrams of design proposals – topological relationships between functions, activities and objects.
3. Step three is to create 3D models based on 2D proposals.
4. Finally, step four – the feedback – can be described as “fine tuning”. It concern the quality of design, controlling images and perspectives and modifying if necessary, the initial proposals.

This paper presents the work of the student teams – various projects for a multifunctional centre in Iasi, designed by means of digital technology (fig. 1). Virtual images – both static and animated ones – describe in detail the proposals at the ensemble volumes level, exterior and interior spaces planning.

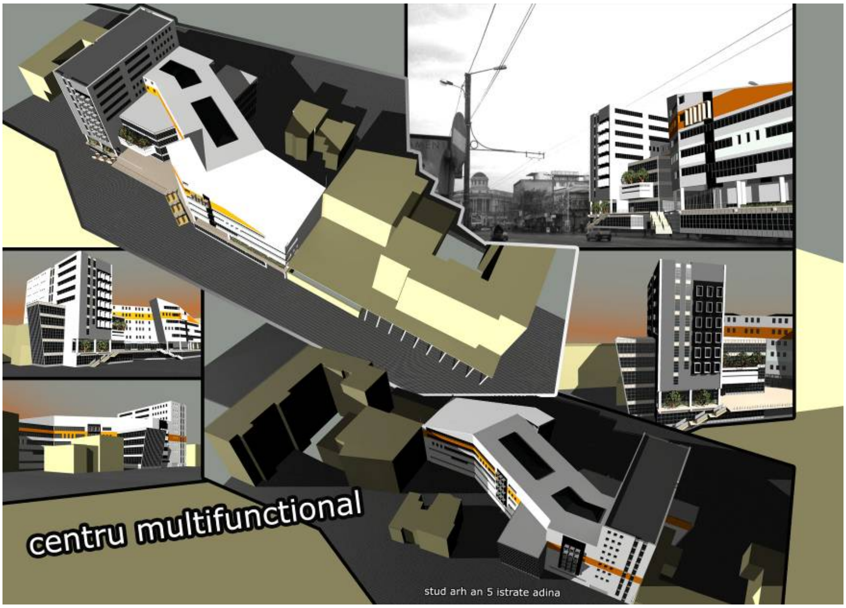

Figure 1. Multifunctional centre project – 3D images

Visual aid techniques and visual impact analysis have been used to assist students in their design and planning practices. First they used different graphic methods to display images and information within a bidimensional (2D) framework.

Then the dynamic 3D modeling has been used to assist students in studying object relationships that otherwise cannot be revealed. Influencing the urban design community, the 3D trend is fundamental for interactive urban design modeling techniques. The use of object oriented technology in CAD facilitate the support of collaborative design work, so the divergent models can be separately modified by different users, potentially using different applications. Panoramic image-based modeling, as a solution for 3D visualization, limits the viewpoints by the number of shots taken.

Figure 2. Bird eye view

The project emphasize relations between build or equipped volumes, relations with unoccupied spaces, valuating interior and exterior perspectives images, sunlit and darken zones.

Figure 3. Main hall area

The virtual reality was simulated in exterior and interior paths, placing VR linked cameras. In an integrated computer environment the urban design process could be enhanced.

The following elements were considered:

- Materials and finishing used at fronts and roofs;
- Pavement, urban furniture, plantations and green spaces for exterior;
- Partitioning, floors, ceilings, etc., for interior.

Figure 4. Street level view

VR facilitates freedom of movement within the scheme and the simulation of movement at ground level minimizes the dangers and misconceptions of bird's eye view. Presentation and comparing the alternative schemes is simplified and building elements can be selected and investigated in greater detail if needed. Most importantly, VR enhances communication channels, offering immediate feedback.

3. CONCLUSIONS

The basis of urban design is collection and analysis of current situation, so CAD technology, with spectacular 3D static or animated urban drawings, is the better choice when applying computer aided design in urban planning. The dramatically technological changes in graphical display, the ability to interactively visualize projects in context and the limited degree of visual manipulation the students can enforce are important advantages of a VR approach to planning.

It is believed that such an informational environment can enhance the planning process both within a multidisciplinary design team and during the evaluation of proposals. VR can be a valuable planning tool for evaluating the quality of residential development. Utilizing VR, students can map on the site and its surroundings census data, land use and behavioral diagrams, and other data enhancing experts assessment and most importantly reducing the need for site familiarity.

The multimedia presentations, animations, renderings, photomontages, computer animated sequences, with stop and accelerations possibilities, are inter-linked.

Hypermedia links enabled movement between different references and sources of information (including different presentation media), improving the effectiveness of the medium. Representing, modeling and evaluating the changes to the built environment using the computer play an important role in planning and design process.

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ACCESSIBILITY LEVELS OF PARTICIPANTS IN THE PROCESS OF MODELLING RESIDENTIAL ENVIRONMENT

Abstract: *The realization of the efficient partaking possibility by direct participants (so-called participation) in modelling one's own residential space and environment contains a set of elements that need prior attention. In the process, building individual family structures has significantly smaller number of limitations in comparison to building multi-family residential structures. The paper proposes possible solutions at various levels where the user of multi-family structures can qualitatively make decisions in modelling own residential environment. In the procedure of sustainable urban development and maintaining its built areas, this question has significance in protecting its spatial boundaries, as well as functional and spatial contents in longer time period.*

Key words: *users' participation, multi-family housing, base designing elements, flats adaptability, changeability capacity*

NIVOI DOSTUPNOSTI NEPOSREDNIH KORISNIKA U PROCESU OBLIKOVANJA STAMBENE SREDINE

Rezime: *Ostvarivanje mogućnosti delotvornog učešća neposrednog korisnika (tzv. participacija) u oblikovanju svog stambenog prostora i okruženja sadrži niz elemenata o kojima prethodno treba voditi računa. U tom procesu izgradnja individualnih porodičnih objekata ima znatno manji broj ograničenja u poređenju sa izgradnjom višeporodičnih stambenih objekata. U ovom radu će biti predložena mogućna rešenja na različitim prostornim nivoima na kojima korisnik višeprodičnih objekata može kvalitetno da odlučuju u oblikovanju svog stambenog okruženja. U postupku održivog razvoja grada i očuvanja njegovih izgrađenih celina ovo pitanje ima značaj u meri očuvanja njegovih prostornih granica, ali i funkcionalnih i prostornih sadržaja u dužem vremenskom periodu.*

Ključne reči: *učešće korisnika, višeporodično stanovanje, bazni elementi projektovanja, adaptibilnost stanova, kapacitet promenljivosti*

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1. INTRODUCTION

Expressive interest of a residential space user is seen in the direct participation during realization of a space. User's "participation" presents a well-known term in modern practice concerning the realization of residential space by its users. Its establishment is influenced by the nature of the process that has its levels defined by various influential factors. All these factors represent restraints of uneven intensity and volume. Moreover, special attention should be provided for multi-family housing.

Prior experiences on the territory of the Republic of Serbia were related to the practice taken over from the time of its membership within the united SFR Yugoslavia. There were unique federal regulations at the time, and within them, there was a system of dictated industry containing the principle of directed (planned) building of multi-family housing.

That building method implied producing the flats for unknown users and at the same time it presented the golden period of typical (type) organizations. The basic principle in this people's policy was to house as many people as possible, which as a consequence had standardized designing implying minimum residential content and cheap flat construction.

The product of such tendencies were also the changes in understanding the very building process, that is, the changes in applied building technologies. They aimed for a building process to be put as much as possible in the industrial plants, where the production of elements and connections for a building became faster and more qualitative, while the assembling was taken to a building site. It made the price for a flat, as a market product, lower or at least more accessible to the part of population who were enabled to participate in another form of flat providing (e.g. credits for flats).

The attempts even went as far as to raise the usability level for the most applied industrialized systems so some of the elements and connections from their production range were built without more serious restrictions.[1] Some of the systems, considering the independency of structural elements from partition elements (skeleton system and panel system with greater span of floor structure) should have been more adaptable from the point of possible interventions of indirect users within the flat, and even at the level of floors. However, all those attempts were left to self-initiating residents.

Then, the arguments started in favour of the "openness" of the system, which should have meant exactly the previously mentioned. It seems that the most of the achievements was rather a consequence of lucky circumstances than well-organized strategy. After the fall of planned industry system and appearance of market conditions, flat construction receives such character. The decrease in flat production turned the users of the industrialized flat production systems, especially "closed", into mammoth companies that slowly started to burn out. Misinterpreted possibilities of industrial production brought technological deterioration in building residential structures, since mass application was taken by improved traditional building that presented itself as adaptable in new conditions.

New approach in the flat building should be based on wider infrastructure – from designing to construction. Concretely, the starting point should be a space made to measure the user with adaptability possibilities in accordance with the life cycle of a

[1] At the time, assembling level was achieved mostly with installation block and sanitary cabin.

family or the needs of a new user. The process begins in the planning and design segments, though timely response by the industry is necessary, since it has to be able to offer on the market construction elements products that can be chosen as every other product in the area of wide consumption, at any time. It would enable direct users, with necessary consulting to a designer, to decide on spatial and functional organization of their environment.

2. INFLUENCIAL FACTORS IN DIRECT USERS' DECISION MAKING

Housing issue is wide and complex, and it is interactively connected to a lot of various segments in a society. The meaning of a flat and housing can be considered from many different aspects: social, economic, sociological, legal, psychological, spatial planning, administrative, political, and many others. Solving the need for modern housing is unimaginable without creating conditions for healthy environment for qualitative and content living. All these aspects, present in all users' appearing forms, can be referred to by the term "accessibility".

First influence refers to the level of freedom, or "accessibility level in the procedure of direct user's participation", and it depends on the type of housing. As this manner moves from the independent, individual family housing towards flat-sharing multi-family housing, so the degree of freedom for direct users who directly decide is decreasing. It is especially true of multi-storey buildings with multi-family housing where the concentration of users is significantly expressed.

The second, one could say prevailing, influence is the consequence of an economical factor, that is, the "level of accessibility to a flat" or people's purchasing power. It is a state strategy that expresses the interest in facilitating the solution to residential issue by the possibility to purchase on real estate market. Only in that situation a physical party can become a potential user, or in other words, they can qualify into the group of direct users with the right to decide.

The third influence results from the social nature of multi-family housing that is transformed into reality via a set of antagonisms by the users, or the "accessibility of sustainable relationships" – age structure, education, employment and economical status, number of household members, cultural inheritance, etc.

The fourth influence deals with designing and within its level it can be located in the field of elaborating a brief for a residential unit or, wider, it can encircle all influential factors in this process. It is important to emphasise that, in any phase of the process, for direct users, it is sufficient to have one agent that would enable them successful decision making, and that one agent is a designer. This is "accessibility to the technical level of participation".

2.1. Accessibility to direct decision making

The notion of accessibility in a wider sense refers to the degree of freedom or independency of every direct user in a residential space to make a decision that adequately satisfies set requirements. It is necessary to emphasise that this process has its limitations depending on the type of flat ownership, as well as the type of dwelling. Individual dwelling enables higher degree of freedom of decision making in comparison to multi-family housing, especially with multi-storey structures that imply a larger number of residents.

In thus organized type of housing, independent action is performed within residential unit boundaries, with the obligation to protect all the existing structural properties, supply system and exterior partitions. However, it is a general interest of all the residents in a multi-family structure to fulfil their own interest in modelling the area outside the flat, floor or intermediate surroundings. In spatial housing organization, residents can also be organized at the level of local government or a city in order to fulfil their interests.

In the area of the users' decision making at the higher spatial levels, it is necessary for social institutions to participate in the realization of the effective decision making need based on the determined rules of behaviour. Those rules of behaviour would be re-determined after public inspection and voting.

2.2. Accessibility to a flat

The fact that in all parts of Europe the accessibility to a flat is limited for those not able to purchase them on a market shows the existence of obstacles in the accessibility to a flat that can be caused by the following:

- Financial problems: the process increase for market-type solutions obviously limits the number of potential purchasers due to limited purchase possibilities;
- Legal problems: criteria for fulfilling the conditions applied for the aimed distribution of public help;
- Physical problems: for different disabled persons, physical approach to flats in normal residential blocks can cause serious problems;
- Cultural problems: minority social groups can have different expectations concerning housing and/or a wish to live closely to each other.

It is general opinion that a state intervention is unavoidable in helping people in inadequate position to come to a flat. Surveys show that dwelling in the public sector, that is, in the flats owned by a state, cities and counties, should be the most important option for the population groups in inadequate position. The number of people not able to solve their residential issue on the housing market is significantly higher than the fund of so-called social flats. In transition countries, public residential fund significantly deteriorated during 1990s, due to mass privatization. Nevertheless, market mechanisms could become useful for socially endangered or rejected, if the public sector applies them together with appropriate public subventions in a manner that all advantages provided by a market are used; appropriate regulative frame of the public policy should be applied in order to correct market imperfects.

Also, other measures concerning users, that is, consumers, and profit-makers should be considered more seriously. Namely, privatization has increased the number of "poor owners", so greater emphasise should be placed onto a general system for residential aid and the public sector aid in renovating residential buildings (which also has to be accessible to flat users with low income), and which would be a supplement to residential building programs and programs of social housing in the public sector.

Different analyses reveal that transition countries apply significantly smaller number of potentially available residential policy means than Western European countries. There are also differences in the types of measures applied: in Western

European countries there is a wide application of a combination of various methods, or in other words, mixed application of measures directed towards public and private sector, as well as towards non-governmental organizations. However, market-oriented measures are still not sufficiently applied in transition countries.

Main problems in the offer of favourable flats for “vulnerable” population groups (the young, single parents, the elder, etc.) are of financial nature. General industrial situation limits subvention possibilities, especially in transition countries. Smaller numbers of flats in the public sector, lack of financial measures, or subventions to satisfy the demands for these subventions which are too low to efficiently help endangered population categories, are some of main causes of problems in accessibility of favourable flats.

Countries utilize various temporary and permanent measures to overcome financial obstacles in the favourable flats offer: social flats and renting social flats, subvention credits and interest-free loans, subventions for renting fees, residential welfare and deposit-free loans.

Most often, local bodies deliver subventions and other forms of social welfare concerning housing vulnerable population categories. In most countries, local authorities are helped by non-governmental organizations included in the distribution of social flats and public flats for rent to the population categories in unfavourable position.

The project of the Council of Europe on human dignity and social rejection refers to the concept of “Social welfare based on three A’s” (adequacy, accessibility, affordability). The starting point of the concept is that state social welfare needs, as much as possible, to indulge the criteria of universality and safety. According to the final report of the project (HDSE (98) 1:139), three key factors in that sense are as follows:

- Adequacy – the minimally offered standard has to be high enough, and it is necessary to include a large percentage of population with the expressive need (quality and quantity should not be considered separately);
- Accessibility – legal regulations determining who has the right to social welfare;
- Affordability – purchase ability: “the ability of an individual or a household to purchase goods and services, that is, to bear purchasing expenses”.

Non-ensuring adequate, accessible and affordable flat in all cases increases the risk of social exclusion, as one of the negative aspects of today’s development. Therefore, diminishing social exclusion and eradicating poverty are issues acquiring special action in European Union countries and special attention at the global level.

Besides instruments that are directly intended for the position of population groups that are already in unfavourable position, in order to prevent the increase of these groups, overall systems of residential subventions are necessary; these widely applied systems would aid in renovating residential buildings and adequate offer of affordable flats. To help persons in unfavourable position to successfully prevail over various types of obstacles on their road towards improving their residential situation, transition countries are faced with the need to introduce more heterogeneous “policy combinations”, followed by a large scale of interventions by the private sector and non-governmental organizations in order to develop new policies and new housing approaches.

The task would be to positively promote the cooperation between various governmental levels and non-governmental and private sectors, as well as to establish basic national and local databases on social housing problems. Some professional strategies propose the establishment of national centres for monitoring housing problems in all European countries, though that question primarily depends on the orientation of certain authorities. Together with defining manners and forms of action, it is certainly necessary to determine the division of roles and responsibilities in solving housing problems of "vulnerable" citizen groups.

2.3. Accessibility to sustainable relationships

Relationship issue among the flat users in multi-family residential structures is a civilization issue, or rather cultural one. It presents itself through several characteristic aspects:

- Large economic stipulations causing migrations of different population to urban and industrial centres creating a conglomerate of population with various habits and customs, which in mutual contacts rather provoke antagonisms than lead to mutual identification (impossibility of a particular agreement);
- Uneven economic status among the residents of a same building, as an attempt to alleviate redistribution structure of different economic power possessed by population in a city territory, neutralizing expressive elitism on one side, and disabling organized activities in a building that are based on financial share of all the residential right bearers on the other side;
- Education degree as an attempt of various mutual influences among residents in order to make their system of thinking even in the process of managing and maintaining the residential structure and direct environment, as well as a certain local community, that, on the other side, can lead to mutual misunderstanding;
- Different age of residents presents a natural structure where different generations, while getting to know each other, build qualitative social relations in general; on the other side, they create significant differences in making mutual decisions concerning building maintenance and its functioning, as well as when making decisions concerning the issues related to the wider social community on the city area.

The aspiration of central and local authorities, when organizing multi-family housing, is to provide qualitative and undisturbed realization of this basic human function. It is derived from the sustainability of cities being territorially limited, while the number of population mainly increases. In such "tense" situation, black special points easily evolve.

2.4. Accessibility to the technical level of participation

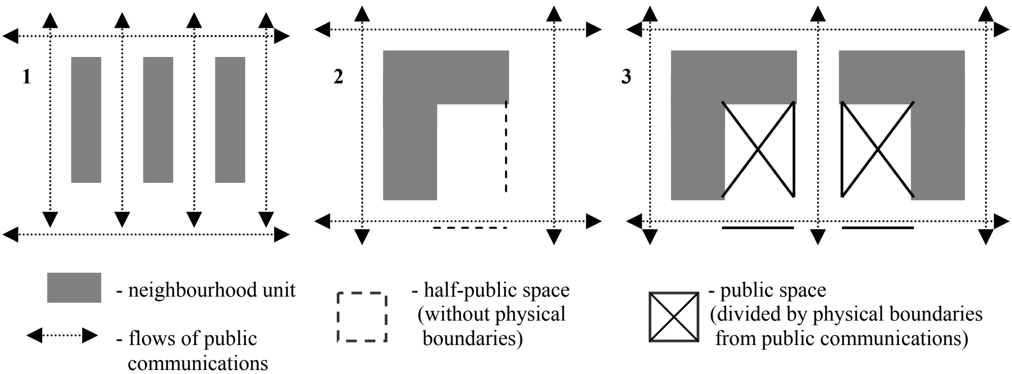
Most prior experiences concerning direct participation of a direct flat user were mainly reduced to the action level *post festum*, that is, in already spatially and functionally defined residential structure. In such situation, direct user acts in very restrained area with a set of limitations.

Modern tendencies allow the user higher degrees of freedom encircled by activities in city forums and presenting one's opinion on the integrality of certain spatial solutions (that can be related both to urban developmental policy and issues concerning these persons), as well as being involved in decisions on immediate residential space. Technical level of participation implies the possibility of direct user's involvement in decision making concerning their living space to the measure of qualitative decision making at individual and subjective level.

There are numerous restrictions in the process. One group of restrictions are the users themselves, which as individuals have to satisfy their interests in the community with other people, but also apart from them. This other, opposite side of human personality is a product of generic and cultural inheritance, education, family-acquired habits and the environment they grew up into. All together, they restrict the degree of participation freedom in the environment inhabited by a number of users.

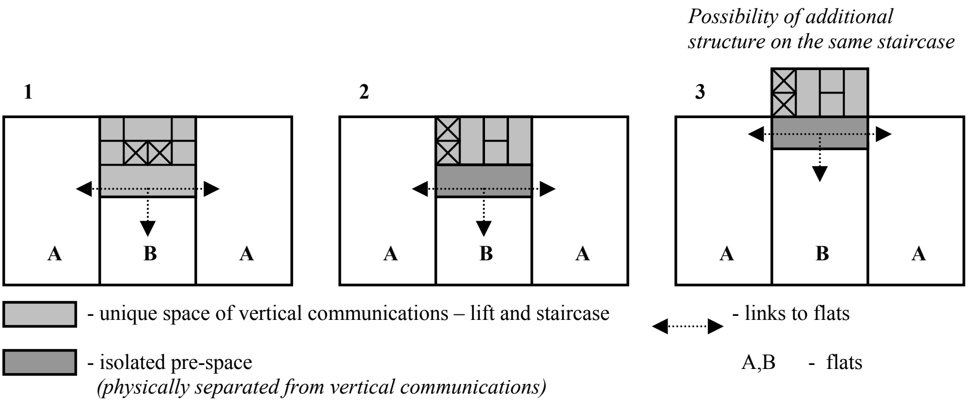
The importance and contribution of profession in such situation is to prepare adequate "working" background, which would be defined to a point that each individual solution for direct user will not influence nor invalidate the solution of another user from the immediate residential surroundings. The foundations should be prepared at different spatial levels, since these are the spaces that directly influence the housing quality. These levels determine the degree of privacy, and, as it increases, the degree of decision making freedom increases as well.

1st degree of privacy – residential neighbourhood (*direct environment*)



At this level, users make the selection of neighbourhood unit and direct environment. Degree of structure isolation and direct surrounding of public communications increases the level of user's safety.

2nd degree of privacy – residential floors



At this level, the users make a selection of residential floors and the position of vertical communication elements – lift and staircase. The degree of exclusion of lifts and staircases from residential floors increases the level of structure's isolation and safety.

3rd degree of privacy – residential unit

- entrance into the residential unit (*central position provides higher degree of organization freedom*)
- entrance pre-space
- installation block
- installation corridor

A,B – active flat space for free adaptations

At this level, the users make selection of spatial organization in a flat. In *situation 1* the selection is conditioned by the defined position of installation block, so the alterations appear in spaces A and B (traditional Voivodinian house inheritance [2]). In *situation 2* the introduction of installation corridor provides the user with the possibility to adapt entire residential space (A).

3. CONCLUSION

The degree of free intervention or active participation by a direct user in qualitative modelling of one's own residential space is conditioned at several levels, and the task of a structure is to transform these restrictions into argumentative elements in the process of analysis and solution selection. Professional and technical restrictions should be taken to the level of neutral position during subjective influence actions, that is, to the level of defining the working frame which is not a subject to change when a final solution is adopted by a direct user.

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HOUSING AND HOUSING ISSUE – OVERVIEW AND MODERN TENDENCY IN WESTERN EUROPEAN COUNTRIES

Abstract: *Housing is a wide and complex issue, interactively linked to a variety of different segments in a society. High value of a flat, together with its durability and immobility, are stressed as specific properties that distinguish a flat from other merchandizes and real estates. A flat is also an important social right in most developed countries. Right to housing is increasingly present within global policy and human rights protection and it is emphasized by many international conventions. In a lot of declarations on human rights, the fundamental nature of housing is acknowledged not just as a "right to housing", but as a right to "adequate housing". Determining the level of "adequate" housing should refer to setting the norms for housing that a society would undoubtedly consider satisfactory, and setting the standards to be achieved as a general and future social objective, including a wide scale of physical, qualitative and legal aspects. The increasing problems in financing the system of social welfare resulted in diminishing the criteria for being entitled to the right to privileges and availability of social welfare flats in many countries. The problem of financial possibilities to purchase a flat is considered as one of key problems in all European countries.*

Key words*: housing and housing issue, minimal housing, national housing policy.*

STANOVANJE I STAMBENO PITANJE – PREGLED I SAVREMENE TENDENCIJE ZAPADNIH ZEMALJA

Rezime: *Područje stanovanja je široko i kompleksno, interaktivno povezano s mnoštvom različitih segmenata jednoga društva. Velika vrednost stana, te dugotrajnost i imobilnost te nekretnine, ističu se kao specifičnosti po kojima se stan razlikuje od drugih roba. Stan je i važno socijalno pravo u većini razvijenih zemalja. Pravo na stanovanje sve je više prisutno u okviru globalne politike i borbe za ljudska prava, potvrđeno raznim međunarodnim konvencijama. U mnogim deklaracijama o ljudskim pravima temeljna priroda stanovanja priznata je, ne samo kao "pravo na stanovanje", već kao pravo na "odgovarajuće stanovanje". Određivanje nivoa "odgovarajućeg" stanovanja trebalo bi se odnositi na normiranje stanovanja koje društvo nesporno smatra zadovoljavajućim, koji standard se treba postići kao opšti i cilj društva u budućnosti, uključujući široke fizikalne, kvalitativne i pravne aspekte. Rastući problemi finansiranja sistema socijalne pomoći u mnogim su zemljama rezultirali sužavanjem kriterijuma za ostvarivanje prava na pogodnosti i dostupnost socijalnih stanova. Problem finansijske mogućnosti kupovine smatra se jednim od ključnih problema u svim evropskim zemljama.*

Ključne reči: *Stanovanje i stambena politika, minimalno stanovanje,nacionalna stambena politika.*

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1. INTRODUCTION

Housing is a part of basic human needs[1], thus imposing itself as a continual and actual research topic among scientific disciplines which consider housing as their boundary field of research, in order to achieve optimal construction form of physical structures and organization of space (in physical and spiritual sense).

Housing area is wide and complex, and interactively linked to a variety of different segments in a society. Due to its importance, housing is usually not left over to the market exclusively; rather, there is a set of different interventions by governments to direct steps in the sector and thus shape the whole system. The importance of a flat and housing can be considered from many various aspects: social, economic, sociological, legal, psychological, urban, administrative, political, and many others. Solving the needs for contemporary housing is unimaginable without creating the conditions for a healthy environment for qualitative and substantial living.

High values of a flat, together with the durability and immobility of the real estate, are stressed as specific properties that distinguish a flat from other merchandizes and real estates. Adequate to this treatment, there is a need to establish market and economic relations in the phases of flat building and utility, as well as to establish forms to manage housing funds as common goods of certain public interest which would direct measures and instruments of housing policy.

Western European countries and different institutions in international level that deal with housing issues have presented final solutions to transition countries. The fact remains that housing policy even in these developed countries, despite similar objectives, is characterized by a large diversity. Therefore, the conclusion could be that there is not a unique successful model for housing policy, but rather a number of diverse, good solutions for specific problems in specific countries, depending on the conditions and opportunities in these countries. Former housing policy in ex-socialist countries was one of the most contradictory developmental areas in those societies.

Today, in the transition countries, housing should be taken from the subsystem of urban economy and set as an important segment of market earning. It should have a special treatment since it creates preconditions for substantial alterations in all social and economic transition processes in a society. A flat presents an important social right in most developed countries. Right to housing is more and more present in global policy and human rights protection, and it is emphasized by different international conventions.

2. HOUSING AND HOUSING POLICY

Understanding and perception on the "right to housing" is one of significant factors that influence housing policy in general, even independently from the degree of its regulation in the legal system of a certain country.

There have been intensive discussions on acknowledging human rights worldwide in the last few years, including the right to housing. That right is globally accepted by United Nations and other world forums, and also incorporated into many conventions on human rights. When a country signs those international agreements, it formally presents

[1] "In the system of human needs, housing needs come on the third place, after feeding and clothing." Encyclopaedia on Economy II, Savremena administracija, Beograd, 1986, p. 121.

certain notions on the general approach to human rights in that country on the level of public policy and on the prevalent orientation concerning housing issues; however, in most cases it does not necessarily imply obligatory action by a government.

In a lot of declarations on human rights, the fundamental nature of housing is acknowledged not only as a “right to housing”, but as a right to “adequate housing”; though, at this point there are only attempts to globally define the term “adequate”. Habitat II– the United Nations conference on settlement held in Istanbul in 1996, separately stressed the “right to adequate housing”, issuing a resolution on complete and progressive realization of that right, and demanding the countries to elaborate the action plan that would include the obligation on the side of a country to help their population in solving their residential problems.

In the final document issued after the conference on human settlements Habitat II the right to adequate housing was defined as follows: “Everyone has the right to adequate shelter that is healthy, safe, available and financially obtainable, and including basic communal services, equipment and adequacies, as well as the possibility to enjoy freedom from discrimination concerning housing and legal security of the user.”

In comparison to world declarations on the rights to housing that have global and general character, European declarations are a little more concrete. In the Revised European Social Charter from 1996, the following was stated: “With a view to ensuring the effective exercise of the right to housing, the Parties undertake to take measures designed to promote access to housing of an adequate standard, to prevent and reduce homelessness with a view to its gradual elimination, and to make the price of housing accessible to those without adequate resources.”

Considering the accessibility of housing and public policy, the members of European Union, according to the report of BIPE from 2000[1], distinguished four layers of rights to housing, those being:

- right set forth by a constitution,
- right determined by legal bodies,
- existing policy on “rights to housing”,
- entitled right to housing.

3. ADEQUATE AND MINIMAL HOUSING

As a rule, each country has their own definitions for the notions of “adequate” and “minimal” housing, conditioned by different specific situation in individual countries, from financial possibilities and the level of general standard, to geographic position, way of life, etc. Those distinctions need not necessarily be lawfully unmistakeably stated and legally obligated, but rather they can be recognized in various special regulations in the area of fiscal, social or housing policies, that is, in limitable regulative prescribing the possibilities for realizing certain rights related to housing conditions. It is not rare that in this manner in one country there are different regulations that define housing norms differently, depending on the scale of proposed measures and possible users, as well as disposable means for their realization.

[1] BIPE, 2000: European public policy concerning access to housing. Project director Patrick de la Morvonnais, consultant Nayih Chentouf. BIPE, Boulogne Billancourt, September 2000

Determining the level of “adequate” housing should refer to standardizing the housing in the manner a society would undoubtedly consider satisfactory, and to naming standards that should be achieved as general and future social objectives, including wide scale of physical, qualitative and legal aspects. Defining adequate housing is not linked only to the area and equipment of a residential dwelling, but also to status issue of legal security in dwelling usage, which is essentially connected to the tradition of housing, continuity of legal system and final orientation of housing policy. In technical and technological sense, adequate level is often modelled by regulations dealing with issues concerning building new flats, that is, standards concerning flat construction, maintenance and utility.

Differently from adequate housing, “minimal housing” should imply lower housing standard from the one that a society considers as satisfactory. Minimal housing refers to the lower level tolerated by a society, though acceptable enough to satisfy basic user’s needs. Defining the level of minimal housing would mostly encircle only physical standards and legal aspects, in order to increase the necessary quality of dwelling units to, at least, such a level that would ensure minimal housing standard to all households.

In the countries in transition, these terms are often not clearly defined, nor considered by housing policy. In Western European countries, great efforts have been presented concerning social aspects of housing, though globally there still exists the need for legally obligated definitions of minimally accepted standards of housing in the European Union.

4. EUROPEAN HOUSING TRENDS

The increasing problems in financing the system of social welfare resulted in diminishing the criteria for being entitled to the right to privileges and availability of social welfare flats in many countries. The problem of financial possibilities to purchase a flat is considered as one of key problems in all European countries. 1990s brought de-monopolization and liberalization of prices and services in all parts of Europe, though by different reasons. Positive aspects of these changes where high efficiency was sought by market type solutions were overshadowed by negative aspects rising from the large increase in real estate prices as a consequence of removing the limits in market values.

Insufficient offer of financially obtainable flats for the population of transition countries is a universally acknowledged problem existing in urban areas, as well as in a considerable part of rural areas. Availability of private flats is already by ownership definition limited for people in unfavourable position; therefore, as a rule, the possibility of housing in the public sector (flats owned by a state, local governments, public institutions, etc.) is more analyzed and solutions are searched in that direction.

General conclusion is that a certain degree of public, that is state intervention, is unavoidable in order to make flats obtainable to the categories of population in unfavourable position, because the market is not able to ensure it. Public, that is state intervention can take very different forms. Usual means of analyzing public intervention is to consider residential sectors, owned and rented. When analyzing the options concerning housing of these categories of population, main topics are housing aid, fund for so-called social flats, distribution of flats owned by a state and local governments, private rent, legal frame, rent control, and legal rent security.

During few last decades, the system of ensuring flats around Europe has undergone significant changes. These changes can generally be characterized as a movement from the strategies of “offering” towards the strategies of “enabling”. The movement also includes the alternate role of governments in housing policies: decreasing direct flats offering and increasing the activities that facilitate and enable other flat providers to take action, like local communities and organizations, non-governmental organizations and private sector.

5. HOUSING POLICY PERCEPTION

The issue of housing is not unambiguously determined as recognizable as a separate industrial branch; it is a multidisciplinary sector linked to a set of various segments of social action. Therefore, when managing one consistent housing policy it is of great importance to observe the significance of that sector, and recognize disposable activity modalities in system modelling. In this sense it is necessary to promote housing, and to create assumptions that the housing policy can be incorporated into wider political actions, as an obligatory part of social and industrial activities in a society.

Problem perception and political opinion is crucial for housing policy, even independently from current disposable financial means. The knowledge on the scale and range of housing policy in the developed countries should contribute to greater conscience on the importance of the issue. Housing policy is essentially characterized by the following facts:

- Housing policy is a long-term policy;
- Housing policy requires certain means;
- Housing policy is especially important for construction as an industrial driving-wheel;
- Housing policy is important for employment and production;
- Housing policy is among the most important for the quality of living;
- Housing policy is essential for observing the perspective of one’s own children and family.

Having the perspective for the existence and prosperity followed by the possibility to solve housing needs and problems, undoubtedly presents the essential issue to all citizens, it is of special interest for every family, and also for a state and a society in general. For housing policy to become consistent and efficient, it is necessary to change the perception of understanding a dwelling and the role of a state in housing. Change of attitudes is necessary. Attitudes should be adapted to new conditions, which is not an easy or negligible effort.

6. STRATEGIC DIRECTIONS OF NATIONAL HOUSING POLICY

6.1. General objectives

Tasks concerning the definitions of national housing policy are elaborated from general developmental objectives and orientations. It is expected that partial objectives are in

accordance with the accepted resolution from the United Nations conference on Human Settlements – Habitat II. These general objectives are as follows:
• Establishing preconditions for improving housing standards and living quality;
• Reducing social endangerment, poverty and unemployment;
• Stopping the ruining of housing funds;
• Encouraging new flats building;
• Protecting and improving environment conditions;
• Increasing the quality of managing settlements in general.

6.2. Tasks for central and local governments

To fulfil strategic objectives, central government needs to take final responsibility in housing, in the context of disposability, availability, and insurance of means, quality and sustainability. Since central government models legal frame, they are also responsible for the organization of housing system.

In accordance with the above, housing policy, as a long-term strategic task and fundamental objective, should ensure the fulfilment of HOUSING AVAILABILITY AND HOUSING FUNDS DISPOSABILITY. Furthermore, it should establish full LEGAL SECURITY IN THE RENTING SECTOR with adequate social protection, and with modelling and founding social (non-profit) renting sector based on non-profit or low-profit housing organizations. Thus, renting institution can be implemented into housing area and become common and safe way of flat usability, as in most developed European countries. Central government can hardly work without local government. Therefore, local government should see the importance of housing issue for their community, and, following the needs and financial possibilities, they should find ways of helping their citizens to solve housing problems.

6.3. Strategic action directions

There are several strategic points in housing that should attract attention by housing policy, those being the following:

1) Modelling social (non-profit) renting sector and ensuring legal safety in the renting sector in general;

2) Stimulating residential building, including family houses and social rentable flats, as well as investing in the existing housing funds;

3) Developing institutional infrastructure;

4) Modelling the system of non-profit and low-profit housing organizations;

5) Housing "vulnerable" groups;

6) Improving housing funds management and maintenance;

7) Housing finance.

6.3.1. RENTING SECTOR IN HOUSING

- It should be improved to a managed housing sector that provides adequate legal security and social protection to citizens,

- Flat renting in a public housing fund should be raised to the level of set and transparent system of the so-called social housing renting, with all the attributes of social and legal protection,
- Improve the system of rental subventions,
- Protected rentals should be in accordance with the criteria for expense security,
- Set the rental register and introduce tax discipline into the renting sector,
- Solve other transition problems (protected credit users in private flats, superintendents' flats).

6.3.2. RESIDENTIAL BUILDING

- Systematically stimulate residential building, including building new multi-family and family houses, as well as investing in the existing housing funds,
- Residential building scale should be increased by e.g. 2010 to the justifiable number of dwelling units per year (determined by housing policy by central and local governments),
- Model and establish the organized system for building social rental flats,
- Develop and stimulate various housing projects and programs,
- Systematically approach to urban renovation and improvement of the existing housing funds,
- Incorporate non-profit (low-profit) housing organizations into the housing system (managing, maintaining and building residential buildings),
- Stimulate private and entrepreneurship initiative in residential building.

6.3.3. DEVELOPING INSTITUTIONAL INFRASTRUCTURE

Establish separate institute for housing as a scientific and research institution; consider aspects for establishing National housing fund

6.3.4. NON-PROFIT (LOW-PROFIT) HOUSING ORGANIZATIONS

There should be an improvement in housing cooperatives – housing cooperatives as non-profit organizations should be involved in housing as active participants in realizing housing policy, whose objective is to create preconditions for fulfilling housing needs of a range of population. Furthermore, the option of introducing other non-profit, that is low-profit housing organizations with different legal characteristics should also be considered, since it could become wider institutional help in ensuring flats disposability and availability, and it should be followed by the improvement of the system for managing housing funds.

6.3.5. HOUSING VULNERABLE GROUPS

Special attention should be presented to housing vulnerable groups of citizens (the young, single parents, the elderly, etc.).

6.3.6. MANAGING AND MAINTAINING HOUSING FUNDS

The issues of managing and maintaining buildings should be set by regulations in details – there should exist such legal instruments which could oblige all joint owners to take

measures and obligations to ensure that in a certain building the minority holds on to the conditions set by the majority.
Consider the possibilities for licensing building superintendents
Ensure adequate credit policy that would enable fluent investment maintenance of residential buildings – there is an option of introducing “guaranty fund”, that is “credit fund”

6.3.7. HOUSING FINANCE

Introduce security papers into housing market and include mortgage banks into the system of housing finance

7. CONCLUSION

The fact is that today’s globalize frames expand the stress of housing policy from the issues of flats and living quality to the quality of general living which also includes the problem of sustainability and protection of human environment.

In democratic societies, after years of promoting housing and a set of substantial housing policies that did not remain only in demagogic sphere of consideration, the result is also the increased introduction of the public into housing problems. General change in awareness and presentation of public opinion considering citizens’ aspirations concerning specific needs of their housing takes larger scale, thus undoubtedly presenting larger influence to the authorities. Hence, housing policies take more interest in hearing the public and wishes of their population, and in their effort to fulfil those wishes, they gradually leave final decisions on selecting housing place and way of housing to the individual.

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TENTH NATIONAL AND
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COMPARATIVE ANALYSIS OF CONCRETE QUALITY CONTROL ACCORDING TO PBAB'87 AND CONFORMITY CONTROL ACCORDING TO EN 206-1

Summary: *Joining contemporary trends in technical regulations means switching to uniform European norms. The paper gives for one concrete factory and one specific object, evaluating whether the prescribed requirements have been met as a part of quality control according to BAB 87 and EN 206-1, giving examples for testing compressive strength. Based on conformity control according to EN 206-1 we realized that the mentioned concrete factory would be always in initial production. The reason is non-existent quality control in the factory. Following everything mentioned it can be concluded that it is necessary for producers of concrete to elaborate quality control system in the concrete factory in cooperation with accredited laboratories in order to reach standard EN 206-1 and improve the level of their services.*

Key words: *concrete compressive strength, concrete class, strength class, conformity control, PBAB'87, EN 206-1.*

UPOREDNA ANLIZA KONTROLE KVALITETA BETONA PREMA PBAB'87 I KONTROLE SAGLASNOSTI PREMA EN 206-1 2000.

Rezime: *Uključivanje u savremene trendove razvoja tehničke regulative podrazumeva prelazak na jedinstvene usklađene evropske propise. U radu je, za jednu fabriku betona i za konkretan objekat data uporedna analiza ocene ispunjenosti propisanih zahteva, kao dela sistema kontrole kvaliteta prema važećem BAB 87 i evropskom standardu EN 206-1, na primeru rezultata ispitivanja čvrstoće pri pritisku kontrolnih betonskih uzoraka. Na osnovu kontrole saglasnosti prema EN 206-1 uvidelo se da bi predmetna fabrika betona bila konstantno u početnoj proizvodnji što delom zavisi i od kontrole kvaliteta koja nije uspostavljen na fabrici. Na osnovu svega iznetog može se zaključiti da je neophodno da proizvođači betona razrade sistem kvaliteta na fabrici betona uz saradnju sa akreditovanim laboratorijama kako bi se približili zahtevima standarda EN 206-1 i poboljšali nivo svojih usluga.*

Ključne reči: *čvrstoća betona pri pritisku, marka betona, klasa čvrstoće, kontrola saglasnosti, PBAB'87, EN 206-1.*

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1. INTRODUCTION

The Project of constructing reinforced concrete pipes use required making a concrete design and performing quality control of the concrete. Based on the available testing data the project was carried out according to the still valid JUS standard and BAB 87 on one hand, and European standards, namely EN 206-1 from year 2000, on the other. In order to offer an insight into the differences between the quality control according to PBAB 87 and EN 206-1 this paper gives their comparative analysis as well as their differences.

According to EN 206-1 concrete quality control consist of conformity control and production control. Production control takes account of the principles of EN ISO 9001. Our concrete factory are not so familliar with QMS. This is a reason why we present concrete conformity control for one project.

2. CONFORMITY CONTROL

2.1. Similarities and differences between PBAB'87 i EN 206-1.

BAB 87 defines the initial and continuous production, simillar as EN 206-1 which definites initial production and continuous production. Sampling and testing plan are verry different according to PBAB'87 and EN 206-1.

According to EN 206-1 continuous production is achieved when at least 35 test results are obtained over a period not exceeding 12 months. The test result shall be obtained from an individual specimen or the average of results when two or more specimens made from one sample are tested at the same age.

According to BAB 87 the average value of results is not considered but only all obtained results. According to EN 206-1 where two or more specimens are made from one sample and the range of the test values is more then 15 % of the mean then the results shall be disregarded unless an investigation reveals an acceptabla reason to justify disregarding an individual test value. According to EN 206-1 sampling is given in the tabele 1.

<table>
<tr><th rowspan="3">Production</th><th colspan="3">Minimum rate of sampling</th></tr>
<tr><th rowspan="2">First 50 m^3 of production</th><th colspan="2">Subsequent to first 50 m^3 of production</th></tr>
<tr><th>Concrete with production control certification</th><th>Concrete without production control certification</th></tr>
<tr><td>Initial (until at least 35 test results are obtained)</td><td>3 samples</td><td>1 / 200 m^3 or
2 / production week</td><td rowspan="2">1 / 150 m^3 or
1 / production day</td></tr>
<tr><td>Continuous (when at least 35 test results are available)</td><td></td><td>1 / 400 m^3 or
1 / production week</td></tr>
</table>

Table 1. Sampling according to EN 206-1

According to BAB’87 there are two criteria for sampling:

- 1 / 50 m^3 or 1 / 75 batches
- 1 / production day

It is evident that there is a difference in sampling which cannot be disregarded.BAB’87 does not prescribe difference between initial and continuous production.

PBAB’87 makes difference between concrete quality control of batchs and the proof of classof concrete. PBAB’87 defines batch as quantityof concrete produced over a period not exceeding 30 days. The producer is obliged to give the proof of concrete class every three months, so it was also called tri-monthly proof of concrete. Conformity control according to PBAB’87 defines also the control of samples taken on building site.

According to EN 206-1 Producer is resposible for concrete quality control. It works confromity control which is an integral part of production control. Concrete control on site is not difininate by EN 206-1. Iti is given in technical specifications.

According to EN 206-1 the characteristic compressive strength at 28 days of 15/30 cm cylinders or the characteristic compressive strength at 28 days of 15 cm cubes may be used for classification, while the PBAB’87 regulations allow other shapes of specimens as well. As for classification of concrete according to compressive strength, it is made using the results of testing 20cm cubes.

Concrete class C 16/20 is a concrete with characteristic compressive strength 16 N/mm^2 oncylindrical specimens 15/30cm or with characteristic compressive strength 20 N/mm^2 of 15 cm cubes.

BAB’87 defines the ratio between compression strengths for 20cm cube and 15cm cube as 0.95. We can say that the equivalent for C16/20 from EN 206-1 is MB 20. Equivalents can be found for other classes of concrete as well if we consider the dimensions of a cube and the difference in fractil which is 10% according to PBAB’87 and 5% according to EN 206-1.

3. PROCESSING OF TESTING RESULTS

Since production control was carried out in the laboratory of factory during the production of concrete, it was necessary to process the testing results as it was prescribed by EN 206-1. In order to do that certain requirements and procedures had to be prescribed.

If there are previous production in the factoru take neceseary testing results as initial testing. If there are no production make trial testing.

Samples were 20 cm cubes. Testing results were overcount to 15 cm cubes with corelation cefficinet.

After that it is necessary to prescribe period for conformity criteria assessment.

The value shall be taken as the estimate of the standarda deviation (σ) of the population. The validity of the adopted value has to be verified during the subsequent production.

Test results are divide into groups of 15 results.

Standard deviation of the latest 15 results (σ_{15}) does not deviate significantly from the adopted standard deviation ($0.63\sigma \leq \sigma_{15} \leq 1.37\sigma$). Where the value of σ_{15} lies outside these limits a new estimate of σ shall be determined from the last available 35 test results. If the standard deviation meets the criterion then we need to control the results of compressive strength testing to see if they meet two criteria:

Criteria 1 : $f_{cm} \geq f_{ck} + 1.48\sigma$

Criteria 2 : $f_{ci} \geq f_{ck} - 4$ N/mm^2

f_{cm} – mean of n results

f_{ci} – any individual test result

f_{ck} – characteristic compressive strength.

4. QUALITY CONTROL (PBAB '87) AND CONFORMITY CONTROL (EN 206-1)

According to demands of the project engineer the required type of concrete was MB 45. Another demand was a strict quality control of concrete, especially when it came to number of specimens to be controlled. As a result, during the production of every pipe three samples of concrete, namely 20cm cubes, were taken. Data base of test results consisted of 474 results of compressive strength testing.

Trial testing of concrete was carried out and mixture design was agreed upon and based on that MB 45 according to PBAB'87 or C 35/45 according to EN-206-1 was projected. All required tests were carried out acccording to BAB'87. After processing of testing results according to BAB'87, the same results were processed according to EN-206-1. The processing of results according to BAB'87 required quality control in batches and control of the class of concrete every three months. The processing of results according to EN 206-1 and the way they are processed is described earlier in this text.

The production of concrete pipes lasted from10th April 2005 to 26th September 2005. Quality control in batches according to BAB'87 is given in table 2, while the tri-monthly proof of type of concrete is given in table 3. According to criteria no.3, article 46 of BAB 87 it was established that the requirements for concrete were met and that it complied with projected concrete class MB 45.

	1	2	3	4	5	6	7	8	9	10
Number of specimens	30	30	30	30	30	30	30	30	30	30
Minimum strength	44.8	43.6	47.8	48.0	51.1	50.0	52.2	53.1	49.0	49.8
Average value	57.2	58.3	59.1	60.2	63.2	61.7	60.6	64.5	60.9	62.6
Standard deviation	5.1	4.8	3.7	3.2	5.0	4.9	4.2	5.2	4.7	4.3

Table 2.

	11	12	13	14	15	16
Number of specimens	30	30	30	30	30	15
Minimum strength	47.1	49.9	42.3	48.7	47.9	50.3
Average value	67.1	60.1	59.8	57.3	60.1	60.0
Standard deviation	4.0	4.0	5.7	5.9	3.0	2.9

Table 2.

	IV-VI	VII-IX
Average strength (N/mm^2)	60.4	61.8
Standard deviation (N/mm^2)	5.4	6.5
Characteristic strength (N/mm^2)	53.3	53.4

Table 3.

The processing of results according to EN 206-1 included an initial period of conformity control which lasted from 10th April to 10th July 2005. Over this period 216 samples were taken and tested, which is far greater number than 35 samples required by EN 206-1. This value shall be taken as the estimate of the standard deviation (σ) of the population. The validity of the adopted value has to be verified during the subsequent production. The initial value of standard deviation may be applied for the subsequent period during which conformity is to be checked, provided the standard deviation of the latest 15 results (σ_{15}) does not deviate significantly from the adopted standard deviation. Where the value of σ_{15} lies outside limits, a new estimate of σ shall be deermined from the last available 35 test results. If standard deviation of the last 15 results lies inside limites sampling is for continuous production, and where the standard deviation of the last 15 results exceeds lmites, the sampling rate shall be increased to that required for initial production.

	σ_{15} (N/mm^2)	σ_{35} (N/mm^2)
Group 1	2.95	7.42
Group 2	4.91	5.97
Group 3	4.13	-
Group 4	5.82	5.23
Group 5	5.23	-
Group 6	5.29	-
Group 7	4.31	-
Group 8	3.73	-

Group 9	3.82	-
Group 10	9.18	7.21
Group 11	4.20	7.17
Group 12	4.38	7.51
Group 13	2.75	5.12
Group 14	2.10	2.30
Group 15	3.95	3.10
Group 16	3.27	-
Group 17	2.55	-

Table 4

In table 4 values σ_{15} and σ_{35} are given in cases where σ_{15} is outside prescribed limits. During the initial assessment of conformity control standard deviation was 5.7 N/mm^2 for the sample of 216 test results.

5. CONCLUSION

As it was said in the beginning, this paper does not deal with the problem of quality control in the factory, which should be taken seriously in future. In this particular project a special quality control of the concrete was carried out.

Based on conformity control according to EN 206-1 we realized that the mentioned concrete factory would be always in initial production. The reason is non-existent quality control in the factory. Following everything mentioned it can be concluded that it is necessary for producers of concrete to elaborate quality control system in the concrete factory in cooperation with accredited laboratories in order to reach standard EN 206-1 and improve the level of their services.

6. REFERENCE

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ACKNOWLEDGMENT

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TENTH NATIONAL AND
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SOLUTIONS FOR CROSSING THE RAILWAY BUCHAREST – ARAD ON DN 7, AT ARAD

Summary: *The north-eastern side of the bypass road around the city of Arad was realized on the occasion of the rehabilitation of DN 7 between km 539+638 and 552+ 814, being classified as European national road and ensures the transit and detour traffic of the Arad city from the directions Deva, Oradea and Nadlac.The route of this bypass road implies crossings both with county roads and with important railways within the national railway network.The level crossing of DN 7 with the railway at km 540+248 represents a critical point in the safe and fluent unfolding of the road traffic. The traffic on the railway requires about 74 closings of the barrier leading to a duration of 180 minutes in 24 hours.The traffic on the bypass road of the Arad city is also very important making the connection with the exterior of the country by the Nadlac border.The elimination of the shortcomings occurring in this area of the bypass road can be realized through a crossing structure.The paper describes crossing solution of the railway Bucharest – Arad by realizing a passage with prestressed prefabricated beams and reinforced earth access upgrades and the particularities concerning the access to the passage generated by the existance in the close neighboring area of the railway of a crossing with a county road.*
Key words: *railway, DN7.*

REŠENJE DRUMSKOG PRELAZA BUKUREŠT – ARAD NA DN 7 KOD ARADA

Rezime: *Severoistočna strana obilaznice oko Arada, koja je izvedena povodom rehabilitacije DN7 između kilometraže 539+638 i 552+814, klasifikovana je kao evropski nacionalni put koji omogućuje tranzit i skretanje oko Arada sa pravca Deva, Oradea i Nadlaca. Pravac prelaza podrazumeva ukrštanje ovog puta kako sa seoskim putevima tako i sa važnim železničkim pravcima. Ukrštanje DN7 sa željeznicom na km 540+248 predstavlja kritičnu tačku u smislu sigurnosti i razvoja drumskog saobraćaja. Željeznički saobraćaja zahteva oko 74 zatvaranja rampi što dovodi do zastoja saobraćaja oko 180 minuti na dan. Rad prikazuje rešenje prelaza preko železničke pruge Bukurešt – Arad primenom prethodno napregnutih prefabrikovanih greda i armiranih zemljanih prilaza.*
Ključne reči: *železnica, DN7.*

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1. THE NECESSITY AND THE OPPORTUNITY OF THE INVESTMENT

The bypass road of the city of Arad has been realized due to the rehabilitation of the national road (DN) 7, between km 539 + 638 and the km 552 + 814, being marked as European National road and helping the through traffic and the bypass traffic of the city of Arad from Deva, Oradea and Nădlac.

The passing, at the same level with the railroad Arad –Bucharest, on the bypass belt Arad North, and on DN 7 at km 540 +248 respectively, represents a crucial point for the normal access and in safe conditions of the traffic on the national road (DN).

The traffic on the railroad Arad-Bucharest requires approximately 74 lowerings of the SAT type barrier, with the duration of barriers lowering of about 170 minutes in 24 hours, this resulting in important losses in exploitation daily, due to vehicle parking at the barrier.

In accordance with the information submitted by the National Roads and Bridges Administration of Timisoara, concerning the traffic in the area, information resulting from the traffic count in 2000, in the point of access no. 917 on DN 7 km 549 + 290, representing the section between km 539 + 638 and km 544 + 292, the values of the current traffic are high, not to mention the traffic for the coming year of 2020, according to the table.

Year	Annual average daily traffic (in Physical Vehicles)	Annual average daily traffic (in Passenger car-units)
2000	3 322	5 707
2005	4 195	7 059
2010	5 414	9 080
2015	6 628	11 034
2020	7 350	12 319

Table 1. Annual average traffic

The structure of the traffic is heterogenons, the weight of the heavy traffic being of major importance.

According to the "Technical standards for establishing the technical class of the public roads" passed by Art nr. 46/27.01.1992, for a period of 20 years, the concerned section fits into the 3rd technical class – road with 2 (two) lanes of medium traffic intensity, and the designed speed in the flat area being 80 km/h.

All these existing problems on the bypass belt Arad North lead to the necessity of eliminating the inconveniences, through the building of an overpass crossing over the railroad Arad – Bucharest, to ensure the free-flow of traffic, the raise of the traffic capacity , of the comfort, as well as traffic safety on the national road.

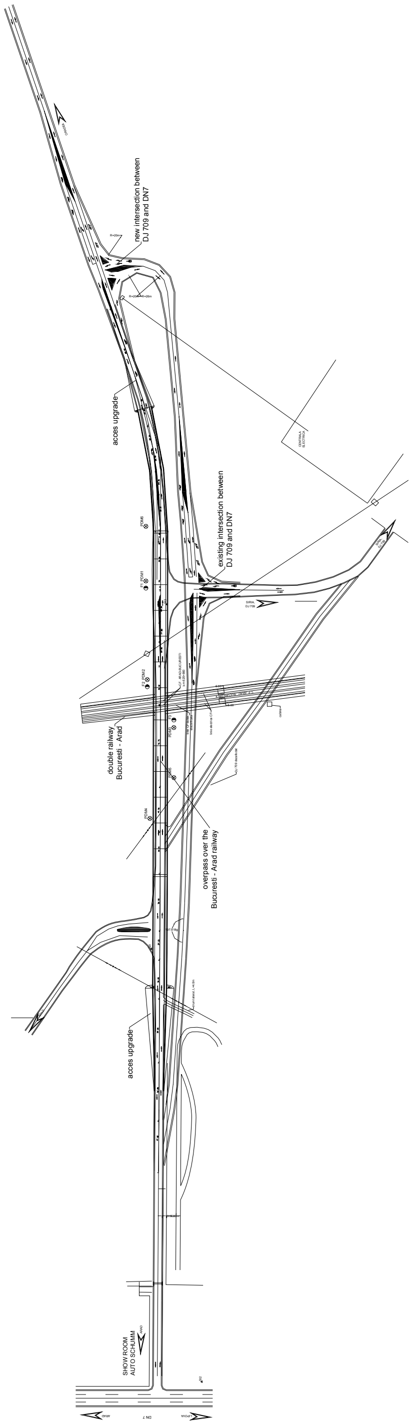

Figure 1. Layout.

2. TECHNICAL INFORMATION

Of the four variants suggested and presented in the pre-feasibility report project stage at CTE – AND Bucharest, ''variant1B'' was approved – overpass over the double railroad Arad – Bucharest, taking onto consideration the idea of doubling the railroad with two raillines, as shown in figure one.

The passage will be designed for '' E '' class concerning the loading, train of calculation A_{30} and V_{80} .

The main characteristics of the construction are the following:

- the length of the arranged route: 722.00 m;
- the length of crossing superstructure: 322.00, with 10 spans of 24 m each, with simple – supported beam structure (with 5 spans on each side of the railroad) and 3 spans of 24 m + 30 m + 24 m across the railroad, with the ensuring of the continuity on the supports; in cross section, there are 10 beams in each span;
- the length of the access upgrades: 400.00m in all, 200.00 m towards to Arad and to Oradea each, out of which: 96,00 m long – earthfilling and out of 104,00m – filling made of earth reinforced with geogrids, in the 1st variant, with retaining walls made of prefab reinforced concrete, on the same lengths, in the second variant respectively;
- the connection with DJ 709 towards Siria: 375 m, is being performed, the realization of bypass variant is necessary, during the execution works of a total of 900.00 m in length, out of which 375,00 m overlaps with the connection slip road and the rest of 525.00 m is a temporary variant that will be eliminated after the completion of the crossing.

2.1. Variant – Crossing with bicycle path

2.1.1. The Superstructure

In the event of replacing the sidewalks with bicycle paths, it is suggested that the construction of the deck in cross section be made of 10 prestressed prefab beams 1.03 m high, that communicate through a concrete overlaid bonded slab of reinforced concrete, 12…20 cm thick, as shown in figure 2.

According to STAS 2924- 91 ''Road bridges. Clearances'', art. 4.2.2.Clearances for bridges with two lanes, table 12 and art.4.4. Clearances for bridges placed on streets, with special works, table 25, the clearance elements are:

- carriageway	c = 7.00 m;
- width of the guiding lane	g_l = 2x0.50 m = 1.00 m;
- additional width (optical effect)	E_o = 2x0.50 m = 1.00 m;
- safety space	S_g = 2x0.50 m = 1.00 m;
- bicycle path	B_p = 2x1.20 m = 2.40 m;
- crossing width	L_p = 12.40 m;
- parapet kerb width	L_b = 2x0.35 m = 0.70 m;
- total crossing width	L = 13.10 m;
- carriageway cross slope:	2%

The way on the bridge will be designed taking into consideration a performant waterproofing layer with a 6 mm protection membrane, according to the standard concerning the execution and quality control of waterproofing for bridges, indicative

AND 577-2002 and bitumen pavement of type BAP 6 cm thick, according to the standard regarding the execution while hot of bitumen surfacings for the way on bridges, indicatives AND 546-1999.

2.1.2. The Infrastructure

Concerning the infrastructure, the foundations will be designed as direct concrete foundations, and the bushings and the elevations will be designed in reinforced concrete.

The abutment elevation will be designed as massive elevations in reinforced concrete, and the pier elevations as frame type piers with reinforced concrete rods, of variable height between 6.50 m and 8.50 m.

This type of elevations will contribute to the pleasant design of the construction.

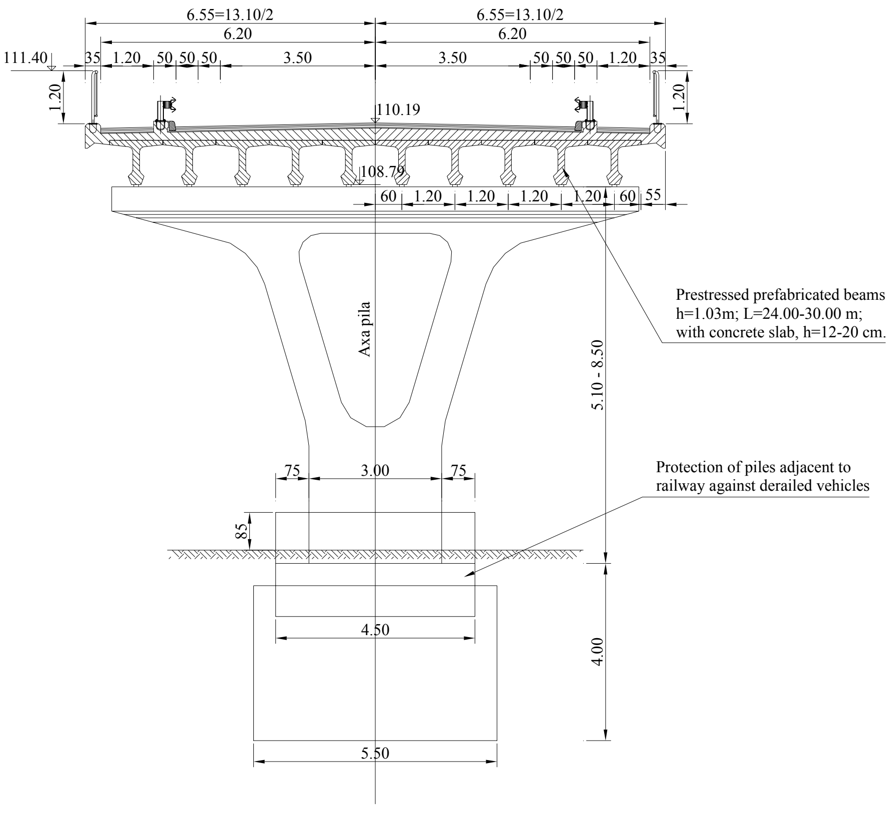

Figure 2. Cross section trough infrastructure and superstructure

2.1.3. Access upgrades

In the 1-st variant , the access upgrades are to be designed in walls of reinforced earth, with filling made of local materials with maximum dimension of 70 mm, interspersed with polyethylene geogrids, while in the second variant the upgrades are to be designed with prefab reinforced concrete fillings within retaining walls; the walls will be of 1.00m to 6.00m high, and the access upgrades up to 1.00 m high will be designed of earth fillings with natural slope, as shown in figure 3.

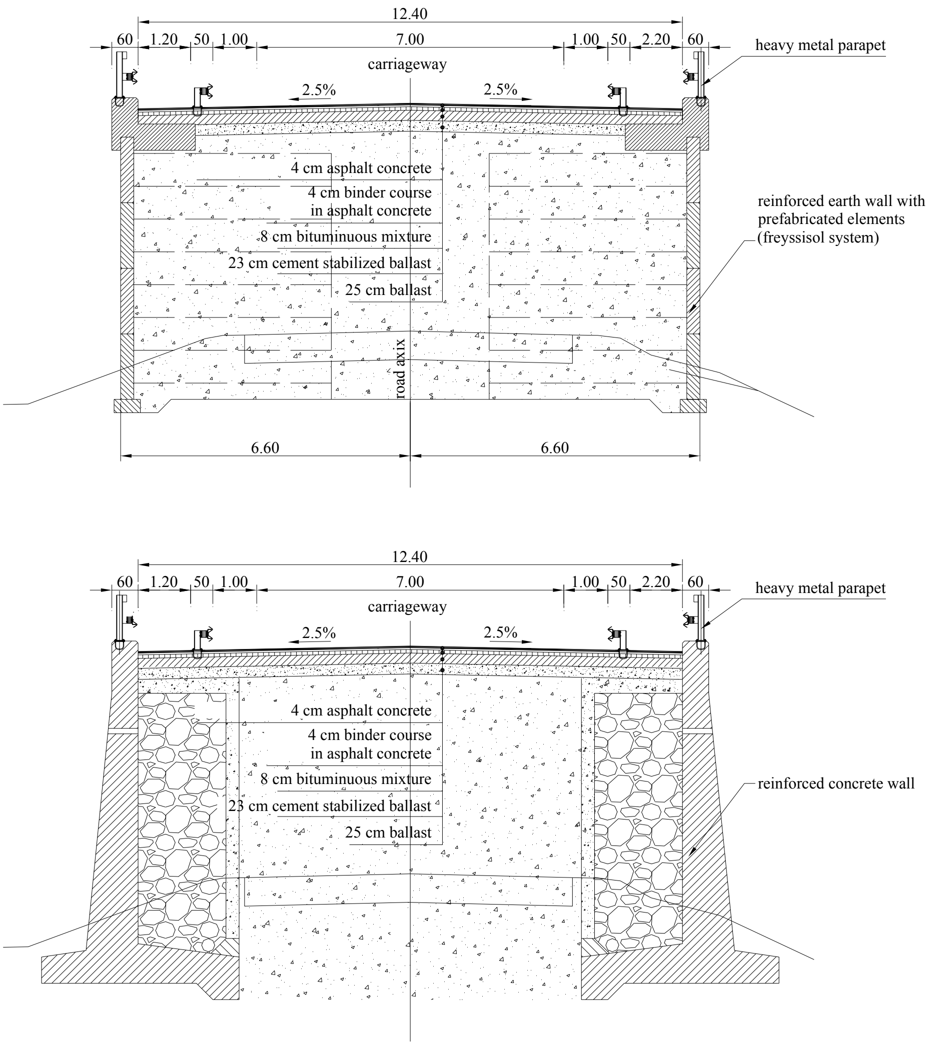

Figure 3. Cross section trough upgrades.

The joining length with the earthworks will be of 400.00m, of which 200.00 to Arad , and 200,00m to Oradea respectively.

The joining upgrades will be made of earth filling plus the pavement up to a height of 1,00 m, their length being of 96,00 m, and from 1,00 m high to the connection with the abutments, will be designed with fillings of earth reinforced with geogrids, of 104,00 m long, in the1st variant, and with fillings of prefab reinforced concrete between retaining walls, of the same length, in the second variant respectively.

The gradient of the access upgrades will be of of 4 %.

The designed pavement comprises the following layers:

- 4 cm wearing course of asphalt concrete type BA 16;
- 4cm binder course type BAD 25;
- 8 cm base course in type AB 2 mixture;
- 23 cm subbase layer of natural aggregates stabilized with cement;
- 25 cm foundation layer of ballast;

Taking into consideration the design for the bicycle path, this structure is to be applied on the entire width of the access upgrades of 12,40 m.

The drainage is made through the descending gradient of the access upgrades.

At the base of the geogrid earth walls or reinforced concrete walls, respectively, along them, stone faced drainage channel will be designed, for collecting and evacuating the water.

2.1.4. The Connection Slip Road

The execution of the connection slip road to the county road (DJ)709 to Şiria with intersection in T having an additional lane with the curve to the left.

The length of the connection slip road to the county road (DJ) 709 is 375,00 m.

Considering the traffic structure and the characteristics of the area around the connection slip road, as a result of pavement dimensions, the following layers are to be found:

- 4 cm wearing course of BA16;
- 6 cm binder course BAD 25;
- 10 cm base course of AB 2;
- 25 cm foundation layer of ballast stabilized with cement;
- 25 cm ballast layer

In cross section , the connection slip roads will have the following elements:

- 7.00 m roadway;
- 0,50 m x 2 = 1,00 m strengthened climbing lanes;
- 0,50 m x 2 = 1,00m earth shoulders.

For the drainage of the roadway of the crossing superstructure, descending of 2 % and crossfall gradients of 4 % will be designed, and on the access upgrades these will be of 2,5 % - the crossfall gradient, of 6 % - the descending gradient respectively.

On the access upgrades and on the crossing, for the protection of the users, there are present metallic parapets, both pedestrian parapets, at the side of the crossing , and safety parapets, in the delimitation area for the bicycle path on the roadway; over the railway area there are protection panels made of wire net.

The section under discussion is crossed by electrical installations which will have to be lifted and adjusted from the point of view of the diagonal traversation of the crossing, as follows:

- a line of 110 kV which crosses the future crossing right in place of the span 9, at about 55 m from the railroad, for which the access will be deviated on a distance of about 100 m , on 2 high metallic poles;

- a line of 20 kV which crosses the future crossing right in the place of span 3, on concrete poles, for which the access will be deviated on a distance of about 200 m and the diagonal crossing will be adjusted by installating 3 new poles;

- a line of 20 kV in the central area of the access upgrade of the future crossing , towards Arad, on concrete poles, for which the access will be deviated on a distance of about 80 m , on 2 new poles.

3. CONCLUSIONS

By the execution of this investment work, the grounds are created for the traffic development on the bypass road of Arad county , in safety and comfort conditions, as well as free – flow and the increas of the traffic capacity of vehicles.

At the same time , the implementation of the investment leads to the elimination of the losses in the exploitation of the vehicles, by eliminating their parking at the barrier.

The stipulations of this project aim at achieving the highest quality, resistance, stability and waterproof insulation standards as well as safety in operation and the protection of the environment.

TENTH NATIONAL AND
FOURTH INTERNATIONAL
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NOVI SAD, NOVEMBER 22-24,2006.

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THE CONSOLIDATION OF AN EXISTING REINFORCED CONCRETE BRIDGE

Summary: *There are a large number of bridges on Romanian road network that don't correspond the actual requirements due to presence of deterioration appeared in use. These distresses are caused because of the materials, the execution technologies, the methods of analysis and the loading cases used in different stages. This is the reason this bridges need to be rehabilitated and consolidated. In this paper is presented the consolidation of an existing reinforced concrete bridge situated on national road D.N.57B Oravița-Iablanița, km 55+070, over Nera river at Bozovici, Caraş-Severin county.*

The existing bridge was constructed in 1958; it was calculated at "I" design load: A13 and S60 loading class was used. The bridge static scheme is continuous girder with four openings: 20.80 m + 2 x 25.00 m + 20.80 m, with a total length of 104.00 m and a width of 10.20 m.

The solution of consolidation presented in this paper consists of coating with reinforced concrete of main and cross girders and the execution of a new overlay slab on top of the existing one. The abutments and the piers are also coated with reinforced concrete. These forseen works will increase the bridge bearing capacity accordingly to "E" design load: A30 and V80 loading class.

Key words: *Consolidation, overslab, connectors, non destructive tests, coating, reinforced concrete*

KONSOLIDOVANJE POSTOJEĆEG BETONSKOG MOSTA

Rezime: *Rešenje konsolidacije prikazano u radu sastoji se od oblaganja glavnih i poprečnih nosača armiranim betonom i izvođenja nove prelazne ploče na vrhu postojeće ploče. Obalni I ostali stubovi su takođe pojačani oblogom od armiranog betona. Ovi predviđeni radovi će povećati kapacitet nosivosti u skladu sa "E" proračunskim opterećenjem: A30 i V80 klasom opterećenja.*

Ključne reči: *Konsolidacija, prelazna ploča, konektori, nedestruktivna ispitivanja, oblaganje, armirani beton.*

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BRIDGE TECHNICAL INFORMATION

The overpass over Nera River at Bozovici, on national road DN57B, km 55+070, is realized with a reinforced concrete bridge with four openings, 20.80m+2x25.00m+20.80m, a continuous girder with constant height.

The total bridge length is 104.00m and the superstructure total length is 92.50m (figure 1). The bridge was erected in 1958; it was designed at "I" design load : A13 and S60 loading class was used.

On plan the bridge is straight, just as Bozovici platform, and Iablanita platform is on right curve. Longitudinally, the bridge is in a slightly gradient of 0.8%. The bridge is perpendicular (normal) on the direction of Nera River.

The cross section of the bridge consists of two reinforced concrete main girders with a constant height of 1.60m. The web of the girders is 60cm in width. The distance between main girders axes is 5.20m (figure1, drawing 1 and 2).

The main girders are linked together by the upper plate and by cross girders. There is a number of four cross girders in the lateral openings and five of them in the central openings. Also there is a number of five cross girders situated over the bearings. The thickness of the plate is 24cm; the connection of the plate with the main girders and the cross girders is realized chamfer edges.

On top of the abutments and the piers, the cross girders are the same in height as the main girders. In openings these cross girders are 60cm less in height. Totally there are 18 plate panels, 4 of them in lateral openings and 5 in central ones, 11 tall cross girders and 8 reduced height cross girders. The width of cross girders in openings is 20cm and 25cm over the bearings.

A 20cm thick plate is realized on the lower part of the main girders, on the lateral panels of the bridge piers. This plate is 2.15m long and is arranged on left and right of the cross girders.

The total width of footway cantilever is 2.20m (figure 2). The carriage way width is 7.80m with two footways of 1.00m each, the total width of the bridge being 10.20m (figure 2).

The cross slope is variable 1.2 … 2.5 %. The surface water drainage is realized with cast gullies arranged near the kerbs. The gullies aren't foreseen with extension tubes.

The pedestrian parapet is reinforced concrete made, heavy type with pillars, handrail and lattices.

The bridge infrastructure is realized from two abutments and three piers with direct simple concrete foundations (figure 1 and 2).

The piers have lamellar elevations with semicircular concrete front and back starlings; the seen sides are arranged with lines imitating natural stone and the support bench pad is realized from reinforced concrete (figure 1).

The abutments are massive, simple concrete made and the wing walls, the elevations and the support bench pad are reinforced concrete made.

The supports are fixed metallic tangential ones on top of the central pier and mobile reinforced concrete pendulum over the rest of the piers and over the abutments. There aren't any anti-seismic devices arranged on the infrastructures. The embankement transition zone is made with concrete pitching cone quarters. The bridge is foreseen with processed stone stairs at Iablanita abutment.

In accordance with P100-92 code, the bridge is situated in "E" seismic zone with k_s=0.12 and T_c=0.7 seconds.

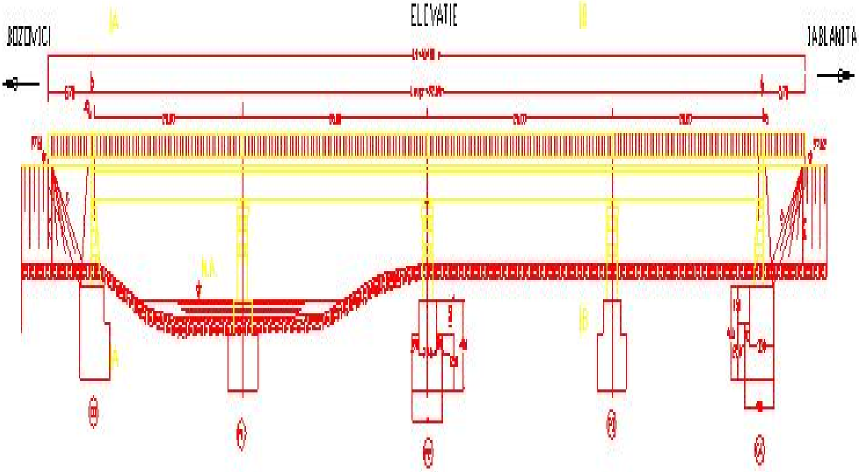

FIGURE 1

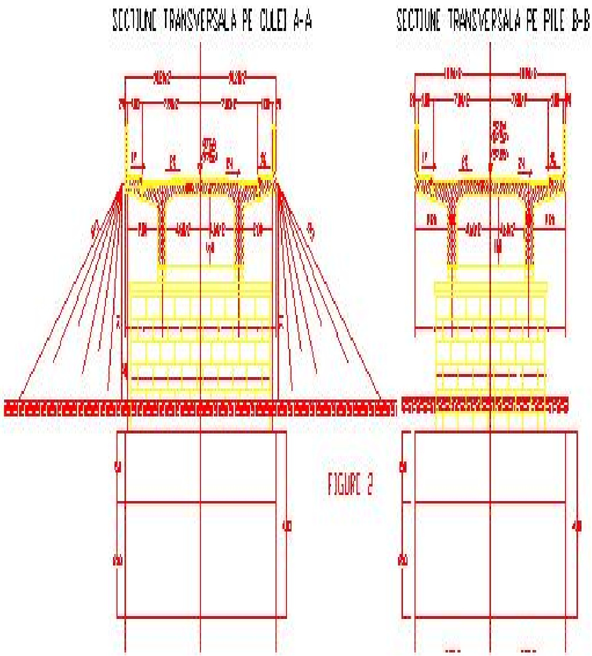

FIGURE 2

1. BRIDGE SUPERSTRUCTURE TECHNICAL STATE

At the bridge superstructure are ascertained water seepages at the lower part of the cantilever slab, at the carriageway plate and on the lateral side on the main girders. These seepages conducted to EFLORESCENTE, DRAPERII ŞI STALACTITE on the concrete surface.

The seepages are due to the deterioration of the lyre expansion joints, of the waterproofing and of the surrounding zones of the box culverts.

The superstructure bridge concrete presents an advanced carbonation grade; this is way the covering concrete layer of the reinforcing bars is destroyed and some of this re-bars are visible and corroded on large surfaces.

Due to these seepages and concrete carbonation the water has infiltrated through the expansion joints and destroyed the main beams ends; the reinforcing bars are visible.

The main beams are in an advanced state of deterioration: the cracking state isn't adequate; these cracks are more visible at the middle of the spans and over the supports.

Also the cross beams from current openings present cracks at the middle of their span. These cracks from the main and the cross girders are visible with a width of about 1mm; this is a cause of corrosion of the reinforcing bars in this beams. The cracks appeared in the beams are due to the constantly increasing vehicle loading on the bridge.

The concrete in some cross girders presents zonal segregation on large surfaces. The superstructure concrete looks friable; many zones have exfoliated concrete with reduction of cross section.

Due to the seepages in superstructure, the water reached the piers and abutments elevation producing serious degradations of the seen side of concrete and of the re-bars.

The road surface presents depressions and deteriorations; this is why the traffic conditions are hardened.

The expertise revealed that this bridge is in IV technical state class, a non satisfactory state with elements in advanced state of deterioration.

In order to decide the superstructure concrete class, non destructive tests have been conducted following the method recommended by codes, the "combined method". The strength of the concrete established in the main and cross girders have the values presented in table 1.

Analysing the results from table 1 we can conclude the following:

- The concrete strength obtained in main beams is C12/15;
- The concrete strength obtained in cross beams is C16/20;
- The superstructure of this bridge can interpreted in analyses having C12/15 concrete class.

Table 1

Element	Position	n_{medium}	v_{medium} [m/s]	Rc^{ref} [N/mm^2]	C_t	R_c [N/mm^2]		Class	
Left girder 1st span Bozovici direction middle	1	41.8	4060	29.9	1.06	31.7	23.1	C20/25	C12/15
	2	40.8	3365	16.8	1.06	17.8		C8/10	
	3	39.5	3587	18.8	1.06	19.9		C12/15	
Cross beam 1st span Bozovici direction middle	1	44	3782	26	1.06	27.6	27.6	C16/20	C16/20
Left girder 2nd span Bozovici direction support	1	39.5	3915	28.6	1.06	30.3	23.8	C20/25	C12/15
	2	37.5	3986	25.6	1.06	27.1		C16/20	
	3	33.3	4028	19.5	1.06	20.7		C12/15	
	4	30.5	3924	16.1	1.06	17.1		C8/10	
Right girder 2nd span Bozovici direction support	1	39.5	3915	21.6	1.06	30.3	23.8	C20/25	C12/15
	2	37.5	3986	23.3	1.06	27.1		C16/20	
	3	33.3	4028	19.4	1.06	20.7		C12/15	
	4	30.5	3924	18.7	1.06	17.1		C8/10	
Slab 2nd span Bozovici direction support	1	27.0	3937	18.9	1.06	20.0	20.0	C12/15	C12/15
	2	20.9							
Right girder 2nd span Bozovici direction middle	1	40.8	3789	24.0	1.06	25.4	23.3	C20/25	C12/15
	2	41.1	3701	22.9	1.06	24.3		C8/10	
	3	40.3	3526	18.8	1.06	19.9		C12/15	
Cross beam 2nd span Bozovici direction middle	1	41.9	3743	24.9	1.06	26.4	26.4	C16/20	C16/20

2. BRIDGE BEARING CAPACITY

The existing bridge was built in 1958 and was calculated at "I" design load: A13 and S60 loading class was used; it is advisable to increase this design load to "E" design load: A30 and V80 loading class.

Considering that the bridge superstructure is realized by two main beams and cross girders all casted and the concrete determined by the tests is only C12/15, it is recommended to realize an overslab of 15cm thick, minimum concrete class C18/22.5.

The necessary re-bars over the bearings can be arranged into this overslab. This new overslab will work together with the existing slab due to the connectors that will be aranged.

SECTIUNE TRANSVERSALA. CONSOLIDARE SUPRASTRUCTURA

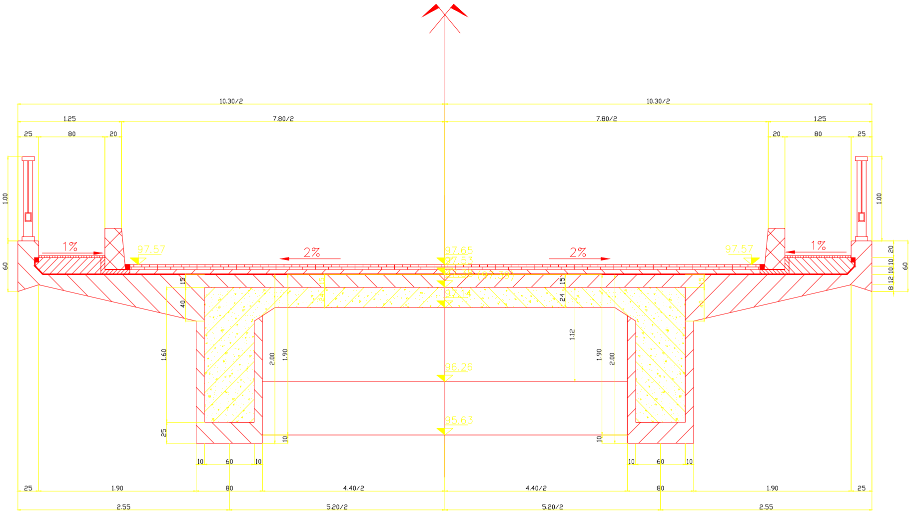

Figure 3

The solution of consolidation foreseen coating of the main and cross girders with reinforced concrete, minimum C18/22.5 class, both on the sideways and on the lower flange of the beams, 10…25 cm thick. The necessary supplementary re-bars will be arranged in these coatings and will be linked with the old structure by connectors.

For this bridge an automatic static analysis was conducted with appropriate software and was determined the new bending moments and shear forces that appear due to the increased loading design class used. The differences between “I” and “E” design load classes was translated into stresses and, further on, in reinforcing bars, necessary to be added on the structure. In table 2 and 3 are shown these differences between A13 and S60 loading class and A30 and V80 loading class. In the table only the bending moments are shown.

Section	Differences between bending moments correspondingly to “E” and “I” design load [kNm]	Differences between bending moments correspondingly to permanent loading on new and old bridge [kNm]
4	+1108.17	+1474.44
10	-1190.41	-2554.31
15	+1347.78	+1225.24
20	-1167.77	-2471.24

Table 2 – Main girders

Section	Differences between bending moments correspondingly to “E” and “I” design load [kNm]	Differences between bending moments correspondingly to permanent loading on new and old bridge [kNm]
a/2 = 5.20/2	+153.66	+101.29

Table 2 – Cross girders

The necessary reinforcing bars for these differences were determined, accordingly with effectual Romanian codes. They are presented in table 4, below:

No.	Bridge elements		The necessary reinforcing area and number of PC52 re-bars
1	Main girders	field	A_a=48.12 cm^2; 10φ25mm; A_{aef}=49.10 cm^2
		support	A_a=73.97 cm^2; 16φ25mm; A_{aef}=78.56 cm^2
2	Cross girders	field	A_a=15.15 cm^2; 5φ20mm; A_{aef}=15.70cm^2
		support	A_a=6.28cm^2; 2φ20mm; A_{aef}=6.28cm^2

Table 4

On the sideways of the main and cross beams, will be arranged reinforcing bars in the coating in order to prevent the concrete shrinkage; these re-bars will be 12mm in diameter, OB37 steel, at 15cm inter-axe distance.

Stirrups will be arranged to make the connection between the old and the new concrete; these ones will be made from OB37 steel and will be disposed at 40cm one from each other. The stirrups will over cross the new overslab, also playing the role of connectors. Between them, other stirrups will be arranged, also at 40cm distance; one arm of these stirrups will stop in the existing plate (between the main girders) and the other arm (in exterior of the main beams) will be included in the overslab. Both types of stirrups will be 12cm in diameter; the final distance between stirrups is now 20cm.

In order to take up the effect of the new shear forces that can appear in the main girders and in the cross girders, some re-bars, from the newly supplemented bars, will be lifted up on the supports.

The infrastructure of this bridge is in an advanced state of degradation, also. For this reason it is recommended the consolidation of the elevation of abutments and piers. The consolidation is realized by a reinforced concrete coated layer, minimum class C18/22.5, 10 cm thick. This coating is linked together with the old elevation by new connectors that will be arranged in drilled holes and filled with fast strengthening cement mortar or with special resins.

The foundations of the abutments and of the piers are considered to be capable to serve the consolidated bridge, no matter of the combination of loadings considered in use. If necessary, these foundations can be coated as well with reinforced concrete, minimum C16/20 class. The coating is recommended to be 40cm thick on all sides of the foundations; it will work together with the foundation block by connectors arranged in drilled holes and fastened with superior mortar cement or special resins.

This way, the whole consolidated bridge structure (both the superstructure and the infrastructure) have an increased bearing capacity, accordingly to "E" design load, A30 and V80 loading class, consonant to Romanian code STAS 3221-90; the overall dimensions of the bridge correspond with the necessary ones for a two lane, national road, just as STAS 2924-91 Romanian code prescribes.

3. CONCLUSIONS

The consolidation of the massive concrete bridges in operation by classical or modern methods are frequently used because of the differences of design load classes used in the past ("I" design load or others) vis a vis of the modern one "E" design load class: A30 and V80 loadings. This consolidation is necessary in order to increase the resistance and the functionality of the bridge according with the actual prescribed exploitation conditions.

Considering the above presented topics in this paper, we can conclude:

- All the suggested works for the rehabilitation of the bridge are necessary in order to bring all the parameters at the corresponding values of functionality, use period, comfort and safety;
- The solution of consolidation with an overslab cast above the existing concrete plate, increases the height of the main beams and, accordingly, the bearing capacity of them for the field; for the supports, the bearing capacity is increased by the re-bars arranged in the overslab;
- By arranging an reinforced concrete overslab the general rehabilitation costs are reduced;
- By coating with a reinforced concrete layer of the main girders and of the cross girders on their side ways, a protection of the reinforcing bars against corrosion is obtained.

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TENTH NATIONAL AND
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Konstantin Kazakov[1]

SOME APPLICATIONS OF THE LAGRANGE AND FOURIER FUNCTIONS IN THE SHAPE FUNCTIONS OF *2D* NORMALIZED SUPERPOSED ELASTODYNAMICAL INFINITE ELEMENT (NSEIE) BASED ON THE FINITE ELEMENT METHOD FORMULATION

Summary: *In this paper some applications of the Lagrange interpolation and the Fourier transform functions in the shape functions of normalized superposed elastodynamical infinite element (NSEIE) appropriate for an adequate Soil-Structure Interaction simulation are described and discussed. These applications are given on the formulation of the 2D horizontal infinite elements (HIE) only, but by similar techniques 2D vertical (VIE) and 2D corner (CIE) infinite element formulations can also be obtained. First the formulation of the normalized superposed elastodynamical infinite element (NSEIE) recently proposed by the author is given in brief. Then some applications of the Lagrange interpolation and the Fourier transform functions in the shape function formulation of NSEIE are presented in more details. Also the continuity along the artificial boundary (the line between a finite and an infinite element) is described and discussed in brief because of the NSEIE and also mapped elastodynamical infinite elements can directly be used in the formulation of the Finite Element Method.*

Key words: *Soil-Structure Interaction, Wave propagation, Infinite Elements, Finite Element Method, Lagrange interpolation functions, Fourier transform functions*

NEKE PRIMENE LAGRANGE-OVIH I FOURIER-OVIH FUNCIJA U FUNCIJAMA OBLIKA *2D* NORMALIZOVANIH DODTIH ELASTODINAMIČKIM BESKONAČNIM ELEMENTIMA (NSEIE) ZASNOVANIM NA FORMULACIJI METODE KONAČNIH ELEMENATA

Rezime: *U radu su prikazane neke primene Lagrange-ovih interpolacionih funkcija i Fourier-ovih funcija transformacija u funkcijama oblika normalizovanih dadatih elastodinamičkim beskonačnim elementima (NSEIE) pogodne pogodnih za adekvatnu simulaciju interakciju konstrukcija - tlo. Ove primene su date u formulaciji 2D horizontalnih beskonačnih elemenata (HIE), ali sa sličnom tehnikom 2D vertikalniim (VIE) i 2D ivičnim (CIE) beskonačnim elementima i na taj način se mogu dobiti. Prvo formulacija normalizovanih dodatih elastodinamičkihl beskonačnih elemenata (NSEIE) je nedavno predložena od strane autora i ovde je ukratko data. Zatim su, znatno detaljnije, date neke primene Lagrange-ovih interpolacionih i Fourier-ovih transformacija u formulaciji funkcija oblika NSEIE. Podduž veštačke granice (linija između beskonačnog i konačnog elementa) je prikazana i diskutovana ukratko jer NSEIE takođe beleži elastodinamički beskonačne elemente koji se direktno koriste u formulaciji Metode konačnih elemenata.*

Ključne reči: *Interakcija konstrukcija - tlo, propagacija talas, beskonačni elementi, metod konačnih elemenata, Lagrange-ova interpolaciona funcija, Fourier-ove funkcije transformacija*

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1. INTRODUCTION

In static Soil-Structure Interaction analysis, the simple truncation the far field with setting of appropriate boundary conditions gives very often reasonable computational results. However, in dynamic cases, an artificial boundary made by truncation makes results to be erroneously because reflection of waves on the boundary. In last three decades, much works has been done on unbounded domain problems and several kinds of modeling techniques have been developed to avoid these effects. Such techniques are viscous boundary, transmitting boundary, boundary elements, infinite elements and system identification method. The basic idea of these approaches is to divide the domain Ω into two parts the bounded part Ω_c and unbounded part Ω_∞, where for the first one is valid $x_i \leq c_i$, and c_i is a coordinate of an arbitrary chosen point. And for the second one is valid $x_i \geq c_i$. Also to simulate appropriate the attenuation of the waves have to set the assumption that function $u(x_i) = 0$ on Ω_∞.

Among these approaches, using infinite elements is a good way to solve Soil-Structure Interaction problems since its concept and formulation are similar to those of the Finite Element Method except for the infinite extent of the element region and shape function in one direction and there is no loss of symmetry of the element matrices. The domain Ω_∞ is partitioned into a finite number of infinite elements directly incorporated with the mesh on the bounded domain Ω_c. In the numerical models these domains are very often called a near (Ω_c) and a far (Ω_∞) field.

In this study some applications of the **Lagrange interpolation** and the **Fourier transform** functions in the shape functions of normalized superposed elastodynamical infinite element (NSEIE) are described and discussed.

2. INFINITE ELEMENT METHOD WORKS

Infinite element method was introduced about three decades ago in the original work of Bettless. Then this method have been developed and refined in many works. The original Bettless formulation is based on and also derived for the Laplace problems. This formulation is very similar to the finite element formulation except for the element domain. In this formulation infinite element domain extends toward infinity in one direction and the corresponding shape functions being non polynomial but integrable over the element. So, infinite elements are directly applicable in the Finite element method. Similar to the Finite element method, the order of the approximation and the choice of shape functions directly related to the accuracy of an infinite element. The mapped infinite elements were developed by Bettess and Zienkiewicz. A mathematically precise variational formulation of infinite elements has only recently been presented. From practical point of view infinite element can be classified into five classes:

Classical infinite elements,
Decay infinite elements,
Mapped infinite elements,
Elastodynamical infinite elements and

Wave envelope infinite elements.

The first class infinite elements are based on the original so called "classical" formulation of the infinite elements. In the decay infinite element formulation are used decay functions from different mathematical types. The mapped infinite elements are developed by using of mapping functions. These functions map the infinite domain of the element into a finite. By this approach the obtained infinite element is similar to the classical finite element. The latest researches of infinite elements are devoted to development of the elestodynamical and wave envelope infinite elements.

3. NORMALIZED SUPERPOSED ELASTODYNAMICAL INFINITE ELEMENT (NSEIE) FORMULATION

The displacement field in the elastodynamical infinite element can be described by a finite number, say *n*, of shape forms based on a finite number, say *m*, of wave propagation functions as

$$\mathbf{u}(x,z,\omega)=\sum_{i=1}^{n}\sum_{q=1}^{m}N_{iq}(x,z,\omega)\mathbf{p}_{iq}(\omega) \tag{1}$$

where $N_{iq}(x,z,\omega)$ are the standard shape displacement functions, $\mathbf{p}_{iq}(\omega)$ is the generalized coordinate associated with $N_{iq}(x,z,\omega)$, *n* is the number of nodes for the element and *m* is the number of the wave functions included in the formulation of the infinite element. For wave propagation in ξ direction the basic shape functions are:

$$N_{iq}(x,z,\omega)=L_i(\eta)W_q(\xi,\omega) \tag{2}$$

where $W_q(\xi,\omega)$ are wave functions and $L_i(\eta)$ are Lagrange interpolation polynomials which has unit value at *i*th node while zeros at the other nodes. For *HIE* infinite elements the local coordinates, *figure1* are: $\eta \in [-1, 1]$ and $\xi \in [0,\infty)$.

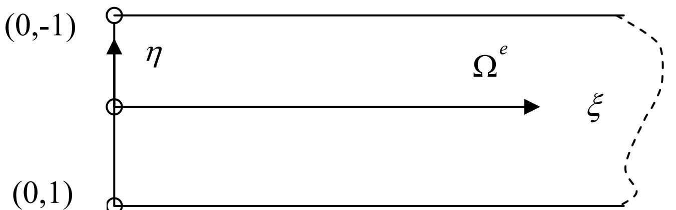


Fig. 1. Local coordinate system of horizontal infinite elements (HIE).

In the formulations of infinite elements using so-called "poles", the mid line of the element is identical with the line $\xi = 0$ in the *figure 1*.

Now, taking into account only the real parts of the wave functions the equations of the wave propagation can be written as

$$\operatorname{Re}W_q(\xi,\omega)=\cos\left(\frac{i\omega}{c_s}\xi\right)e^{-\alpha\xi} \text{ or } \operatorname{Re}W_q(\xi,\omega)=\cos\left(\frac{i\omega}{c_p}\xi\right)e^{-\alpha\xi} \tag{3}$$

where c_s , c_p are the wave velocities for *S-waves* and *P-waves* respectively, and α is a constant, called an attenuation factor.

Expanding these functions in a Fourier-like series for all wave functions included in the formulation of the infinite element the shape functions for *HIE* can be written as

$$\mathrm{Re}W(\xi,t)=\frac{1}{m}\sum_{q=1}^{m}A_q\cos\left(\frac{i\varpi q}{c_s}\xi\right)e^{-\alpha\xi},$$

$$\mathrm{Re}W(\xi,t)=\frac{1}{m}\sum_{q=1}^{m}A_q\cos\left(\frac{i\varpi q}{c_p}\xi\right)e^{-\alpha\xi} \quad \text{or} \tag{4}$$

$$\mathrm{Re}W(\xi)=\frac{1}{m}\sum_{q=1}^{m}A_q\cos\left(\frac{i\varpi q}{c_s}\xi\right)e^{-\alpha\xi},$$

$$\mathrm{Re}W(\xi)=\frac{1}{m}\sum_{q=1}^{m}A_q\cos\left(\frac{i\varpi q}{c_p}\xi\right)e^{-\alpha\xi} \tag{5}$$

where ϖ is the lowest frequency and $\omega=\varpi q$. The coefficients A_q are:

$$A_q=\int_0^{T_x}\mathrm{Re}W(\xi,t)\cos\left(\frac{i\varpi q}{c_s}\xi\right)dt \quad \text{or in the form} \tag{6}$$

$$A_q=\frac{1}{\Omega_e}\int_0^{\Omega_e}\mathrm{Re}W(\xi)\cos\left(\frac{i\varpi q}{c_s}\xi\right)dt \tag{7}$$

Using this approach a finite number of wave shape functions can be sum up as

$$N_i(x,z,t)=\sum_{q=1}^{m}N_{iq}(x,z,\omega)=L_i(\eta)\mathrm{Re}W(\xi,t) \quad \text{or} \tag{8}$$

$$N_i(x,z)=\sum_{q=1}^{m}N_{iq}(x,z,\omega)=L_i(\eta)\mathrm{Re}W(\xi) \tag{9}$$

and

$$N_i(x,z,t)\mathbf{p}_i(t)=\sum_{q=1}^{m}N_{iq}(x,z,\omega)\mathbf{p}_{iq}(\omega)=L_i(\eta)\mathrm{Re}W(\xi,t)\mathbf{p}_i(t) \tag{10}$$

$$N_i(x,z)\mathbf{p}_i=\sum_{q=1}^{m}N_{iq}(x,z,\omega)\mathbf{p}_{iq}(\omega)=L_i(\eta)\mathrm{Re}W(\xi)\mathbf{p}_i \tag{11}$$

Then equation (1) can be expressed as

$$\mathbf{u}(x,z,t)=\sum_{i=1}^{n}N_i(x,z,t)\mathbf{p}_i(t) \quad \text{or} \tag{12}$$

$$\mathbf{u}(x,z)=N_p(x,z)\mathbf{p} \tag{13}$$

The procedure described by the above equations can be treated as a superposing procedure based on a finite number of terms, the real components of the wave functions as preliminary shape functions or basis functions from mathematical point of view, and the coefficients (6) or (7) are generalized coordinates with only one component, corresponding to the node i or weight coefficients from mathematical point of view.

4. ELEMENT AND MASS MATRICES

The component matrices k_{iq} and m_{iq} can be written as

$$k_{iq}=\int_{\Omega_e}\overline{B}_i^T D\overline{B}_q\, d\Omega_e \quad \text{and} \quad m_{iq}=\left(\int_{\Omega_e}\overline{N}_i^T N_q d\Omega_e\right)I \tag{14}$$

where $\overline{B}_i=[\partial](\overline{N}_i)=[\partial](L_iW)$; $[\partial]$ is a linear differential operator matrix.

5. LAGRANGE INTERPOLATION FUNCTIONS

Lagrange interpolation functions are very often used in one dimensional problems and can be written as:

$$L_i^r(\eta)=\frac{\prod\limits_{\substack{j=1\\ j\neq i}}^{n-1}(\eta-\eta_j)}{\prod\limits_{\substack{j=1\\ j\neq i}}^{n-1}(\eta_i-\eta_j)}, \tag{15}$$

where n is the number of date points (nodes) on a line, $r=n-1$ is a rang of the polynomial and i is a date point. As was demonstrated in equation (2) ***Lagrange interpolation functions*** can be used to interpolate in η detection the shape functions for the wave propagation. When if $n=2$, then:

$$L_1^1(\eta)=\frac{(\eta-\eta_2)}{(\eta_1-\eta_2)}=\frac{1}{2}(1-\eta) \tag{16}$$

$$L_2^1(\eta)=\frac{(\eta-\eta_1)}{(\eta_2-\eta_1)}=\frac{1}{2}(1+\eta) \tag{17}$$

And if $n=3$, then:

$$L_1^2(\eta)=\frac{(\eta-\eta_2)(\eta-\eta_3)}{(\eta_1-\eta_2)(\eta_1-\eta_3)}=\frac{1}{2}\eta(1-\eta), \tag{18}$$

$$L_2^2(\eta)=\frac{(\eta-\eta_1)(\eta-\eta_3)}{(\eta_2-\eta_1)(\eta_2-\eta_3)}=1-\eta^2 \quad \text{and} \tag{19}$$

$$L_3^2(\eta)=\frac{(\eta-\eta_1)(\eta-\eta_2)}{(\eta_3-\eta_1)(\eta_3-\eta_2)}=\frac{1}{2}\eta(1+\eta)$$

(20)

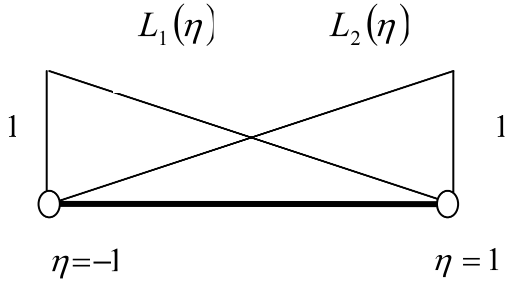
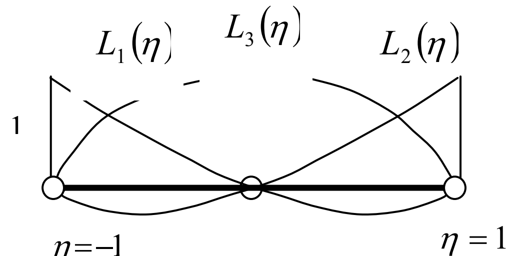


Figure 2 An illustration of expressions (16) and (17).



F illustration of expressions (1 d (20):

As was demonstrated the Lagrange interpolation functions can directly be used in the finite direction of the shape functions of the infinite element. Also the Lagrange interpolation functions can be uesed in the infinite direction when a mid, so-called bubble nodes, located on a finite distance from $\xi=0$ are used in the formulation of the infinite element.

6. GENERAL FOURIER TRANSFORMATION

Generalized Fourier transformation can be given as

$$f(x)=\int_{-\infty}^{\infty}F(\xi)T(\xi,x)d\xi \tag{21}$$

$$F(\xi)=\int_{-\infty}^{\infty}f(x)T^{-1}(\xi,x)dx$$

(22)

where in an elastodynamics variant,

$$F(\xi) = \Im_x[f(x)]\langle\xi\rangle = \int_{-\infty}^{\infty} f(x) e^{-2\pi\xi x i} dx \tag{23}$$

called the ***forward*** Fourier transformation, and

$$f(x) = \Im_\xi^{-1}[F(\xi)]\langle x\rangle = \int_{-\infty}^{\infty} F(\xi) e^{2\pi\xi x i} d\xi \tag{24}$$

called the ***inverse*** Fourier transformation.

In mechanics, we very often refer to write the transformation in circular frequency ω then oscillation frequency *f*. However, this destroys the symmetry and the resulting expressions are:

for the ***forward*** transformation

$$F(\omega) = \Im_t[f(t)]\langle\omega\rangle = \int_{-\infty}^{\infty} f(t) e^{-i\omega t} dt$$

(25)

and for the ***inverse*** transformation

$$f(t) = \Im_\omega^{-1}[F(\omega)]\langle t\rangle = \frac{1}{2\pi}\int_{-\infty}^{\infty} F(\omega) e^{i\omega t} d\omega . \tag{26}$$

To restore the symmetry of the transformations the conventions expressed as (25) and (26) is sometimes used as

for the ***forward*** transformation

$$\overline{F}(\varpi) = \Im_t[f(t)]\langle\varpi\rangle = \frac{1}{\sqrt{2\pi}}\int_{-\infty}^{\infty} f(t) e^{-i\varpi t} dt \tag{27}$$

and for the ***inverse*** transformation

$$f(t) = \Im_{\varpi}^{-1}[F(\varpi)]\langle t\rangle = \frac{1}{\sqrt{2\pi}}\int_{-\infty}^{\infty} \overline{F}(\varpi) e^{i\varpi t} d\varpi . \tag{28}$$

Since any function $f(x)$ can be split up into an even end an odd portion say $f(x) = E(x) + O(x)$, then the Fourier transformation can be expressed in terms of the Fourier trigonometric transformation as:

$$F(\xi) = \Im_x[f(x)]\langle\xi\rangle = \int_{-\infty}^{\infty} E(x)\cos(2\pi\xi x)dx - i\int_{-\infty}^{\infty} O(x)\sin(2\pi\xi x)dx \tag{29}$$

The Fourier transformation of a first derivative $f'(x)$ of a function $f(x)$ is simply related as:

$$F'(\xi) = \Im_x[f'(x)]\langle\xi\rangle = \int_{-\infty}^{\infty} f'(x) e^{-2\pi\xi x i} dx \tag{30}$$

Using integration by part expression (30) can be written as:

$$F'(\xi)=\left[f(x)e^{-2\pi\xi xi}\right]_{-\infty}^{\infty}-\int_{-\infty}^{\infty}f(x)\left(-2\pi\xi i.e^{-2\pi\xi xi}\right)dx \tag{31}$$

The first term in right side is the function $f(x)$ times by oscillating function $e^{-2\pi\xi xi}$. In this case the term vanishes. Then finaly:

$$F'(\xi)=2\pi\xi i\int_{-\infty}^{\infty}f(x)\left(e^{-2\pi\xi xi}\right)dx=2\pi\xi iF(\xi). \tag{31a}$$

7. SOME BASIC FEATURES OF MODELS WITH MAPPED INFINITE ELEMENTS

There are three basic characteristics that give some advantages of the mapped infinite elements.

- To map the infinite size of the element to a finite not necessary need a conformal mapping, because of the interest on the near field results only. So the inverse mapping is omitted.
- Once the integration on the infinite direction is done, using or not using the mapping techniques, only the finite size of the element is included in the infinite element matrix.
- The requirements of the continuity between a finite and an infinite element, i. e. on the infinite direction, is exactly the some as between the finite elements.

8. CONCLUSION

In this paper some applications of the Lagrange interpolation and the Fourier transform functions in the shape functions of normalized superposed elastodynamical infinite element (NSEIE) appropriate for an adequate Soil-Structure Interaction simulation are described and discussed. First a formulation of the normalized superposed elastodynamical infinite element (NSEIE) recently proposed by the author is given. It was demonstrated that the Lagrange interpolation and the Fourier transform functions have wide application in the shape functions of NSEIE. Also some basic chracteristics of mapped infinite element are formulated.

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TENTH NATIONAL AND FOURTH INTERNATIONAL SCIENTIFIC MEETING

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Tatjana Kočetov Mišulić[1]
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INFLUENCE OF NAIL'S NUMBER ON CYCLIC RESPONSE OF PARTICLEBOARD-SOLID TIMBER JOINTS

Summary: *The experimental investigation of sheathing-to-framing joints in shear walls under monotonic and cyclic loading protocols was made in order to obtain necessary data for modelling of inelastic response of lateral resistance of timber shear walls. The joints (timber frame - double sided particleboards - nails E28/50) were tested with different number of nails (1, 2, 3) in a row in order to compare the influence of nailing number effect on joint cyclic response. Performed experimental investigation and numerical analysis, enclosing material properties testing, were made according to EC5 and corresponding EN standards. Obtained results are discussed trough three sets of parameters- mayor events observed from envelope curves, load degradation/stiffness deterioration, and equivalent viscous damping, regarding the influence of number of nails in a row.*

Key words: ***timber shear walls, sheathing-to-framing joints, nails, hysteretic response***

UTICAJ BROJA EKSERA U REDU NA ODGOVOR VEZA IVERICA -DRVO PRI CIKLIČKOM OPTEREĆENJU

Rezime: *Eksperimentalna istraživanja veza obloge i okvira pod monotonim i cikličkim protokolima opterećenja sprovedena su u cilju dobijanja neophodnih podataka za modeliranje ponašanja – nelinearnog odgovora nosivih zidnih panela pod dejstvom seizmičkog opterećenja. Veze (monolitno drvo-dvostrano postavljena iverica-ekseri E28/50) ispitivane su sa različitim brojem eksera u redu (1,2,3) u cilju određivanja njihovog uticaja na histerezisni odgovor veze. Sprovedena eksperimentalna istraživanja veza i osnovnih materijala, kao i numeričke analize, sprovedeni su saglasno EC5 i odgovarajućim EN standardima. Dobijeni rezultati diskutovani su kroz tri grupe parametara – definisanje tačke tečenja i duktilnosti, pada čvrstoće i krutosti, kao i ekvivalentnog viskoznog prigušenja, uzimajući u obzir efekat broja eksera u redu.*

Ključne reči: ***nosivi zidni paneli,veze obloga-okvir, ekseri, histerezisni odgovor***

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1. INTRODUCTION

The huge number of residential and commercial timber frame buildings (TFB) around the world imposed the question of their safety and constant improvement. Research campaigns resulted by new knowledge and approaches, and influenced building codes. The earthquake resistance was always very actual issue, especially after great natural disasters. The seismic performance of basic structural elements of TFB - shear walls - was investigated trough appearance of new sheathing panel products, fasteners and anchors. Years of experience and investigation led to the great extend in the field of structural connectors and clear up the minimum requirements for structural detailing in seismic areas *[Ceccotti&Touliatos, 1995]*. Besides a number of structural detailing issues that should be satisfied in the conceptual design of timber shear walls (TSW) to provide good seismic performance (sheathing thickness, nailing, chord and strut design, panel proportions, anchorage requirements, etc. *[Faherty&Williamson, 1989]*), the behaviour of sheathing to framing joints under cyclic loading is recognized as basic for prediction and modelling of TSW performance *[Wakashima&Sonoda, 2001]*.

The variety of joint design, mechanical properties of local timber and sheathing materials, different types of fasteners, demand an extensive worldwide testing campaign that enables comparison of behaviour of different joint configurations. Constant need for data basis on sheathing/framing joints performances, as well as potential possibility of response generalisation, were the initial demands for several European regional research projects on this topics (Slovenia - Republic of Macedonia; Slovenia - Czech Republic; Republic of Macedonia - Serbia). All experimental testing, what results are partially discussed here, are performed at Faculty of Civil and Geodetic Eng. Ljubljana, Slovenia.

2. TESTING PROGRAM AND TEST SETUP

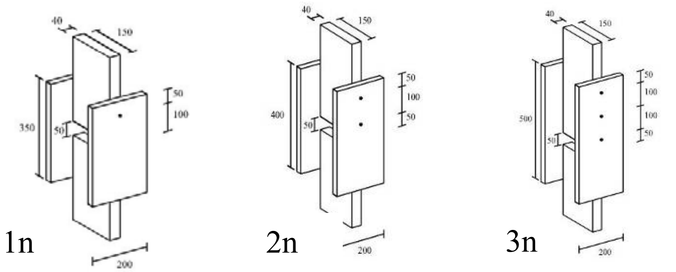
Data on monotonic and cyclic response of joints are basic and valuable source of information needed for development and calibration of inelastic computational models of TSW and TFB. Experimentally obtained load-slip loops and post failure visual inspection of tested joints are the main source of information regarding joint response under applied loadings. Connection testing was fallowed by constitutive specimens' material testing.

2.1. Material testing

All applied materials in test specimens are available on the regional market and are regularly used by wooden house producers in region. The correlation of materials properties between JUS and EN codes was proved by tests EN 408, EN 384, ISO 3131. The properties' tests were conducted on 3 samples with 15 specimens in each in order to estimate necessary data about timber strength class for ongoing tests and in general for adequate timber population. For the purpose of connection testing the coniferous 1st class timber (JUS) is estimate as S-P-F C30 (EN). The particleboard sheathing is estimated without testing, i.e. by codes' nominal characteristics. Applied 3-horizontal layers particleboard PB TP20 (JUS.D.C5.031-032) is adequate to P4 (CEN, prEN 312-4). The applied smooth nails E28/50 of Bulgarian production were certified according DIN 1151.

2.2. Joints specimens' configuration

Specimens for connection testing were made from 40/150mm C30 timber frame and P4 CEN double sided particleboards (d=1,2cm). The smooth nails E28/50 (DIN 1151) were applied on distance of 10 cm. The length of specimens depended on number of applied fasteners in a row (1, 2, 3), Fig.1, with idea to simulate the continual sheathing to framing joint in typical TSW. The tests were conducted parallel to the grain. All specimens were tested in normal weather conditions with adequate moisture control.



material:
S-P-F C30
PB (TP20) P4
fasteners:
smooth nails E28/50
number of specimens:
mono:3x2
cyclic: 3x6

Fig. 1 Connection test specimens with 1, 2, 3 nails in a row

2.3. Testing Equipment

The specimens were tested in laboratory of FCGE University of Ljubljana. The Roll Master HA hydraulic jack press, with capacity of 100 kN was used for the experimental investigation. The special holders with punch metal plates, Fig.3, were used for gripping the wooden part of the specimen, and to provide uniform distribution of acting force. Only the upper half of sheathing plate was fixed to timber frame element with tested fasteners. Nails were hammered in wood parallel to the grain direction. The lower part of board was glued to wooden element to provide higher shear resistance than upper part. Relative displacements are measured with inductive measuring devices positioned on the both sides of test specimens.

2.4. Loading protocols

Connection specimens were tested according to EN 26891, prEN 12512, prEN 1380. Applied loading protocols were monotonic and deformation controlled quasi static cyclic test for ordinary ground motion. Monotonic test was applied with loading rate of 0,2mm/s with idea to estimate the measure of max deformation capacity of the specimens and ultimate strength. Cycling load protocol consisted from two parts with loading rates of 0.1 mm/s and 0.5 mm/s. Each displacement level was repeated three times. After reaching the load-carrying limit of the joint, the test continued monotonically 'till joint failure.

3. TEST RESULTS AND COMPARISON

3.1. Monotonic test

The monotonic tests were used to determine the reference displacements and ultimate deformation capacity on the basis of average values of experimentally obtained data from three samples with 2 specimens in each, regarding to different number of nails in a row (Fig. 2, Table1). Curves 1n and 3n show good coincidence near reaching ultimate load, while 2n curve shows the trend of connection "set up" and lesser values.

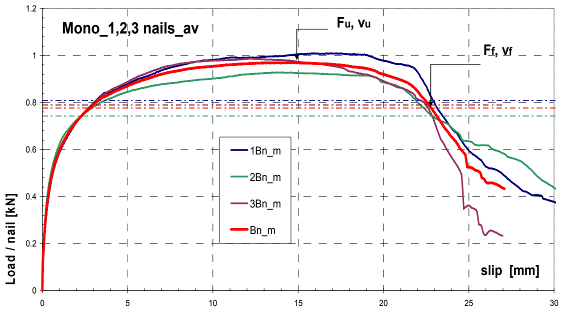


Fig.2 Effects of nail's number in on monotonic response

smp	**1n**	2n	**3n**	**av.**
F_u [kN]	1.01	0.93	0.99	0.98
v_u [mm]	16.9	13.5	12.3	14.2
F_f [kN]	0.81	0.74	0.79	0.78
v_f [mm]	23.0	23.1	22.6	22.9
v_y [mm]	0.45	0.49	0,35	0.43

Table1. Mono - reference values

3.2. Cyclic test

Hence the cycling response of joints is the main source of information regarding structural hysteretic behaviour, the typical load vs. joint slip relationships and other relations derived from hysteresis loops are presented. Each type of tested joints is presented with the average envelope calculated from hysteretic responses of three samples with six specimens with the same number of nails. Three sets of parameters obtained from the cyclic test results are chosen for detailed discussion. All sets of parameters are discussed regarding the influence of number of nails in a row.

In Fig.3 the average envelopes of all three cycles for each tested joint configuration are presented. 3n joints show decrease of load capacity and stiffness at higher magnitudes. The basic information about the parameters of average hysteresis envelopes is summarized in Table 2. The definition of Yield Limit State (YLS) and Ultimate Limit State (ULS) are shown in Fig 4. The applied definitions are adopting as suitable for prEN 12512 criterions for nonlinear curves. Tested 3n joints exhibit higher ductility, while 2n joints have good agreement with previous testing [2].

The stiffness of single hysteresis loop was calculated according Eq. 1, where F_{max} and F_{min} are peak values of the attained load in the observed positive and negative cycle and δ_{max} and δ_{min} are the corresponding joint slips. The equivalent viscous damping was calculated from hysteresis using the Eq. 2, where E_d is dissipated energy per cycle and E_p is the available potential energy :

$$K^i = \frac{F^i_{\max} - F^i_{\min}}{\delta^i_{\max} - \delta^i_{\min}} \qquad (1) \qquad\qquad \nu_{eq} = \frac{E_d}{2\pi \cdot E_p} \qquad (2)$$

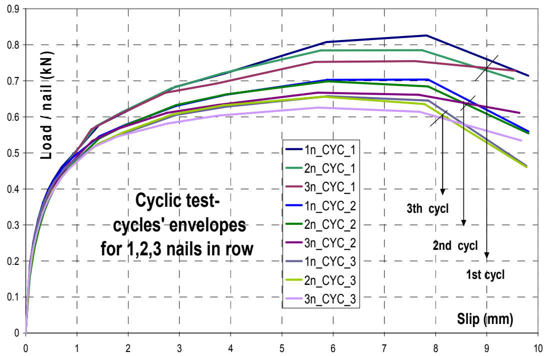


Fig. 3 Effects of number of nails in a row trough comparison of average envelopes of all cycles

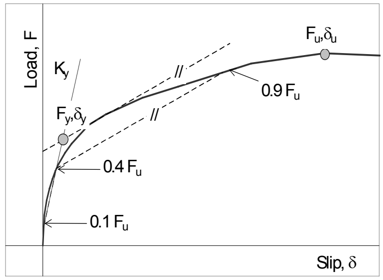


Fig. 4 Definition for Yield Limit State (YLS)

Table 2: Parameters of average hysteresis envelope (F- force, δ - joint slip) - first cycle

Name of specimen	YLS			ULS		Fy/Fu	Ductility
	Ky [kN/mm]	Fy [kN]	δy [mm]	Fu [kN]	δu [mm]		du=δu/δy
1n_cyc	1.18	0.47	0.4	0.83	7.82	0.57	19.5
2n_cyc	0.94	0.47	0.5	0.79	7.47	0.60	14.9
3n_cyc	1.2	0.36	0.3	0.75	7.6	0.48	25.3

Diagrams of load degradation, Fig. 5, have roughly linear, descending shape in all cases of tested specimens and in both comparisons of test cycles, what is especially obvious on 3n specimens. Levels of 0.95 and 0.8 of the first cycle load give information needed for the definition of YLS and ULS according to known concepts. The method for YLS determination used in this paper shows that YLS is in all the tested cases above the 0.95 limit. Stiffness deterioration curves are very similar and have shapes of power function. The initial elastic stiffness of the joint is relatively high, but after several cycles in the range of low slip amplitudes the stiffness drops to the one that defines YLS.

All stiffness degradation curves were normalized by stiffness and joint slip at YLS, Fig 7. Obtained power function from performed tests has a very good coincidence with previously suggested equation by *[Dujic&Zarnic, 2003],* where Ky and δy are stiffness and joint slip at YLS, respectively:

$$\frac{K}{K_y} = 0.8\left(\frac{\delta}{\delta_y}\right)^{-0.7} \tag{3}$$

Proposed equation (3) may be useful for the construction of "synthetic" load - slip curve for the joints with inelastic linear behaviour from the very beginning.

Equivalent viscous damping in tested connection was calculated from the first cycle and has the range between 0, 23 to 0, 13 as it was reported by *[Ceccotti, 1995]*, and *[Gatto&Uang, 2003]*.

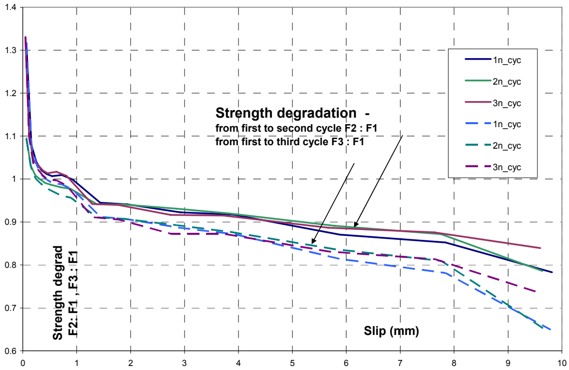


Fig. 5 Load degradation from the first to the second and third cycle for different number of nails in a row

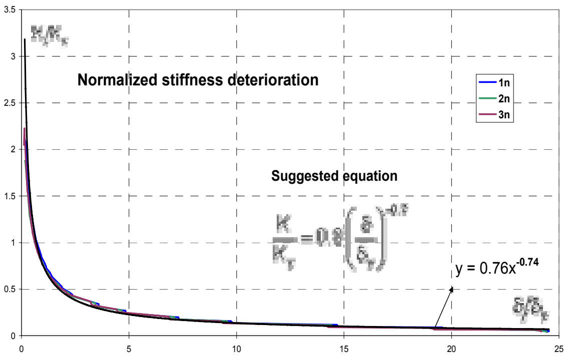


Fig. 6 Normalized stiffness deterioration

Fig. 7 Failure modes, sheathing damage and characteristic holes in timber

4. CONCLUSION

Trough analysis of obtained results it can be seen that realistic choice for test specimen is 3n model that had a good coincidence in shape of backbone curve with common 1n specimens. 2n joints exhibited good numerical coincidence, but with different curve shape. In general, four different failure modes could be observed in tested cases: (a) pull-out, (b) pull-trough, (c) tear-out and (d) fracture, Fig. 7. For monotonic and cyclic testing, fastener failure was predominantly pullout and pull through, with some tear out. Generally, all failures occurred after the last cycle defined in the load protocol.

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TENTH NATIONAL AND
FOURTH INTERNATIONAL
SCIENTIFIC MEETING

iNDiS 2006

NOVI SAD, NOVEMBER 22-24,2006.

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Milan Malenčić[3]

BASSARA DAM AND IRRIGATION PROJECT - IRRIGATION DESIGN

Summary: *Bassara Dam and Irrigation Project will be of major social and economic importance for development of the region. It will create possibilities to obtain a large volume of agricultural products, extremely necessary to supply the population with food and processing industry with raw materials. The strategic goals for agriculture in Iraq should be efficient and stable growth, increased food security and high rural employment, all to be achieved in an economically efficient, socially acceptable and environmentally sustainable manner. In the short term, the priorities should be on immediate reconstruction of damaged facilities and re-establishing key agricultural services and improving their efficiency. With the establishment of a democratic government, liberalization of the economy and opening of markets, it is very likely to contribute significantly to the country's income, food security and poverty reduction.*

Key words*: Bassara, dam, irrigation.*

PROJEKAT BASARA BRANE I SISTEMA ZA NAVODNJAVANJE

Rezime: *Projekat Basara brane i sistema za navodnjavanje biće od velikog socijalnog i ekonomskog značaja za razvoj regiona severnog Iraka. Stvoriće se mogućnosti za proizvodnju velikih količina poljoprivrednih proizvoda neophodnih za snabdevanje stanovništva hranom. Strateški ciljevi poljoprivrede u Iraku trebalo bi da budu dovoljna i stabilna proizvodnja, povećana zaposlenost i sve to u ekonomski i socijalno prihvatljivom okruženju. Ukratko, akcenat treba da bude na rekonstrukciji uništene infrastrukture i ponovnom osnivanju poljoprivrednih servisa i usavršavanju njihove efikasnosti. Uspostavljanjem demokratske vlasti, liberalizacije ekonomije i otvaranjem tržišta, veoma je verovatno da će se značajno poboljšati prihod cele zemlje, obezbeđenje hranom kao i smanjenje siromaštva.*

Ključne reči*: Basara, brana, navodnjavanje.*

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1. INTRODUCTION

1.1. Location of Bassara Irrigation System

Bassara Irrigation System is located between 34°30’ and 35°45’ north latitude and 44°30’ and 45°30’ east longitude, in the north-eastern part of Iraq. It comprises the catchment areas of Khasa Chai River, Bassara River and Tuz Chai River, which give rise to Adhaim River.The northen boundary of the region is the left watershed of Lesser Zab River. The southern boundary is the watershed of Dyala River. The eastern boundary of Region is limited by the highlands near Sulaimaniyah, while the western boundary sets the main chanel of the Kirkuk Irrigation System.

Bassara River has the largest catchment in the Region. It rises from the uplands west of the Sulaimaniya, runs throught the Bassara pass and then south-west to the Qadir Karam, then to the west to the Daquq and ultimately discharges into Adhaim River. The whole region covers 9000 km^2.The altitude ranges from 200-250m to 1500-1800m. The terrain is deforested. It is intersected by many wadies providing conditions for active erosion processes.

The boundary of the irrigation system is rather undulating due to the rough relief. The area suitable for irrigation and farming amounts to about 2600 ha. The altitude is from 600 m to 700 m. Generally, the relief of the irrigation areas is flat, slightly sloping to northeast-southeast. Dry wades and ravines intersect the ground surface. Most typical in this respect is the area of Bassara.

1.2. Hydrology

Roughly, 90 percent of the annual rainfall occurs between November and April, most of it in the winter months from December through March. The remaining six months, particularly the hottest ones of June, July, and August, are dry. During the summer period 76 mm fall or 19.9 % of the annual sum while the remaining 305 mm or 80.1 % fall during winter. The annual precipitation is small and irregularly distributed over the seasons. The period of the active crop vegetation is completely dry. Under these conditions only either crops i.e. wheat, barley and some winter vegetables can be grown without irrigation.The highest mean monthly temperatures occur during July, 35.4°C and August 34.9°C. It is characteristic that the period of highest temperatures, i.e. from April to October coincides with the dry weather, thus increasing the harmful effect of the high temperatures on the planet.

The combination of rain shortage and extreme heat makes much of Iraq a desert. Because of very high rates of evaporation, soil and plants rapidly lose the little moisture obtained from the rain, and vegetation could not survive without extensive irrigation. Some areas, however, although arid do have natural vegetation in contrast to the desert.

All available meteorological data were processed: precipitation, air temperature, relative air humidity, winds and evaporation from free water surface. The evapotranspiration of the crops to be grown in Bassara Irrigation Field is calculated after the Thornthwaite, FAO 24 Pan, Hargreaves and Turc methods. Accordingly to all available data following results were obtained and given in Table 1 and graph of evapotranspiration. After comparing the results, the FAO 24 Pan method was adopted as the most acceptable.

Table 1. Calculated evapotranspiration

	Thornthwaite	FAO 24 Pan	Hargreaves	Turc
January	5.1	36.5	41.1	25.2
February	9.5	48.2	48.7	40.6
March	20.6	77.5	67.5	67.6
April	59.7	106.1	105.9	94.8
May	113.8	177.8	139.2	143.2
June	217.7	204.5	140.8	180.4
July	268.3	235.0	144.3	224.9
August	236.3	206.0	137.6	221.3
September	153.7	164.0	103.1	140.3
October	84.3	119.1	79.2	86.8
November	32.5	61.7	46.4	44.3
December	4.9	37.3	42.4	23.6
Total (mm)	**1,206.40**	**1,473.70**	**1,096.30**	**1,292.90**

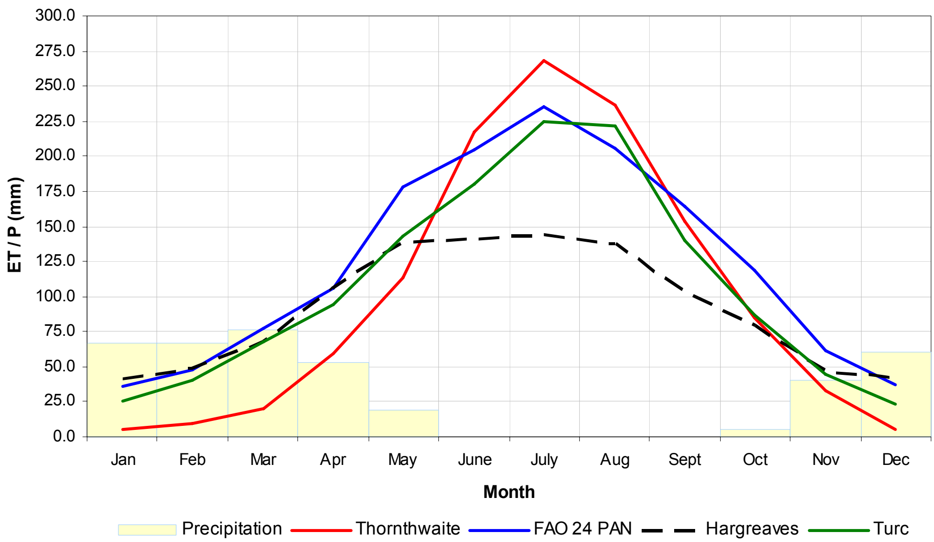

Figure 1. Comparison of Precipitation and Evapotranspiration (mm) after the Thornthwaite, FAO 24 Pan, Hargreaves and Turc method

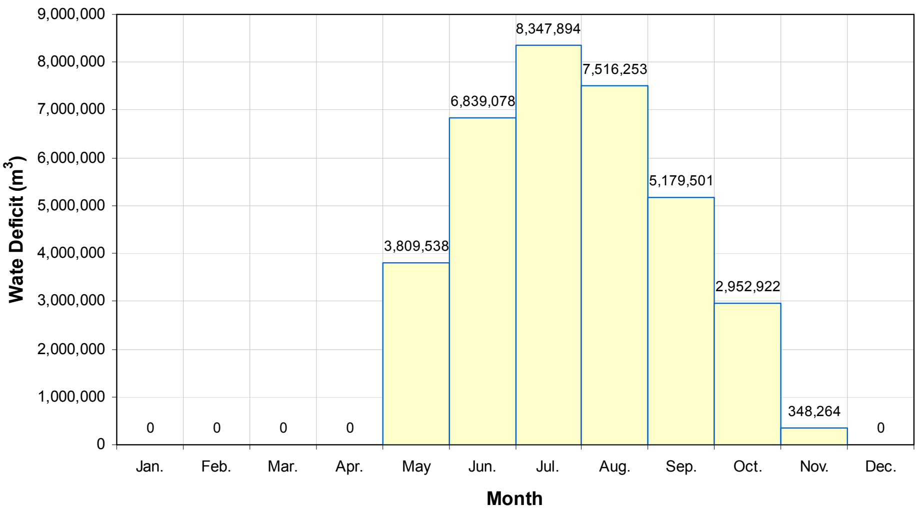

Figure 2. Calculated water deficit (Gross)

1.3. Pedology

The area of Bassara Irrigation System is predominantly flat, situated between mountain chains representing a structural part of the Alpine-Himalayan belt having a north-west-south-east direction. Bassara irrigation area consists mainly of conglomerates with calcareous cementation, intercalated with fine-grained sandstones. The rocks are compact, hard. The above-described materials consisting the parent rock of the soils, have conditioned the mineralogical and chemical composition of the soils.

Generally, the soils in Bassara irrigation area have a humus horizon 35 to 55 cm deep containing a satisfactory quantity of organic matter. The soils are not saline; water-soluble salt content is maximum. This indicates that no potential danger of soil salinization exists under dry farming or irrigation with non-saline water.

The soil in irrigation area is suitable for irrigation. The main limiting factors for the soil fertility are the arid climate and the high carbonate content in the soils, resulting from the arid climate. Nevertheless, on the grounds of the existing natural and soil conditions a considerable improvement of the soils and their fertility is expected under intensive irrigation faming.

1.4. Agrarian Economy

Since the beginning of recorded time, agriculture has been the primary economic activity of the people of Iraq. In 1976, agriculture contributed about 8 percent of Iraq's total Gross Domestic Product (GDP), and it employed more than half the total labor force. In 1986, despite a ten-year Iraqi investment in agricultural development that totaled more than US$4 billion, the sector still accounted for only 7.5 percent of total GDP, a figure that was predicted to decline. In 1986, agriculture continued to employ a significant portion--about 30 percent--of Iraq's total labor force. Part of the reason the agricultural share of GDP remained small was that the sector was overwhelmed by expansion of the oil sector, which boosted total GDP.

Iraq's agriculture has a lot of potential for growth, given the right support and policy environment. With the establishment of a democratic government, liberalization of the economy and unshackling of markets, it is very likely to contribute significantly to the country's income, food security and poverty reduction.

Bassara Dam and Irrigation Project will be of major social and economic importance for development of the region. It will create possibilities to obtain a large volume of agricultural products, extremely necessary to supply the population with food and processing industry with raw materials.

The average yields and costs per hectare determine one of the most important factors in agricultural production – Income per Hectare.

Table 2. Income per Hectare

	Area (ha)	**Income from irrigation**	**Total Production Cost**	**Net Income From Irrigation**	**Total Income**
Crops		**USD/ha**	**USD/ha**	**USD/ha**	**USD**
I. Main crops					
Wheat	850	66.08	15.25	50.83	43,208.16
Barley	450	54.90	12.59	42.31	19,041.30
Cotton	500	20.40	23.07	-2.67	-1,337.23
Sunflower	450	83.16	35.79	47.37	21,316.89
Sesame	100	65.66	255.17	-189.51	-18,951.01
Tomatoes	75	143.00	264.35	-121.35	-9,100.96
Onion	75	136.00	88.48	47.52	3,563.73
Other vegetables	100	90.00	87.88	2.12	212.48
Total	**2600**	**659.20**	**782.57**	**-123.37**	**57,953.36**
II. Second crops					
Maize	540	440.00	31.40	408.60	220,644.36
Tomatoes	180	182.00	265.24	-83.24	-14,983.47
Broad beans	100	541.67	51.99	489.68	48,968.05
Carrots	25	136.00	88.31	47.69	1,192.31
Other crops	240	75.00	79.42	-4.42	-1,060.67
Total	**1085**	**1,374.67**	**516.35**	**858.31**	**254,760.58**
Grand Total	**3685**	**2,033.87**	**1,298.93**	**734.94**	**312,713.94**
SUM INCOME 2600ha		**5,288,062.00**	**3,377,214.97**	**1,910,847.03**	

2. DESCRIPTION OF DESIGNED SOLUTION

2.1. Main pipeline network Ø1600mm

Water supply system was designed on basis of adopted variant solution in Preliminary Report. Main pipeline network is a pipeline with initial begining at dam site, some 600m from waterintake. Main pipeline is marked with capital letter P, total lenght is 11,072 m and diameter is Ø 1600 mm. Route of main pipeline P goes next to the existing road, approximately parallel to the Bassara River. This is because construction of pipeline is simplified and access to the route too, as well as favourable terrain configuration. Ductile iron pipes were adopted because of its characteristics.

The gravity pipeline is designed on minimum cross section slopes, to relieve earth excavation where river banks are already steep. Minimum slope adopted is 1.50‰, and is variable depending on topography. Air valves are installed on the highes points of pipeline, totally on three location (with two valves on each location), diameter of air valves is DN 200 mm. Outlets are installed on the lowest route elevations, totally three outlets, with diameters 300 mm. Excavation works, pipe bed preparation and laying of pipes into trench, as well as ways of anchoring are descibed in project.

Solution in which pipeline supply reservoir with water (given in Preliminary Report) is rejected, because it is established that hydraulicaly loses are to high, and 4.1 bars would be destoyed if this soultion is built, and end parts of distributive pipelines in this case would not have enough pressure.

2.2. Primary and secondary pipeline network

Water supply system consistes from primary-distributive pipelines P1, P2 and P3. These pipelines are „foundation" of a system, they distribute water to the rest of primary pipelines, which through secondary pipeline network supply consumers-farmers. Disposition of those pipelines is in correlation with terrain and parcels determined for irrigation as well as vicinity of roads (80% of all sections of irrigation system is close to some main road or other local road).

Primary Pipeline P-1 is separated at junction J1 from main pipeline Ø1600mm, total lenght of this pipeline is 1214 meters with pipe diameters Ø250mm and Ø200mm. It supply the smallest area of system 61.05ha, which is situated between two rivers, next to South-East border of system. Primary Pipeline P-2 is separated at junction J2 from main pipeline Ø1600mm, at the end distance at km 11+072. , total lenght of this pipeline is 4204 meters with pipe diameters Ø300mm up to Ø1000mm. Through this pipeline runs most of water and it supply most of the system. At highest consumption, it distributes 1525 L/s. From pipeline P2 other pipelines continue network: P-2-1, P-2-2, P-2-3, P-2-4, P-2-5 i P-2-6. Diameters less than Ø600mm are from PEHD for pressures up to 10 bars and represent the secundary pipeline network. Secondary network consists of approximately 70 km of pipelines, diameter Ø150mm, and lenght is estimated. Primary Pipeline P-3 is separated at junction J2 from main pipeline Ø1600mm, at the end distance at km 11+072 (same as P-2). This is the longest primary pipeline, total lenght is 6,569 meters and pipe diameters Ø400mm up to Ø900mm. P-3 supply Northern and North-West part of system. From P-3 other primary pipelines spreading to the system. Total area of irrigated system covered by these pipelines is 2875.01 ha.

2.3. Considered variant solution

Considered solution is to divide main pipeline in junction 20, (in scheme is under no. 16) on three, first pipeline P-1 would supply South and South-East part of system, diameter is Ø700mm, and lenght is 7065.12 metars. In this case, part of main pipeline that belongs to P-2 is separated in between junctions J-8 and J-9 (section 38). Complete results of hydraulic calculations are given in project. In this case pressure on end junctions is of primary pipeline are satisfactory and range from 4.5 to 7 bars in this part of the system.

Designer has decided to accept first variant, because even without constructing this pipeline (Ø700mm), it can be accomplished satisfactory water supply of system with needed pressure in pipline. Complete results of hydraulic calculation are given in project and schemes are given at the end of hydraulics calculation (next chapter).

2.4. Hydraulics calculation

Hydraulics calculation is done for four border cases:
1A - level of water in acumulation is on 701m, and consumption is maximal
1B - level of water in acumulation is on 701m, and consumption is minimal ($0.1Q_{max}$)
2A - level of water in acumulation is on 716m, and consumption is maximal
2B - level of water in acumulation is on 716m, and consumption is minimal ($0.1Q_{max}$)

Disposition of pipeline network have been entered with all needed data (terrain elevation,junction discharge, section lenght, pipeline diameters, level of water in acumulation, roughnes coefficient of pipe), Model is calculated by using EPANET software.

2.4.1. About software

EPANET performs extended period simulation of hydraulic and water quality behavior within pressurized pipe networks. A network consists of pipes, nodes (pipe junctions), pumps, valves and storage tanks or reservoirs. EPANET tracks the flow of water in each pipe, the pressure at each node, the height of water in each tank, and the concentration of a chemical species throughout the network during a simulation period comprised of multiple time steps. In addition to chemical species, water age and source tracing can also be simulated.

EPANET provides an integrated environment for editing network input data, running hydraulic and water quality simulations, and viewing the results in a variety of formats. These include color-coded network maps, data tables, time series graphs, and contour plots.

EPANET was developed by the Water Supply and Water Resources Division (formerly the Drinking Water Research Division) of the U.S. Environmental Protection Agency's National Risk Management Research Laboratory.

EPANET's hydraulic simulation model computes hydraulic heads at junctions and flow rates through links for a fixed set of reservoir levels, tank levels, and water demands over a succession of points in time. From one time step to the next reservoir levels and junction demands are updated according to their prescribed time patterns while tank levels are updated using the current flow solution. The solution for heads and flows at a particular point in time involves solving simultaneously the conservation of flow equation for each junction and the headloss relationship across each link in the network. This process, known as hydraulically balancing the network, requires using an iterative technique to solve the nonlinear equations involved. EPANET employs the Gradient Algorithm for this purpose.

The hydraulic time step used for extended period simulation (EPS) can be set by the user. A typical value is 1 hour.

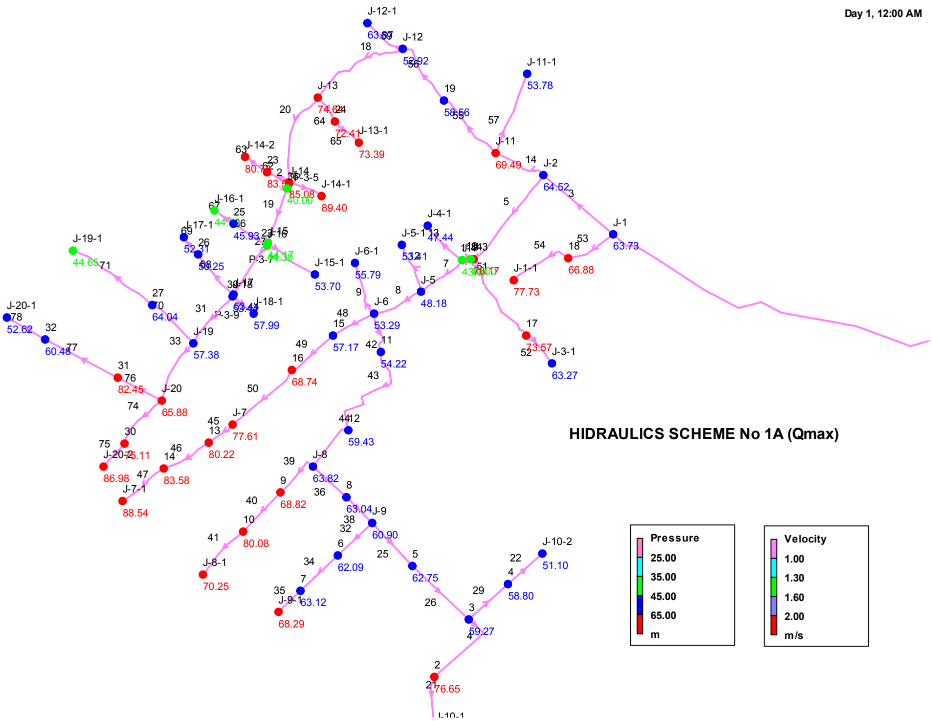

Figure 3. 1A - level of water in acumulation is on 701m, and consumption is maximal

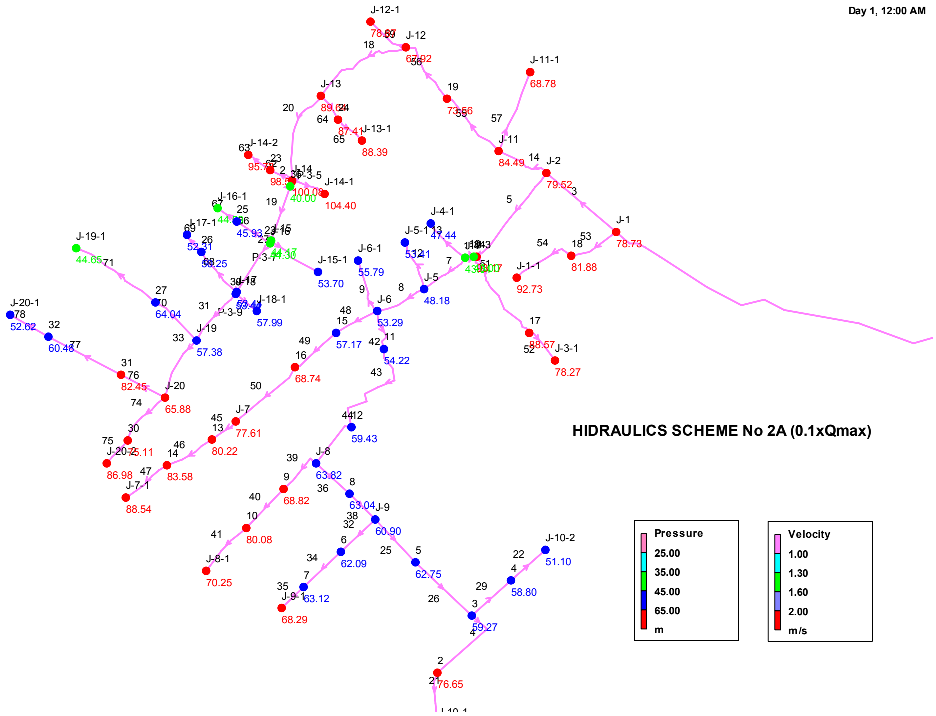

Figure 4. 2A - level of water in acumulation is on 716m, and consumption is maximal

3. CONCLUSION

Planning Report has be designed according to analysis and solutions presented in Preliminary Report. Detailed geodetic works and reconnoiter were conducted, and terrain-location has been visited. It was mentioned in Preliminary Report that according to old topographic maps scaled 1:100 000 and according to technical analysis conducted by Bulgarian and French team of engineers, we made our Prelimiary Report. Big problem occurred when we discovered that there was no detailed pedological map with marked area that were cultivated.

Geodetic works and reconnoiter were done during the June, July and August 2006, and leading designer engineer spent together with geodetic and goemechanic team one week on areas planned to be irrigated. As a result of this field trip, we decided to rationalize solution made in Preliminary report. Here it should be mentioned that minimal working level for dam is 701m, and it is higher few meters than it was planned in Preliminary Report (695m).

As first, rationalization is correlated with exploitation expenses regarding pumping of water. Preliminary Report considering two solutions marked as Variant I – with main pumping station and pipeline to the reservoir, and Variant II, with gravitational pipeline from dam site to the reservoir. Economic analyses of pumping expenses for those two variants were:

- Variant I

27.013.212,00 kWh/year x 0,05 USD = 1.350.660,00 USD/ 2600 ha = **520 USD/ha**

- Variant II

12.270.935,00 kWh/year x 0,05 USD = 613.546,75 USD/ 2600 ha = **236 USD/ha**

After analysys of geodetic survey data, defining cultivated areas, detailed reconnoiter of terrain and adopted working levels of dam, (701 -715m), next was concluded:

1. According to available water from accumulation around 35.000.000 m^3 it can be irrigated 2600 to 2900 ha of arable land. If we assume that all system will be in use, i.e. all 100% to be irrigated, that is 2600 ha. Experience shows that on similar systems, maximum can be expected up to 90% of irrigated areas in one year. After processing geodetic data and analysis of arable land, it can be concluded that area marked as Zone I has enough quality soil for irrigation (≈2875ha).
2. Ground elevation ranging 568 to 600m on south and south-west and on north and north-west ≈640m. Designer concluded that if we consider the water level in accumulation (701 – 715m) we can irrigate whole system by gravity without pumpin stations and pumping water on irrigation system, it is possible to save up to 613.546,75 USD or 236 USD/ha.
3. Land in zones II and III is not suitable for gravitational irrigation system because it is on higher land and also there are many parcels that are scattered with particulary turbulent land configuration.
4. Variant II with reservoir adopted in Preliminary Report was modified, and reservoir was rejected because techno-economic reasons. Hydraulic calculation shows that energy level in main pipeline that runs from dam to earlier predicted place for reservoir for maximal discharge of $3m^3/s$ is between 690 and 700 m (depends on water level in accumulation). Groun elevation on reservoir site is 660 m and it is clear that we have to destroy 3 to 4 bars of water pressure, only to put water to reservoir and after that same water should be pumped to irrigation system so sprinklers could work. Because of same reason, main canal was rejected as solution. Because of all this, it is saved 613.546,75USD or 236USD/ha.
5. Accepted solution in Planning Report is cheaper from investment viewpoint, and there are no expenses for pumping:

I.	PRIMARY PIPELINES	19.477.700,62 (21.628.000,00 Preliminary Report)
II.	SECONDARY PIPELINES	6.404.551,55 (were not calculated in Preliminary Report)
III.	IRRIGATION EQUIPMENT	1.272.000,00 (was not calculated in Preliminary Report)
	TOTAL :	**27.154.252,17 USD (9.444.96 USD/ha)**

6. Maximal quantities of water that will be used for distribution with main pipeline from dam, were revised to $3m^3/s$, we mention that because system is gravitational, depending of water level in accumulation, main pipeline will pass through more water. Correction was made according to Preliminary Report, because losses in sysytem will now be much lower without reservoir and main canal that was 7200 m long. For calculation it is considered that 2875 ha (90%) will be maximaly operational during one year.

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TENTH NATIONAL AND
FOURTH INTERNATIONAL
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NOVI SAD, NOVEMBER 22-24,2006.

Dragana Konstantinović[1]

INTERACTION OF ARCHITECTURE AND ENVIRONMENTAL TECHNOLOGY-TOWARDS BIOCLIMATIC OFFICE BUILDING

Summary: *Interaction of architecture and technological development is evident through history. Built environment immensely reflected social-technological level of the civilization, and is considered for the subject of constant investment and improvement. The paper deals with research of interaction between new aesthetic/social categories and current technological advances, and within these boundaries systems for the indoor climate, as for the most robust ones, without detailed insight into their operation. The paper will highlight the specific systems and pioneers of environmental design within office building typology, where the interference was the most dramatic. The stress will be given to the review of the change phases in the modification of architectural form to meet the air-conditioning requirements..*

The history of environmental management is loosely divided in two phases. Systems of the first generation consider mechanical systems developed for their outbreak until seventies, and so on, which dominate the indoor climate systems after they have been difficultly implemented. Second generation systems represent the attempt of reconsidering and optimization of their non-energetic ancestors, as a result of the new energy situation. This turnover made its mark in designing strategies and presented new forms of successful bioclimatic design.

Keywords: Bioclimatic design; environmental services; office buildings

INTERAKCIJA ARHITEKTURE I ENVAJRONMENTALNE TEHNOLOGIJE-KA BIOKLIMATKOJ POSLOVNOJ ZGRADI

Rezime: *Interakcija arhitekture i tehnološkog progresa evidentna je kroz istoriju. Građena sredina uvek je bila najistaknutija manifestacija socijalno-tehnološkog stadijuma civilizacije, predmet stalnog ulaganja i unapređenja. Tema rada je proučavanje interakcije novih estetsko-socijalnih kategorija i aktuelnih tehnoloških mogućnosti, te u tom svetlu i sistema za kontrolu unutrašnjeg komfora, kao najrobusnijih, bez detaljnijeg obrazlaganja njihovog operisanja. Rad će istaći karakteristične sisteme i pionire envajromentalnog dizajna u domenu poslovnih zgrada, gde je primena i usklađivanje sistema i forme nejevidentnije. Težište će biti dato na pregled postupnosti promena u modifikovanju forme kako bi se postigli kriterijumi unutrašnje klime.*

Istorija envajronmentalnog menadžmenta uslovno je podeljena u dve faze. Sistemi prve generacije obuhvataju mehaničke sisteme razvijane od njihovog nastanka do 70-ih godina, pa i na dalje, koji nakon mukotrpnog procesa implementacije dominiraju u obezbeđivanju unutrašnje klime kuće. Sistemi druge generacije predstavljaju pokušaj revalorizacije i optimizacije svojih ne-energetskih prethodnika, i posledica su novonastale energetske situacije. Ovaj zaokret ostavio je traga u projektantskim strategijama i predočio nam nove forme uspešnog bioklimatskog projektovanja.

Ključne reči: Bioklimatsko projektovanje; sistemi unutrašnjeg komfora; poslovni zgrade

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1. INTRODUCTION

Interaction between technology and built environment is evident through history. The built environment was the most exposed indicator of technological progress since the new technological solutions sought quick and efficient application and historical valorization to become accepted and widely spread. With the development of humanity the changes and shifts in the technological means were more rapid, followed by social turbulences and aesthetical movements. At some point, the synchronization was broken, and the architecture faced revolutionary break with tradition in the form of Modernism.

With the sunrise of Modernism, the 200 years delay of the architectural language was redeemed by radical design shift that will influence our environment and impose new design and living taste. Although the shift to new and updated style was initiated by acceptance of the mechanistic aesthetic and celebrated through the beauty of the mechanical elements, new architecture was very strict and hostile to anything that could jeopardize its absolute geometric beauty. Therefore the implementation of mechanical features into architectural practice was very hard and slow, despite the fact that only its implementation made this new architecture habitable. The paper outlines the outcome of architectural accommodations to embrace new service features, and how this interaction took the course from complete neglecting to installation expressionism.

The importance of the subject for nowadays practice in Serbia is tremendous. There is a leaking gap in knowledge and specially in practicing high environmental quality office buildings in Serbia. It is the mutual and over-all design strategy, which involves architects and service engineers from the inception of the projects to the very end, and understanding of the interaction is of utmost interest for both.

2. DEVELOPMENT OF THE INDOOR THERMAL COMFORT SYSTEMS

Development of systems for the indoor thermal comfort was accompanying shelter development in prehistoric settlements. At the very beginning two basic strategies were proposed [1]:

- **Structural type** that imposed built structure for the main strategy to cope with the problems of indoor climate;
- **Energetic type** initiated with the discovery of fire, which used artificially produced energy to achieve indoor comfort.

After the built structures were developed, the indoor climate was generated by the simple means of design solutions for the local climate. The built habitat emerged in three basic forms as the response to local resources availability and outdoor climate [1]:

- **Conservative habitat**, basic archetype with four walls and ceiling;
- **Selective habitat,** which combined the advantages of indoor and outdoor conditions simultaneously or alternatively so they could be enhanced;
- **Regenerative habitat**, where the indoor climate comfort is reached by application of technical means and appliances.

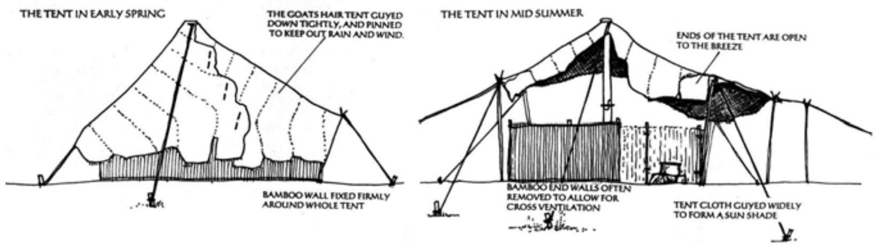


Figure 1.Selective type of shelter- nomad tent of Lury tribe. It accommodates outdoor conditions: in summer it is open to allow pleasant wind breeze; in winter it is closed to keep the heat inside [6].

It is obvious that all these strategies of environmental management are fused to become system we know nowadays-the hybrid one that combines active and passive solutions. The ratio of the strategies employed in contemporary indoor climate systems is the one that determents its energy efficiency.

3. TECHNOLOGY AND DESIGN

Significance of the systems for environmental control was underestimated by architects and was left to engineers to resolve it. From the 19th century, after the electrification of the cities, rapid and intense development of the appliances follow up, modernizing household. Public building commissions reflected this attitude even more, since the urge to achieve the highest design standards was even more present and even better funded.

Modernism divided practice in two opposite standpoints to deal with technology domination situation. One of the attitudes promoted new design beliefs and rationality derived from Industrial outbreak, new social aspirations and consumers' needs. Architecture is promoted as the contemporary follower of the construction industry, with pure and sophisticated language that was suitable for the new taste of the society. The concrete replaced brick and the wide glass surfaces promoted new hygiene and healthy living in rather plain, simple and bright environment of the new buildings. The European architects imposed the question of new form without being ready to deal with tremendous number of issues that followed: the comfort of the inhabitants before all.

3.1. American Ingenuity

First interest for development of the air-conditioning systems was initiated by society that nurtured industry as the contemporary advantage to meet society appetite for coziness. American continent had delayed acceptance of Modernistic design methodology and language but, on the other hand, led the way in technical appliances implementation in design itself. The rational side of the dominant industrial population had different vision- to enrich built environment by new technical conveniences and at the same time to impose new questions: how to mechanically support further coming design solutions.

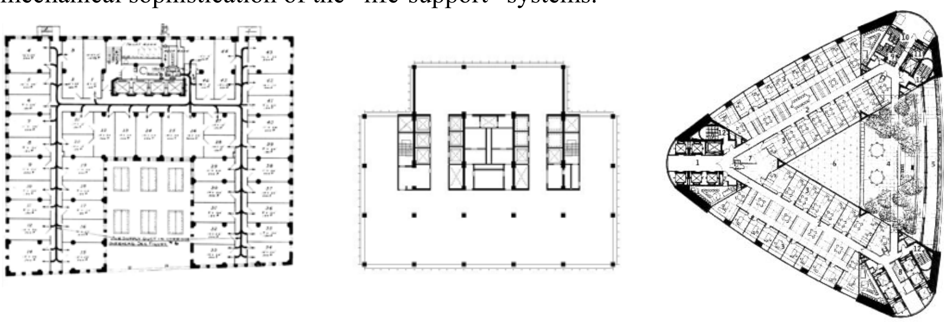
Willis Havilland Carrier is considered for the father of air-conditioning, and the benefits of his patent were widely accepted and promoted. He anticipated the air-conditioning we know today, and made possible its implementation to every kind/type of buildings.

3.2. Concealed systems- Development of new elements

After the small air-conditioning unit was available at the market, the problems of air-conditioning of the large volumes of office buildings were imposed. The presented strategy supposed artificial conditioning of entire volume from the one spot, basically production of the conditioned air and full regulation of the indoor climate. That also meant that the same air needs to be distributed all around the place by the means of ducts and pipes.

The first systems installed were rather robust compared to domestic ones and thus placed in the basement areas. The columns and beams construction system was more than convenient for ducts distribution. Further development will bring new structural solutions and new changes in design in order to reconcile two objectives: inner comfort and aesthetic features.

With the outbreak of air-conditioning and its absolute necessity to support office building of the time, few other appliances were introduced. The development of illumination appliances followed electrification of the households in the 19^{th} century, and the final promotion of the fluorescent lamps will make significant change in the office building volume. By that time, the office buildings were developed around the inner atria, or in the form of recessed volumes, allowing natural illumination and ventilation of the working spaces inside. With the introduction of the air-conditioning and fluorescent lightning, building volumes became more compact. This also allowed new typology to be developed - office skyscraper, as the synthesis of modernistic purity of form and mechanical sophistication of the "life-support" systems.



Figures 2, 3, 4. Development of the office building plan to meet new environmental technology: recessed plan, compact plan, plan with atrium (Milam Building by George Willis, Seagram Building by Mies van der Rohe, Commerzbank Headuarters by Foster and Partners)

3.2.1. <u>Curtain Wall</u>

Development of the centralized air-conditioning system left the envelope to be designed and guided by purely architectural concepts. The new membrane was the result of very delicate process of interaction between design objectives, demand for simplicity, wide glazed areas and lightness. The idea of skin that wraps and seals the building so it could achieve perfect geometric appearance was supported by engineers' demand for autonomous indoor volume that could be regulated without outside interference. These two demands perfectly coincided to present the envelope that will mark the office buildings of the 20th century – curtain wall.

For the first time the all-glass aesthetic was presented, with the introduction of Mies's skin and bone language of cosmic purity and simplicity caught in architectural form. Façade had only dividing purpose, making half-transparent and half-reflecting demarcation line between inside and outside, but very clear difference between indoor and outdoor environment.

3.2.2. Suspended ceiling

Final acceptance of the mechanical features of the architectural space led to development of completely new subculture of the structural elements that will assist in bringing back the space to its origin- geometric purity and precision of design intent. Suspended ceiling was introduced to help cover unpleasant sight on ducts and pipes that curved around the building. Also, the solution had integrated the problem of lightning, installation accessibility and very strict fire regulations.

Discovery and refinement of system was synchronized with improvements in glass curtain wall construction, with enormous compromises made to overcome the quest for undisrupted rhythm of the mullions.

3.2.3. Technical floor

With the densification of the business districts and beginning of chaise for the maximum building height, the rationalization of the installation was needed. One point distribution led to robust ducts that occupied significant area of the expensive office space. The rationalization followed in the form of air-conditioning plants distribution on several spots: in the basement, on the top of the building, and what was new, in the corpus of the building itself. To reduce duct diameters, specialized floors were constructed to serve office spaces above and below them. This functional intermezzo was disturbing for the perfect geometry of the envelope, and sought for adequate design solution. This came through two possible strategies: intentional stressing of the different floor height by façade materialization change, or by concealing the inconvenience as much as possible.

3.3. Exposed systems

The reconciliation between architecture and technology started with shy exposition of the ventilation ducts against dark ceilings in the period after World War Two, when architectural practice gave itself a relief of strict modernist manifests. The presence of the installations is simply accepted and left to bee seen by inhabitants. Some of the architectural theorists consider Unite d` Habitation for the leading example of the new attitudes, but it cannot be stated with complete certainty if that was Le Corbusier 's need to rediscover the ducts, or just sculptural layout at the roof top.

Luis Kahn's Richards Laboratories witness another concept of intentional separation of the served and serving spaces. Although we can have serious doubts in author's statement that he had no other solution in dealing with mechanical addition to the architectural form, but to strongly and visibly divide them in different juxtaposed forms. Nevertheless, the fact that this building was built had its major influence in the following commissions, and open the whole new area of architectural practice, which started to consider mechanical installation for the relevant tool of architectural iconography.

3.3.4. High Tech Architecture

In 60es and 70es the postmodern practice was establishing its way and the pluralism of movements reflected the new spirit, described by Venturi's quest for inclusion, instead of exclusion. High-tech architecture brought installation expressionism to the highest level by inclusion of mechanical systems into architectural vocabulary of the same importance as structural elements. The service spaces and volumes are of the utmost importance for the operation of the architecture, and they are visually exposed and even stressed. The Lloyd Building in London is leading example in office typology.

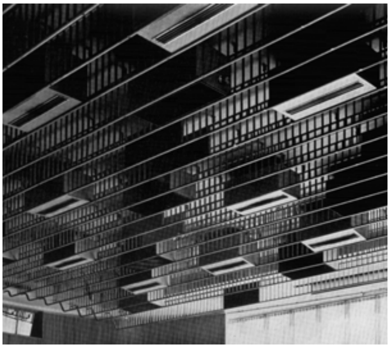



Figure 5: Perfect geometry of the "skin" (Lever House by SOM); Figure 6: Visible air-conditioning ducts (UN Building-Chamber of the Trusteeship Council, by Finn Juhl), Figure 7: High-tech architecture (Lloyd Administration Building, by Richard Rogers Partnership)

4. BIOCLIMATIC OFFICE BUILDING

At the same time, the period of 60-es and 70-es highlighted new issues such as sustainability of the civilization with the outbreak of oil crises. Energetic concept of building which furiously ruled the architectural scene for the previous 70 years needed to be reconsidered seriously. Servicing technology was highly developed by that time and offered easy solution for every climate. Bioclimatic practice reestablished regional and local values and their meaningful reflectance in built environment. The building models became traditional dwellings and their passive, zero-energy solutions for particular climate. Most agree that this discourse of practice had long and continuous history and that Frank Lloyd Wright's organic approach just gave it world-wide importance.

Bioclimatic practice offered two standpoints for practitioners: **reversionary practice** that accepts vernacular for the resource and the language of contemporary practice; **progressive approach** which relies on state of the art technology that is capable of dealing with new energy requirements and will be crucial for development of bioclimatic office building [4].

4.1. Atrium and Sun-Spaces

In the field of office buildings, the shift towards sustainable practice affected the usual office building conception. It did not lead to old pre-air-conditioning era forms, but to reconsideration of their benefits certainly did. The atrium is introduced again, but this time, thanks to advancements in glass technologies, as the solar mediator between inside and outside. The atrium of the office building is contemporary sun-space, where the benefits of solar radiation are utilized and passive solar strategies employed.

However, the great come-back of the atrium in the field of regular architectural practice was neither fast nor easy. It was not the question of architectural choices, but complex adjustments of the office building design, environmental management and passive solar strategies that needed dedicated cooperation between design team members from the very beginning.



Figure8, 9: Inner atrium for passive ventilation-section and elevation (Genzime Centre, by Behnisch, Behnisch & Partners); Figure 10: Glazed atrium as a social spot, two sided illumination of the offices obtained (John Menzies Headquarters by Bennets Associates)

4.2. New Architectural Iconography

Apart from reestablishment of the traditional spatial models, new energy criteria jeopardize the iconic feature of the office buildings: curtain wall. New standards of design and environmental quality neglected sealed buildings as inhuman and bad for health conditions, and sought for more selective approach. This meant serious remodeling of the envelope to satisfy new standards and keep the design fresh and updated. This came first in the form of double glazed curtain wall with operable elements, and then followed by ventilated curtain walls that merged air-conditioning and architectural element into one system [5].

Beside this, whole new assemblage of remodeled traditional elements shows up, all in the need to help low-energy ventilation requirements and consequently enhance the people awareness about sustainable practice. Solar chimneys and wind towers marked some of the leading examples of sustainable office building practice, together with PV modules and wind turbine installed. Some practitioners went even further and proposed self-efficient office buildings that utilize all available natural resources for their operation by modification of the form to "follow energy".

Figure 11, 12, 13: Double glazed ventilated facade (RWE Headquarters by Ingenhoven,Overdiek, Kahlen and Partners); sun pace in the form of double glazed facade (Thomson Advertising Agency by Schneider&Schumacher); solar chimneys and sun shade (BRE Office of the Future by Fellden Clegg Architects)

5. CONCLUSION

The importance of understanding the constant interaction between architecture and its supporting mechanical system is immense in the course of contemporary interdisciplinary practice. Current Serbian practice shows no real strategy in the sense of central air-conditioning, and not even an interest for passive guidelines that are widely applied in Europe. One of the major reasons would be economic interest of contractors and investors. Since they are not the future owners of the property, they are not interested in cutting exploitation costs, and certainly not in additional investment. The lack of property industry results in underdeveloped rentable-office-space market that tends to be very stimulating for the bioclimatic design application. This way the property owners would be competitive by offering lower energy bills and healthier office environment.

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TENTH NATIONAL AND
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Jelena Atanacković-Jeličić[2]

COMPARATIVE ANALYSIS OF MULTI-FAMILY HOUSING COMPLEXES ON THE OUTSKIRTS OF CITIES IN VOJVODINA PROVINCE

Summary: The ***purpose*** *of this paper is to examine the ways of territorial expansion of cities in Vojvodina in the post-war period when large multi-family areas were built on the city fringes reflecting the formal «new town» planning concept with separation of space in different functional zones. The collection of the cases presented in the paper covers 6 multi-family complexes located in the peripheral zones of 5 cities of the Province (Satelit and Detelinara in Novi Sad, Prozivka in Subotica, Bagljaš in Zrenjanin, Mikronaselje in Kikinda and Pesak in Senta). The intention is to explore common processes and principles of their spatial and functional formation as well as to illuminate existing differences.*

Key words: multi-family housing, peripheral zones, urban morphology, building typology, city centre, elementary school

KOMPARATIVNI PRIKAZ NASELJA SA VIŠEPORODIČNIM STANOVANJEM NA PERIFERIJI VOJVOĐANSKIH GRADOVA

Rezime: Cilj ovog rada je da se kroz komparativnu analizu odabranih naselja višeporodičnog stanovanja, koja tek u poslednjih 50 godina oblikuju periferiju vojvođanskih gradova, istraže dominantni pravci razvoja i prostornog širenja urbanih centara regiona. Kao predmet istraživanja odabrano je 6 naselja, lociranih u perifernim zonama 5 gradova Vojvodine (Satelit i Detelinara u Novom Sadu, Prozivka u Subotici, Bagljaš u Zrenjaninu, Mikronaselje u Kikindi i Pesak u Senti). Valorizacijom na različitim nivoima utvrđene su sličnosti morfoloških, funkcionalnih i ambijentalnih karakteristika, ali i osobenosti analiziranih naselja, nastale uglavnom pod uticajem lokalnih faktora.

Ključne reči: višeporodično stanovanje, periferija, urbana morfologija, arhitektonska tipologija, centar grada, osnovna škola

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1. INTRODUCTION

The massive urban growth is one of the most important and addressed phenomena of twentieth century, becoming even greater at the beginning of the new century. The cities are growing by expanding their boundaries, which is creating very severe environmental, social and economic problems. These problems occur at a rate and in ways unparalleled in history of cities. Most specific spatial transformations of urban periphery are occurring today on a global scale, so a rich diversity of suburban forms has been generated in different parts of the world. Many studies address these issues from a variety of perspectives, but the lack of those analyzing physical patterns of suburbanization is also noticeable. More detailed analyses are therefore needed to examine physical characteristics of new urban extensions, which are affecting the lives of millions and soon billions of people.[1] These studies should point out specific local, social and environmental conditions in order to understand different processes and physical transformations at the periphery of cities throughout the world.

1.1. The aim of the paper

The issues of evolution of cities in Vojvodina Province and transformation of their overall composition and physical characteristics have continuously been related to the theme of development of the new peripheral and suburban extensions. The aim of this paper is to approach the phenomenon on both macro (the regional scale) and micro level (level of the residential complex). The primary purpose is to examine the ways of territorial expansion of cities in Vojvodina in the post-war period when large multi-family areas were built on the city fringes reflecting the formal «new town» planning concept with separation of space in different functional zones. The collection of the cases presented covers 6 multi-family housing complexes located in the peripheral zones of 5 cities of the Province (Satelit and Detelinara in Novi Sad, Prozivka in Subotica, Bagljaš in Zrenjanin, Mikronaselje in Kikinda and Pesak in Senta). The intention is to explore common processes and principles of their spatial and functional formation as well as to illuminate existing differences.

1.2. The criteria of the analysis

The following criteria are set to avoid fragmentary approach and to perform analysis stressing the multi-faceted nature of the urban environment:

1. the position of the residential complex in the city organism,
2. the characteristics of the physical structure,
3. functions, programmes and contents,
4. traffic networks and modes,
5. open and green spaces.

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[1] During the period between 2000 and 2030 2.2 billion people will be added to the world population and 2.1 billion of those will be urban dwellers. The source of this forecast is United Nations Population Division World urbanization prospects: the 2001 revision, US Census 2000. (Stanilov K, Case Scheer B, eds.: "Suburban Form – an international perspective", Routledge, New York, London, 2004., pp.1)

The characteristics of new peripheral complexes are also compared to the images of the city centres – traditional places with many various layers of history, culture and social memory.

Analyzed residential areas are in this paper presented using comparative method in order to investigate similarities and differences of their structures.

2. COMMON CHARACTERISTICS AND PROCESSES

All analyzed multi-family housing complexes were built based on very similar principles and technologies that have transformed the urban periphery and marked the loss of continuity with the older parts of the city. Former identity of the city fringe with many rural elements and dominated by individual housing has been changed by new and almost always inadequate building types and urban fabrics. Rationalization and industrialization in the construction of residential areas was considered here as necessary to resolve the issue of housing shortage. This has created a number of functional, environmental, social, technical and economic problems in analyzed neighbourhoods. Similarities of this kind could not however been understood as regional characteristics since they are typical of many other multi-family housing complexes built in the postwar period throughout Europe and also in other parts of the world.

Common characteristics of analyzed areas are investigated from the aspects of urban morphology and architecture and its styles, but the analysis also includes themes of functions, programmes and contents.

2.1. Urban morphology

Great similarities are identified on the level of urban morphology and the basic elements of urban form – street patterns, urban blocks and the density of building structures.

The street grids are orthogonal and do not differ much from street layouts of former peripheral structures. They are however different from networks of streets in the city centres, which are irregular and have in all analyzed cities (apart from Kikinda) been formed as a result of genius loci. Rigidity of street layout is an implication of "one-sided planning concept" typical of all compared areas. This issue is also connected to the quality of traffic conditions since it promotes the idea of radical separation of vehicular and pedestrian traffic.

The block structures of analyzed multi-family housing complexes are orthogonal, with an exception of Pesak in Senta where the block is triangular. In this case the regular geometry is also generated, which again represents a radical difference compared to the irregular shapes of blocks in the centres of cities in Vojvodina. The exception is Kikinda where the whole city form is planned and built as orthogonal and regular in shape. In all analyzed peripheral areas urban blocks are much bigger than those in historic cores. The oversized blocks have open perimeter since the building types also differ from those of city centres. Rows of single-family houses are as a typology replaced here by tower and slab buildings. The concept of open block perimeter in the peripheral multi-family housing complexes also introduces new relations between public domain and semi-public spaces and marks the great changes of traditional relationship between buildings, lots and streets that characterize centres of cities in Vojvodina Province. Repetition of identical buildings in the "free space" of empty green is the main characteristic of urban milieu in analyzed neighbourhoods.

The compared residential areas also have similar densities of building structures. While city centres are densely built, these peripheral zones have been planned with much lower density of buildings. At the same time concentration of people leaving in the new city extensions is much bigger than in the city cores. Multi-family housing complexes located at the periphery of cities in Vojvodina are taking the role of a "centre" in the life of many people, unfortunately lacking functions, symbolic elements and complexity of traditional city spaces.

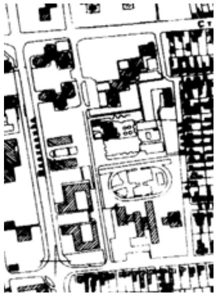
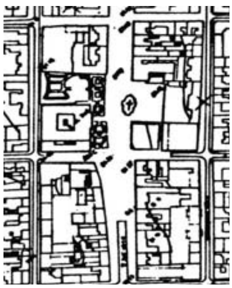
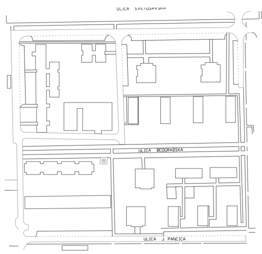
Multi-family housing complex	Typical urban block located in the city centre	Typical urban block in the residential area
BAGLJAŠ in Zrenjanin		
MIKRONASELJE in Kikinda		
PESAK in Senta		

Table 1. Comparative analysis of the typical urban blocks located in the city centres and in the multi-family housing complexes at the periphery

2.2. Architecture - its styles, elements and character

The components that defined the image of former urban periphery are radically changed also in the field of architectural characteristics. In the past centuries peripheral zones of the cities in Vojvodina have been characterized by what could be referred to as "architecture of the earth"[2] and "architecture without architects". In the compared multi-family housing estates architectural objects were built using prefabricated system in order to make the building process as cheap and effective as possible. Diversity of materials, sizes and combinations of elements was not among the primary purposes of this process, so the architecture was plain and un-decorated, characterized by simplicity. Flat roofs dominate and give a strict geometrical character to most of the buildings. Symbolic or semantic components which would create a specific identity of compared residential areas could not been identified. That also stresses the difference to the architectural characteristics of city centres. Another concept was introduced to the traditional urban structure by building multi-family housing complexes at the periphery. High rise buildings were built creating structures incongruent with the traditional horizontal setting of urban centres in the Province.

Multi-family housing complex	Identity of the historic core	Typical image of the residential complex
DETELINARA in Novi Sad		
PROZIVKA in Subotica		
PESAK in Senta		

Table 2. Comparative analysis of specific images of the city centres and peripheral structures

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[2] This is the building form with many rural elements that exist even today at the periphery of all cities in the region. (Pušić Lj: "Urbanistički razvoj gradova u Vojvodini u XIX i prvoj polovini XX veka", Matica srpska, Novi Sad, 1987., str. 183)

2.3. Functions, programmes and contents

All analyzed complexes were built in the post-war period when functional urban segregation was the main planning principle. Housing and working were considered two separated and opposed programmes. Functional segregation of this kind produced a centralized model of the city even in this region. The compared areas lack working places and also many necessary services and facilities. Sub-centres containing these programmes were in many cases planned but remained the "second phase" that was never or only fragmentary built. Absolute domination of housing in all analyzed estates has a huge socio-economic impact on the overall quality of space and at the same time on the quality and intensity of urban processes and urban life.

The analysis of multi-family housing complexes from the aspect of functions, programmes and contents showed another explicit model typical of many other "new towns" built in the post-war period in different countries. Elementary schools and public nurseries were built in all compared areas (except the smallest complex Pesak in Senta). This "educational block" is in all analyzed cases positioned in the centre of community in order to create equal distances to all parts of the neighbourhood. The architecture of school and nursery buildings is characterized by the similar kind of unification of building elements and components as multi-family blocks, showing no diversity, flexibility or adaptability of plan.

DETELINARA in Novi Sad	BAGLJAŠ in Zrenjanin
	
PROZIVKA in Subotica	MIKRONASELJE in Kikinda
	

Table 3. Elementary school buildings in analyzed residential complexes

3. EXISTING DIFFERENCES

The specific characteristics of analyzed complexes are generated as a result of the relations they form with their surrounding structures. Differences and variations are identified as an implication of local conditions, particularly concerning relationships with the historic city cores and surrounding countryside and natural resources of various kinds.

Different types of hierarchical systems are formed depending on the city size and the level of services and commercial activities in the residential areas. Since all analyzed cities are centralized, regardless of their size, the complexes located at the periphery are dependant on city centres because of the lack of working places and various functions and programmes, but also because of the lack of symbolic elements that form identity and collective memory. The relations of this kind are tighter in larger cities, but some exceptions are identified in this study. The grade of dependance can be great even in small cities where the residential areas are located not far away from the historic core. This is the case of complexes Mikronaselje in Kikinda and Pesak in Senta that are poorly equipped with services and other functions apart from housing. On the other hand, in the case of Satelit and Bistrica residential areas in Novi Sad, located 7 km away from the city centre, lower grade of dependance is identified, since the structure is much adequatly equipped with various functions and urban contents and the transportation and traffic conditions are better.

Relations that compared complexes form with the surrounding countryside and natural resources also differ. Both analyzed peripheral areas in Novi Sad are not surrounded by cultivated and arable land. In other cases residential areas are not well-integrated with their natural surroundings, but rather opposed forming contrast images. Again the exception is complex Pesak in Senta located along the river bank where the inhabitants benefit the adventage of this specific location.

4. CONCLUSION

This study marked an effort to investigate whether characteristics of postwar multi-family housing complexes located at the periphery of cities in Vojvodina could be typified. It comprised analyses and comparisons of the built form of 6 residential areas from the perspective of multitude of factors. Remarkable patterns and similarities are determined related to urban morphology, building typologies, function, programmes and contents and to traffic networks and modes, services, maintenance and the overall quality of life and intensity of urban processes and activities. Great similarities are also recognized and stressed related to positioning of the «educational centre» in the analyzed neighbourhoods. Variations are more distinctive when formed as a result of specific local conditions, particularly concerning relationships with the historic city cores and surrounding countryside and natural resources of various kinds. Different types of hierarchical systems are identified depending on the city size and the system of services and commercial activities in the neighbourhood.

Multi-family housing complexes changed the former image of peripheral zones of the cities in the Province introducing different physical elements, patterns and spaces. Non-urban development patterns are replaced by those with more urban elements, which on the other hand showed many weak points. The future planning policies of urban growth should therefore be thought on much more detailed and elaborated level.

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TENTH NATIONAL AND
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Milena Krklješ[1]

ANALYSIS OF PUBLIC SPACES FOR CHILDREN AND YOUTH IN NOVI SAD

Summary: Public spaces with their contents reflect the needs of citizens, but also the whole socio-economic conditions in a society during a certain period of time. It is an open space in the city, accessible to all the different social groups of citizens. Its attractiveness depends on a group of sanitary-technical, architectural and horticultural conditions, buildings and other spaces around it. The distribution of public spaces in the city, their proportions and dimensions, programs and contents, should be designed as to create harmony in relationship between spaces and their users, making the whole process of socialization successful and more intensive. The importance of public spaces for children and youth is based on possibilities of the realisation of different needs for their psycho-physical development, but also for their integration into society. Children and youths are one of the most frequent users of public spaces. Their surroundings are not only the home or a yard, but the whole neighbourhood which creates a stage to induce children activities and development. Continuous and various games in open spaces, with numerous physical and emotional reactions, have great significance for children's health. Environment, accepted as children friendly, offers everyday challenges according to children's development levels. As a population which can not independently materialize their needs and wishes, children and youth are usually neglected due to the needs of other more aggressive groups. Careful analysis and valorisation of public spaces for children and youth could be a step forward to new and better images of the whole community and the city. Therefore, this paper discusses the different typologies of public spaces such as open spaces in schools, children playgrounds, sports fields, public green spaces in the whole area of Novi Sad.

Key words: *public spaces, children playgrounds, sports fields, public green spaces*

ANALIZA JAVNIH PROSTORA OKUPLJANJA DECE I MLADIH U NOVOM SADU

Rezime: *Javni prostori u gradu svojim sadržajima reflektuju potrebe korisnika, celokupnu sliku socijalno-ekonomskih uslova u društvu tokom određenog vremenskog perioda. Njihova atraktivnost korisnicima, zavisi od niza sanitarno-tehničkih, arhitektonskih, hortikulturnih uređenja, obrade parternih površina, objekata, ulica i uređenja svih okolnih prostora. Značaj javnih prostora za decu i mlade zasniva se na mogućnostima ispoljavanja brojnih potreba za pravilan psiho-fizički razvoj, te socijalnu integraciju u širu društvenu zajednicu. Deca i mladi spadaju u najčešće korisnike javnih prostora. Njihovo okruženje nije ograničeno na dom ili dvorište, već je čitavo susedstvo pozornica koja podstiče dečje aktivnosti i razvoj. Kao populacija koja nema mogućnosti da samostalno materijalizuje svoje potrebe, često je zapostavljena usled potreba drugih, agresivnijih interesnih grupa. Pažljiva analiza i promišljanje ovih prostora, može biti korak ka novim kvalitetima i slikama javnih prostora okupljanja dece i mladih, a samim tim i boljoj sveopštoj slici društva. Radi sagledavanja celokupnog stanja prostora njihovih okupljanja, istražene su i valorizovane različite tipologije prostora za decu i mlade na području Novog Sada: slobodni prostori uz predškolske ustanove, osnovne i srednje škole,sportski tereni,dečja igrališta, javne zelene površi.*

Ključne reči: *javni prostori, dečja igrališta, sportski tereni, javne zelene površi*

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1. INTRODUCTION

Public spaces are places where we can observe everyday life of a city. In this paper, the phrase *public space* stands for inbuilt places in the city, accessible to all different social groups of citizens. It is a place of meetings, events, moving and exchanging and many other events of social life. We value these spaces through its morphology, architectural and urban structure, social, psychological and ambient characteristics. Public space in today's city is "a stage in the arena of urban life: the place of anarchy, conflicts and codex it has as its concrete result."[1]

This paper deals with analysis of public spaces where children and youth meet and gather in Novi Sad. Today's cities have fewer spaces which we can mark as acceptable for children and youth. Complex and artificial environment where children grow up, offers them fewer opportunities to find about the world around them, with their own senses and "cramped quarters for moving and meeting with other children, have as a result specific isolation from real life, intensified by an effect of hypnovision and other mediums of artificial information. In this way children are deprived of their own possibilities for their own authentic experience which is important in their development".[2]

Children play is one of the most important activities during their growing up and development. Playing different games, they experience different emotional reactions; they explore and understand their environment, which results in creating special relations with it. Activities, taking place in public spaces, which children make up themselves, are of special importance [4].

2. THE IMPORTANCE OF PUBLIC SPACES FOR PSYCHO-PHYSICAL DEVELOPMENT OF CHILDREN AND YOUTH

Children and youth are social groups that are most frequent users of public spaces in urban fragments of a city. Many needs for proper psycho-physical development of this particular group of population happen in these places.

Certain fields of psychology and pedagogy, engaged in different aspects of children play, think of it as a basic type of children activity, as it reflects the whole neuro-physical state of a child, their biological and physical needs [11]. This theory which sees play as the central activity in the childhood is generally accepted. It also emphasizes the importance of play in any child's intellectual development, also presented by the findings of the cognitive theory of children play, which confirm the connection between the way children play and their intelligence [9]. According to Karl Gross, children play, which originates from their natural needs and impulses, has biological significance: it prepares us for life; creativity sharpens intelligence and psychologically speaking gives pleasure. These plays are special creative activities which result in a communication between a child and their surroundings. While playing games, children develop their psycho-

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[1] Bobić M., Grad između arene i scene, Urbani spektakl, Clio, Beograd, 2000., p. 15

[2] Kamenov E., Intelektualno vaspitanje kroz igru, Zavod za udžbenike i nastavna sredstva, Beograd, "Svjetlost" OOUR Zavod za udžbenike i nastavna sredstva, Sarajevo, 1989., p. 8

physical characteristics of personality and exercise socialisation. Apart from physical, cognitive, creative and emotional aspects, play also includes social one [5].

Until XIX century, not much attention was paid to spaces for children's staying and playing. The emancipation of woman allowed the problem of child care to be considered. In 1920s scientists defined the term children development and play, and started to think about children playgrounds. During the 1970s, a group of scientists supported the idea of the necessity for the standardisation of public spaces, hygiene and establishing authority over children. The other group, however, believed that children should be given free imagination while playing and the variety of spaces in order to establish their self discipline. In contemporary tendencies, architects who have an important role in creating spaces for children first have to consider educational elements in creating spaces for children with many different activities, to stimulate children's imagination and their exploring spirit [8].

Continuous and various movements in open space, richness in physical and emotional reactions are influences of great importance in children development. It is believed that more often then not social and environmental factors as well as intrapersonal ones influence their behaviour. Research has proved that dissatisfied needs of children and youth, could result in different types of antisocial behaviour, neuro-pathological conditions, even in certain neuropsychiatric illnesses [11]. Children and youth environment and their tempo of life, especially in cities, require our special attention while studying and creating open spaces for them, in order to reduce bad influences, to encourage their proper psycho-physical development and to allow children to express themselves as individuals in their communities.

3. SOCIAL ASPECTS OF THE GATHERING OF CHILDREN AND YOUTH IN PUBLIC SPACES

Doing research of public spaces and gathering of people in the city, we can conclude that there are number of social aspects and their influences on children's and youth's life. The assumptions of some scientists that family and school as institutions are the only adequate surroundings for children, can allow us to reach wrong conclusions and act wrongly when designing public spaces for children and youth. This kind of thinking has negative consequences. It results from neglecting the social importance of spending time in different surroundings as well as exaggerating the importance of the family and school. As a result, there is inferiority and confusion of children and youth in interaction with people of different age and social groups [13].

Children better develop social behaviour and, at the same time, overcome the sense of loneliness, which is often proved to be fatal in their psycho-physical development, by spending time in playgrounds and sport grounds than at home. Although we can define the classification of public spaces based on their function, the definition of these spaces socially speaking is unique. The significance of these spaces reflects in the fact that children and youth "usually act as a link between socially different groups. Children ignore differences between them and make friendships with children from social groups their parents don't interact with."[1] The distribution of public spaces in the city, their proportions and dimensions, programs and contents, should be designed to create

[1] Vujović S., Ljudi i gradovi, Mediteran, Budva, 1990., p. 49

harmony between spaces and their users, making the whole process of socialization more successful and intensive.

4. BASIC SPACE AND FUNCTIONAL FEATURES OF PUBLIC SPACES FOR CHILDREN AND YOUTH

From children perspective, space can be defined as a serious of different structural elements whose relations forms an entity. For this particular research, the definition of space as sequence of experiences and events is more important. Children recognize space as a series of activities connected with a particular place, but only after certain experiences can create their own image of that space. According to C. N. Schulz, space can be defined using five space concepts: pragmatic space of physical activities, perceptual space of direct orientation, existent space that represents the stable image of people's environment, cognitive space of physical world and abstract space of pure logical relations [14]. This differentiation can be used to define what children's spaces should be like as their point of support throughout life.

Acceptable environment for children and youth consists of everyday challenges suitable for their levels of development, age and gender. Environment model suitable for children should encourage them to explore and discover new possibilities of their own surroundings. Country environment is considered to be most acceptable model of surroundings that satisfies children's needs. During our studying and creating of children's spaces, we should consider some of basic terms connected with a way children can make a dialogue with spaces: senses (of sight, hearing, smell, taste and touch) which helps children to perceive space; psycho-mobility (climbing, running, jumping) which enables children's development and coordination of their own body in different spaces; symbols, which are in a domain of creativity and enable children to perceive world through imitation before they are able to demonstrate their own ideas; interrelations, as a result of direct contact with other children and enable comprehension of socialisation and collective life [8].

We can define features of public spaces where children and youth spend time and have different activities taking into consideration and accepting children's needs from the standpoint of medicine, psychology, pedagogy and sociology. Some basic demands for children's public spaces are:

- Location in the direct vicinity of buildings children live in and spend time,
- Accessibility to all potential users, especially for disabled children,
- Safety of children in playgrounds,
- Spacious of playgrounds, in order to allow free and various activities,
- High quality of public space, in the sense of the natural light, wind protection, adequate vegetation,
- Hygienic conditions,
- The presence of water, sanitary facilities and some storage spaces,
- Appropriate equipment for play, in the sense of used materials, dimensions and locations,
- Long life and sustainable development of public spaces, with possibilities for flexible changes in modern tendencies and needs of children and youth [11].

The selection of elements used in public spaces and children playgrounds is of great importance. A typical characteristic of all elements in use should be their challenging role in children's play. Use of standard products mustn't make playground boring for children, as there is a risk of their trying to find some more attractive and unusual places for play which are usually also dangerous for their stay.

5. THE TYPOLOGY OF PUBLIC SPACES FOR CHILDREN AND YOUTH

The typology of public spaces for children and youth is based on different age groups (according to children's psycho-social needs) and functional characteristics of spaces. We can define division of analysed spaces according to age groups of children and youth:

- Public spaces for children aged 1 to 3, accompanied by adults,
- Public spaces for children aged 3 to 7,
- Public spaces for children aged 7 to 14,
- Public spaces for children and youth aged 14 to 19.

Each of the previously defined spaces has specific characteristics (contents and safety aspects) for stay and play of certain age groups of children and their proper psycho-physical development. Children and youth gather in public spaces, which have various, attractive and interesting elements for play. As a result, these are the different types of spaces for play:

- Public spaces-children playgrounds with different elements for play, usually located near children's institutions and blocks of flats. Elements on playgrounds offer, but also impose certain kinds of physical games and activities.
- Green spaces in open or semi open urban blocks enable children to imagine and create games by their own rules.
- Sports fields - places for recreation and various sport activities of children and youth, usually located near schools and in urban clusters.
- Public spaces in kindergarten and school yards, with various places for play and sport activities and spending free time of children and youth.

Sports fields between urban clusters in Liman III.

New playground in Kotorska street.

Playground near SPENS.

Picture 1. Children playgrounds and sport fields.

6. PUBLIC SPACES FOR CHILDREN AND YOUTH IN THE CITY OF NOVI SAD

Novi Sad has urban grid with many different public spaces. However, indifference and negligence of spaces has made many of the playgrounds ruined. Urban growth and fast urban transformation in the city have as a result changes in relations between built and inbuilt spaces, public/semi-public and private spaces. Creating of public spaces for children and youth which are necessary for their proper psycho-physical development are neglected. As the population, which can not independently materialize its needs and wishes, children and youth are usually left to spend their time in badly designed and very often spaces in extremely bad condition.

The analysis of public spaces in Novi Sad has confirmed hypothesis that there is a connection between certain typologies of buildings and urban clusters, and types of playgrounds for children and youth. The subject of this analysis are public spaces in urban fragments of Novi Sad, defined by main boulevards and named after existing classification of the city (Novo Naselje, Satelit, Telep, Limani, Grbavica and Adamovićevo naselje, Centar, Podbara and Salajka, Bulevar and Rotkvarija, Detelinara and Sajmište). Different types of public spaces for children and youth are analysed and valorised: open spaces in kindergarten and school yards, sports fields, playgrounds, green spaces and open spaces in urban clusters acceptable for play.

Open spaces in kindergarten yards in the institutions of "Radosno detinjstvo", are usually designed and built at the same time as the buildings themselves. A few kindergartens, however, do not have their own yards, so children have to go to playgrounds in the area. These playgrounds are generally intended for children aged one to seven. Basic characteristic of these spaces is presence of many different elements for play. These are space compositions, made of wood, metal or plastic materials, in bright colours, which enable children to develop and improve their psycho-physical abilities. Arranged greenery is present in these places, so we can make a conclusion that these spaces are usually adequate for children to spend their time in. These playgrounds are places of gathering for many children from neighbourhood during weekends.

Open spaces in both primary and secondary schools comprise two parts. One part of a school yard is greened or paved and with various urban furniture, where children and youth spend breaks and sometimes meet in the evening. Another part of a school yard comprises sports fields and is designed for lessons of Physical Education, but are often used in children's and youth's free time at weekends, owing to shortage of sports fields in certain areas of the city (such as Telep, Podbara, Salajka, Grbavica, Adamovićevo naselje). Most of sports fields are designed and equipped for playing football and basketball, and there is an evident shortage of urban furniture and other facilities (such as toilets, bathrooms and changing rooms). The design of the school yard in Music School is very special – it has been transformed into a rather small summer stage, which is used for concerts in nice weather. These occasions make this space the place of gathering of many music lovers, and people who simply want to enjoy in live music.

A small hill is enough for children to create their own play.

Ruined playground with concrete elements.

Children playing in Dunavski Park.

Picture 2. Spaces of children's play

Public spaces in some urban clusters (Novo Naselje, Limani and on Bulevar Oslobođenja) are organised as sports fields, and they are usually crowded with children and youth during summer time. However, some of these spaces are run-down or even ruined and with certain elements such as hoops and goal constructions dangerous for play. More green vegetation close to sports fields would make sports activities much more pleasant and healthier. The areas of the city with detached and semi-detached houses only do not have any open spaces for sports activities. This presents a real problem since children and youth in these areas play on, sometimes rather busy, roads. This is unsuitable and particularly dangerous.

Children playgrounds in urban clusters and other green spaces have been partly revitalized in the past two years. Many broken elements for play have been replaced with new ones or even with the whole new compositions of play elements. There is a visible lack of greenery in these spaces which can occasionally make play impossible during summer, because of unbearable heat. In addition, some urban furniture and lighting may contribute to a better image of these spaces.

Open green spaces and other open spaces in the city enable children to play a number of different games and do activities, including the ones they make up rules for themselves. Unfortunately, these public spaces in the city are not adequately equipped and looked after.

7. CONCLUSION

Upon analyzing many of the public spaces for children and youth in Novi Sad, we can conclude that the unified need of children and youth for gathering and play, sports and other leisure activities does exist in diversified architecture and urban projects of the city. Open spaces in kindergartens, schools, playgrounds and other open spaces in Novi Sad have fewer and fewer attractive places for gathering and spending free time. Moreover, there is an obvious shortage of spaces for play and other leisure activities in many of the newly built urban clusters, which have been built without proper planning and thus neglect providing free space around them.

One of the main reasons why public spaces for children play and other activities are being neglected or do not exist is insufficiently and unclearly defined standards of their designing and constructing. Spacious green spaces with no contests are not interesting enough, thus they lose their importance as places for socialization of the youngest residents. On the other hand, lack of open spaces in some urban areas results in difficulties when constructing sports fields there. Furthermore, the existing playgrounds for children and youth are often poorly looked after or even ruined which makes them less attractive for everyday play. Some of them are absolutely not the right and appropriate places for children and youth. One of the most important moments of growing up and becoming mature of any human being is the sense of belonging to a certain space, and discovery and exploration of self-expression and individualism. Our duty as architects is to design and provide spaces for children and youth which will be accepted as the environment where they will grow up and be able to express their individualism. Some experiences and examples from developed European countries can teach us about the necessity for defining a national strategy for children spaces. The needs of children from various social and age groups should be considered. Therefore, local experts and residents should have an important role in this process.

While considering possible ways of revitalization of children spaces for play we should bear in mind that the perception and needs of children and youth considerably differ from those of adults. Aware of this fact, we must try to apply our own experience when drawing up proposals for playgrounds which would be acceptable to younger generations. Spaces for children and youth "should not be something definite and final, but in continuous process of transformation and places of open possibilities."[1]

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NOVI SAD, NOVEMBER 22-24,2006.

Slobodan Krnjetin [1]
Dragana Konstantinović[2]
Miljana Zeković[3]

BUILDING WITH EARTH MATERIALS-REEVALUATING TRADITION OF THE AREA

Summary: *Building with earth materials is traditional construction technique in Vojvodina Province. It represents significant part of region built heritage and most of the existing structures are inhabited nowadays although more than century old. Presented paper deals with research conducted at the Institute of Architecture and Civil Engineering which goal is to restore traditional techniques of earth building and implement them into contemporary practice. Considering appealing quest for cutting energy consumption and remarkable displacement of the architecture from its surroundings, we found this to be the most sufficient solution for both. The research covers three years life span and is developed through several stages, aiming to cover all relevant aspects of the subject. First stage of research includes fieldwork and laboratory testing of the samples in Institute facilities in cooperation with industry representatives. Data base created in this stage of work is the start point of all other research steps including demo building design brief creation, construction of a real structure, as well as incorporating relevant topics in educational program..*

Key words: *Earth buildings; Building thermal performance; Energy balance; Sustainability; Low-tech design; Recycling; Traditional building techniques.*

GRAĐENJE MATERIJALIMA NA BAZI ZEMLJE-PREISPITIVANJE TRADICIJE REGIONA

Rezime: *Građenje materijalima na bazi zemlje je tradicionalna tehnika građenja u Vojvodini. Ona predstavlja veoma značajno graditeljsko nasleđe regiona. Prezentovan rad prikazuje istraživanje sprovedeno na Institutu za arhitekturu i građevinarstvo čiji je cilj da preispita tradicionalne tehnike građenja materijalima na bazi zemlje i primeni ih u savremenoj praksi. Uzimajući u obzir zahteve za redukcijom potrošnje energije i primetno izmeštanje arhitekture iz njenog okruženja, nalazimo da je predloženo rešenje najadekvatnije za oba problema. Istraživanje je razvijano tokom tri godine kroz nekoliko faza, s ciljem da obuhvati sve relevantne aspekte teme istraživanja. Prva faza obuhvata prikupljanje podataka i laboratorijska testiranja uzoraka u Institutu u saradnji sa predstavnicima građevinske industrije. Baza podataka oformljena u ovom stadijumu rada predstavlja polaznu tačku svih ostalih koraka u istraživanju, uključujući pripremanje projektnog zadatka za demo objekat, konstruisanje realne strukture, kao i inkorporiranje relevantnih tema u programe edukacije.*

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Ključne reči: *Objekti od zemlje; termičke performanse; energetski bilans; održivost; niskoenergetsko projektovanje; reciklaža; tradicionalne graditeljske tehnike*

1. INTRODUCTION

In the course of contemporary practice sustainability is labeled for the feature that will redefine our design actions. Sustainability is the strategy for planetary development and therefore strongly involved in building production, where it seeks for fresh starting point to meet new energy requirements. Under these circumstances, designers have taken two opposite standpoints to cope with presented issues: low-tech design and construction is guided by reinvention of the old building techniques and architecture remodeling for the new taste and market; high-tech architecture is contemporary follower of the new technology and materials development that uses state of the art techniques to achieve desired energy criteria. Both of the attitudes found their indebtedness in certain typologies, where the final purpose of the design supports the technique of design itself.

Design and construction of dwelling typologies has long and continuous history. It is traditionally indebted in local and reflects the builder's sincere understanding of the surrounding presented through the form and choice of materials. Therefore, buildings are closely related to the local climate, constructed with least amount of energy and materials, mainly from local resources. In Vojvodina, north Province of Serbia, which is the plain agricultural region, building with earth is long and continuous construction technique. Material used is from local resources and construction process is based on household members' labor due to its simplicity and low technology features. In post war period this technique is abandoned and replaced with brick building since the outbreak and rapid development of construction industry.

2. OBJECTIVES

The research conducted in our Institute found this construction technique highly sufficient to meet nowadays need to cut energy consumption without deterioration of living standards of the occupants. Earth structures are durable, easy to construct, with outstanding indoor climate achieved with less amount of energy what makes them most suitable for harsh climate of the region with hot summers and very cold winters. The research proposal addressed the following issues:

Revival of the traditional building method and its modernisation to meet nowadays standards. This is achieved through exploration of the built heritage of the area which had a purpose to form data base and justify the decision of the team to deal with the subject. Testing of the recipes and new construction details will eventually lead to final construction solution for contemporary earth house;

Popularisation of the technique and its adoption for contemporary building method, through demonstration of its performance and advantages; these are to be done through construction standard outline and demo building;

Educational purpose: high enrolment of students in process, specially in design and construction stage; also, education of the public, contractors and all relevant experts involved in construction industry is expected.

3. RESEARCH PROCESS

As previously stated, the scope of research conducted at the Faculty of Technical Sciences is set on revitalization of traditional methods and its implementation into regular building practice. It evolves through several stages by focusing on following issues:

Research of the **rammed earth and adobe building history** in Vojvodina region. Two major objectives are set for this stage: first, to reveal original house configuration in the sense of structure and space distribution; second, to gather and analyze old recipes that will be tested in the following phases of the research.

Rammed earth and adobe heritage inspection on several locations- gathering data concerning date of construction, original plan and remodeling effects through the lifetime, construction type and wall thickness, heating and cooling energy requirements, etc.

Testing of the samples in the laboratory environment. Data gathered will be crucial for the decision of the final recipe;

Development of **new construction techniques** and their implementation into traditional. These include development of the construction details for construction joists, window casings, slab and ceiling construction, foundation techniques, etc.

Preparation of the **Earth Building Standard Outline/Preposition** based on existing foreign one. It will be based on the research of the New Zeeland Standards and its modification to meet Serbian legal constraints.

In the final stage **Demo house** is expected to be built and so demonstrate results in real-time conditions.

4. DATA GATHERING FOR EXISTING STRUCTURES

As mentioned earlier, development of earth building techniques in Vojvodina is long and continuous, mostly related to dwelling typologies and supplementing facilities of the agricultural household. The existing structures are mainly situated in rural settlements-villages and smaller towns, where agriculture is still one of preoccupying professions. The structures where present even in the large scale cities because the plain scenery of the region allowed cities to spread with ground floor dwelling units as the main constituent. After rapid development of the cities after World War Two and densification of the urban areas they are replaced with multi-storey apartment buildings.

For the first stage of research we needed data for existing structures, so we could develop consistent strategy for the future work and set our focus properly. The fieldwork that followed was focused on inspection of the existing structures, interview with the owners, measurement of the wall thickness, window openings, construction details, heating and cooling energy requirements and finally deteriorations of the structure, if any. The fieldwork was conducted in August, and we recorded that none of the inspected houses has air-conditioner, and that occupants are pleased with the indoor climate as it is. The energy consumption for the heating during the winter was rather difficult to compare. Most of the households are old, and comfort temperature differs significantly. Also, none of the houses is heated in whole, so we couldn't compare the results easily, nor make energy per square meter ratio. Most of the houses are heated by gas stove, and even more often by coal stove, which is alternatively heated by corncob and similar agricultural household products. This made the comparison even more difficult.

5. LABORATORY RESEARCH

Information gathering in the field of existing structures belonging to Vojvodinian heritage, followed by detailed processing and comparison of defined features, presented the input process to laboratory researches. Team of experts from Institute of Architecture and Civil Engineering at the Faculty of Technical Sciences in Novi Sad worked on collected information material both theoretically and practically. After carefully established criteria on the basis of those data, material has been appropriately classified and ready for practical investigation.

- Laboratory researches took three parallel paths:
- Testing of mechanical characteristics of elements made of unbaked earth
- Testing of physical characteristics of elements made of unbaked earth
- Reexamine and reevaluating old recipes for elements made of unbaked earth in laboratory conditions

Three types of unbaked earth elements went through testing procedure during mechanical characteristics research that took place in Institute's laboratories during 2005. Samples tested and compared were: those that were taken from old buildings, the hand-made samples, and the samples made by a machine – hydraulic press.

		Characteristical values	
		Density [kg/m³]	Compressive strength [MPa]
Type of samples	Samples from old buildings	1.680-1.820	1.3-2.13
	Hand-made samples	1.283-1.673	0.3-1.25
	Samples made by machine	1.720-2.020	3.3-6.7

Table 1. Results of the compression testing [6]

Further testing of adobe samples made by hydraulic press included changes in composition (additives such as straw and cement in different proportions were mixed with earth), changes in dimensions of adobe blocks (in search for optimum dimensions), as well as comparison between baked and unbaked samples.


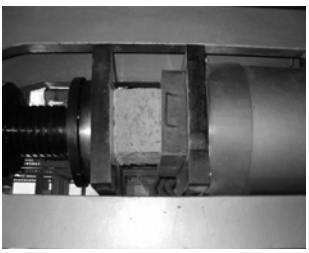


Figure 1, 2, 3.Labory testing

Testing of thermal conductivity followed. An excellent result had been achieved processing hand-made samples circle in basis (diameter 5.5 cm, composition: earth, straw and whitewash). Its thermal conductivity coefficient value was **λ=0.37-0.38 W/mK** which represents an excellent characteristic of building's microclimate. In the following table we made comparison of the thermal features of standard construction materials and proposed one:

Material	Construction thickness d [cm³]	Thermal Conductivity Coefficient λ[W/mK]	Thermal Conductivity Resistance R[m²K/W]
Compact brick	25	0.64	0.391
Compact brick	38	0.64	0.594
Hollow brick	25	0.52	0.481
Hollow brick	38	0.52	0.731
Adobe	40	0.38	1.053
Adobe	50	0.38	1.316

Table 2: Review of thermal characteristics of the samples

Reevaluation of old recipes showed high quality results, with its highpoint in invention of the new patent – **strudel**. Elements of unbaked earth were made according to instructions given by E. Gender in "Vojvodinian country house made of unbaked earth" containing: earth (clay with 20-40% of sand), straw, earthen plaster, and cane. Rolled together around a wooden lath which served as a holder, elements had 15 (17.5) cm in diameter and were 3m long. They showed high compressive strength and satisfying deformability under pressure, which qualified them for building walls of high quality. Tables of results follow:

Samples with diameter d=15cm	Pressure P=16 kN
Longitudinal deformation [cm]	1
Growth in the area of contact leaning in hydraulic press [cm]	from 42x10 to 42x12.5

Samples with diameter d=17.5cm	Pressure [kN]		
	P=20	P=42 (starting from 20)	P=80 (starting from 20)
Longitudinal deformation [cm]	5	7	8.5
Growth in the area of contact leaning in hydraulic press [cm]	from 42x10 to 42x12.5	from 42x10 to 42x14	from 42x10 to 42x17

Table 3,4. Results of the strudel deformation in the press [6]

Achieved results in laboratory testing of elements made of unbaked earth convinced us in justification of this kind of approach in contemporary building. Satisfactory physical and mechanical characteristics of samples, as well as achieved energy savings should be our guides in revitalizing earth materials as one of the holders of sustainable development.



Figures 4,5,6: Strudel testing

6. EARTH BUILDING STANDARD OUTLINE

Final result of the research will be construction standard outline, that will help standards to be adopted in the future and so become one of the concurrent construction technique. At the moment we do not have any document which covers this area, although there is significant built heritage. Reconstruction and remodeling is done in-situ without any regulation or even technical recommendations.

Contemporary practice records earth buildings in different parts of the world, even if they are not integral part of the building tradition. However, this practice is strictly regulated and standardized. For our practice we found New Zeeland Standards to be the most convenient for the starting point of making our own. This will be long and delicate process of thorough investigation, study and testing in order to meet our existing construction standards and finally become their integral part. Only that way we can expect certain adoption of the new construction procedure in every day practice and its acceptance as the relevant one.

7. DEMO HOUSE

After the fieldwork and laboratory testing design brief is planned to be developed. Its preparation started in the earliest stage of research and it depends on numerous factors that need to be reconsidered. One of the major issues that concern architects of the research team is the setting of the new design criteria, and promotion of the forgotten rammed earth aesthetic value. In that course thorough typology analysis is conducted, using available archives and built funds, in order to distinguish appropriate value of modernization that won't endanger typology identity. That is of utmost interest especially in the situation when we are facing complete absence of any understanding of local and traditional in dwelling typologies, which led to anarchic development and devastation of the rural identity. Restoration of the traditional techniques needs to be followed by planned remodeling of the traditional housing types and its refinement to meet nowadays standards and habits of new owners. This will consequently lead to restoring identity on the rural/urban scale and perhaps enhance the people's interest in traditional values.

8. RESEARCH BENEFITS

Concerning theoretical results of our research first step – inspection of the built heritage of unbaked earth in Vojvodina, as well as reexamine of old building techniques and laboratory testing of different elements (research models and constructive elements) made according to old recipes (including research on the recipes themselves – correction of mixtures, adding additives, changing technologies of preparing mixtures, etc.), building the real structure in real conditions presents the next challenge. The idea of making a model in adequate scale in laboratory conditions is also considered. The expectation of constructing a real structure reflects in improvements on important fields:

Improvement of physical and mechanical characteristics of building elements made of unbaked earth – real time measurements of all adequate characteristics (thermal conductivity, moisture percentage, overall energy balance), as well as tracking every changes and comparison with other materials characterize this activity. By reinforcement of earth materials with organic fibers much better mechanical and insulation values are achieved.

Improvements in the field of popularization of advanced building techniques and development of construction methods (inventing new details, joints, interaction with other materials), as well as encouraging improvement of architectural design and practice

Economical improvements – Energy savings are achieved in two levels: in production process, known as embodied energy of the material, since unbaked earth (row material) stiffens by natural drying. Compared to energy requirements for production of ordinary bricks, production of compressed earth walls consumes 5-10 times less energy:

- Adobe consumes 556-1946 Kwh/t, that is 2200 Kwh/m3
- Ordinary bricks (ceramic) consume 6200 Kwh/t, that is 14900 Kwh/m3. Use of unbaked earth material is justified only in local conditions, thus cutting transportation costs [2].

Apart from embodied energy calculation we considered considerable energy conservation that occurs during building exploitation. It is assumed that earth structures consume 30-50% less energy that the classical brick house. Average household in Vojvodina spends some 250 Kwh/m2 annually for keeping the indoor climate pleasant, while we expect for household made of new materials consumption of energy around 130 Kwh/m2. The savings made for energy use in the case of earth buildings are not affecting initial investment what is the case with high-tech solutions [2].

Ecological improvement - Structures made of unbaked earth elements save energy because of great thermal characteristics and at the same time offer better microclimate and healthier living conditions. Possibility of recycling old material represents one more advantage of these structures. Also, construction process is environment friendly and makes the least impact to the surrounding on the macro and micro level.

9. CONCLUSION

Research work presented here focuses on implementation of traditional building techniques and usage of local materials in achieving better living and economic conditions. As today's technologies should gravitate to an ideal of sustainable development, improvements attained through revitalization and reinventions of these activities convince us that low-tech approach accomplishes significant advantage in the field of constructing.

Contemporary trends in architectural design field have shown irrelevant in coping with actual problems. Gravitating to overall globalization architecture nowadays excludes almost any kind of identity- regional, local or personal. Return to traditional building techniques and materials, besides almost revolutionary energy savings, would show results in outlining of high quality regional architecture, improvement of regional values and maintenance of identity of the region.

Social aspect of promoting building with earth materials has also been considered. Simplicity of manual work that is required for building a structure of these elements enables people to build it themselves. Under a specialized supervision, and production of unbaked material in situ (using movable presses), all organization of work should be simplified. Possibility of easy recycling the old material and its quick return into previous state makes this process sustainable.

Institute of Architecture and Civil Engineering at Faculty of Technical Sciences in Novi Sad as a leader in innovating and reinvention of constructive technologies in this region promotes sustainable building techniques and materials, and also encourages further researches on these issues.

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TENTH NATIONAL AND
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NOVI SAD, NOVEMBER 22-24,2006.

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PERFORMANCES EVALUATION OF A VENTILATED FAÇADE

Summary: *The ventilated façades represent a constructive system, often used especially at the buildings developed in the vertical direction, because of the energetic performances and of the favourable behaviour at heat and mass transfer. The paper presents the results of a study concerning the behaviour, during winter conditions, of university buildings that include in the envelope this system too. The analysis with the IR thermography gives the possibility of a quality appreciation of the façade performances and it emphasizes the influence of some factors that have in view the constructive system, orientation and way of operation.*
Key words: *ventilated façade, thermography, numerical simulation*

OCENA PERFORMANSI VENTILISANE FASADE

Rezime: *Ventilisane fasade predstavljaju konstruktivni sistem, koji se često koristi pogotovo za zgrade koje su razvijene u vertikalnom pravcu, zbog energetskih performansi i zbog povoljnog ponašanju pri prenosu toplote. U radu su prikazani rezultati istraživanja koji se odnose na ponašanje tokom zimskih uslova zgrade univerziteta, u čijem je omotaču takođe uključen ovaj sistem. Analiza sa IR termografijom daje mogućnost kvalitativnog razumevanja performansi fasade i naglašava uticaj nekih faktora koji se odnose na konstruktivni sistem, orijentaciju i način eksploatacije.*
Ključne reči: *ventilisana fasada, termografija, numerička simulacija*

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1. INTRODUCTION

The establishing of friendly relationships between buildings and the environment, basic imperative for the diminishing of the energy consumption in exploitation and atmospheric pollution, generated concerns for the developing of new constructive systems and adequate technology for façades realisation. They have to satisfy both the modern architecture requirements and performance requirements, sometimes contradictory, concerning the functionality. For this reason, the façade subsystem must simultaneous provide:

- water-tightness (protection against rain) and air and water vapour permeability;
- incident solar energy use in winter and transition seasons and avoiding overheating during summer period;
- natural ventilation and protection against external noise and pollution;
- freedom for architectural creative conceptions, so flexibility and adaptability.

Also, modern façades must be adaptable of external environment changes from mechanical and physical point of view, through reversible changes of the constituent materials proprieties, using electro-, thermo- and photosensitive processes.

The evolution in time of the façade subsystem reflects the technological progress in strong relationship with energetic context. At each historic moment, the energy context can be defined by the conflict between the level of resources and that of requirements, which, together, determine either a "crisis-dependence" situation or an "abundance-autonomy" situation. Over the first 8 decades of the 20th century the high buildings and light façades, based on steel and glass, were preferred almost all over the world. But, in the present-day energy and environmental context, the concern for energy conservation and to create a healthy and comfortable indoor environment with minimum energy use has become the desiderate for the optimisation of the building envelope solutions.

As a result, facades with predominant opaque parts appeared, made by several layers of different materials, each of them contributing at the partially or totally performance exigencies fulfilment.

The problem of the hygrothermal efficiency and the systems' behaviour at destructive action of the climatic agents was the subject of vast researches at prestigious institutes (CSTB-Paris, Fraunhoffer Institute für Bauphysik – Stuttgart, etc.). The results led to the development of 2 constructive systems for facades:

- ETICS system, that has an external layer for mechanical protection made by a thin plaster, with mineral or organic bonding material, reinforced with a fine network of steel, glass or plastic fibres;
- the ventilated facade system, with an external protection made by panels from different materials (ceramic, glass, natural stone, iron plate, composite materials) disposed at some distance from the thermal insulation. The air flows in the intermediate cavity, improving the hygrohermal behaviour of the facade structure.

The ventilated facade constructive system has a large applicability, especially for high buildings, due to the technological, economical and hygrothermal advantages, such as: costs for execution and maintenance lower than that of the classic façades, condensation risk is eliminated and also the summer overheating is reduced, etc.

At the same time, passive facade systems were developed, specific for solar architecture, based on the greenhouse effect, the black body effect and the air movement through free or forced convection. The „Trombe wall" system, dynamic insulation, diode walls, solar facades, etc. have becomes well-known.

At present time, the “double skin facade” system, organically integrated in the structure and architecture of the buildings, has a large area of utilisation, because it combines the advantages of the ventilated facades with those of the solar facades.
Although the number of the building with ventilated façades is more and more high, the system problems are not completely mastered. The complex functions of the ventilated facades and the regional and local climatic factors have strong influence on their hygrothermal behaviour and energy performance.

In Romania, the ETICS system is widely used in constructive solutions, especially for residential buildings. The double skin facade system, with or without ventilated parament, is used for public buildings, in a small measure. For the first system, the prescriptions and calculus methods permit the energy performances and hygrothermal behaviour evaluation. For the second, there are no data in concordance with the specific climatic conditions. The monitoring of the existing buildings correlated with numerical simulation of the flows and transfer phenomena, could conduct to developing an evaluation instrument and to optimizing the constructive details.

2. THE MONITORED BUILDING AND FACADE SYSTEM PRESENTATION

The paper subject is a university building located in the centre of the city of Iasi, Romania. One of the paper authors designed this building. The small ground surface led to the necessity of developing the building on the vertical, with the distribution of functions in as compact volumes as possible. This is an eloquent example of how the energy performance constraint appears at the interface with other designing constraints, the latter ones having an indirect influence on the energy performance of the building.

The opaque parts of the envelope have been conceived as a ventilated facade, with the following disposal:

- 20 cm thick cellular concrete masonry between the structural elements of the buildings made of monolithic reinforced concrete;
- 10 cm thick Isover rigid mineral wool insulation to the exterior for thermal bridges effect attenuation;
- 60x60 cm Marazzi ceramic panels mounted on an aluminium structure at 4 cm distance from thermal insulation (figure 1).

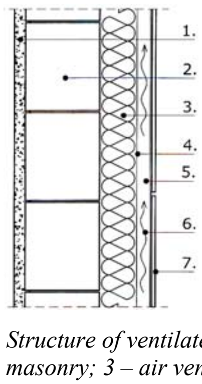

Figure 1. Structure of ventilated façade
1 -inner plaster; 2 –cellular concrete masonry; 3 – air ventilated air layer; 4 – ceramic panels.

The glazed elements are made of aluminium frames and insulating glass. The system ensures a good thermal insulation. Providing some areas that cumulate the greenhouse effect and the buffer zone proves to be an efficient measure in the increase of the energy performance of the building. Such spaces are provided on the four facades, excepting the North-western one, that require visual relations with the central zone of the city. From here, large vistas open towards the city, strengthening the communication function between interior and exterior spaces, with remarkable psychological effects. This is an additional proof that the functionality of the building has not been sacrificed in favour of the energy constraint. The use of the mechanical ventilation system contribute to a better management of resources and, implicitly, to the increase of the energy performance of the building.

3. THE ANALYSIS OF THE VENTILATED FAÇADE BY MEANS OF THE IR THERMOGRAPHY

The IR thermography is a modern technology for teledetection and telemetering of surfaces temperatures being at rest or movement, based on the emission and absorption phenomenon of the infra red radiations. The teledetection instrument – the IR camera – transforms the object image from the invisible spectrum radiations in visible images in white-black or colour. In the buildings sector, the IR thermography allows the visualization and representation of the temperature distribution on the building envelope surface, emphasizing the geometry visualization and area of the thermal bridges influences, the visualization of some imperfections of the thermal protection, areas affected by moisture, unheated areas or with stagnant air, etc.

The IR investigation of the University “Petre Andrei” building from Iaşi was made during the winter of the year 2005, when the sky was opened and the exterior temperature was -4.5ºC. Inside the air temperature was approximately +22ºC. There were get images both from inside and outside the building. The examination of the IR images gives information concerning the temperature distribution on the façades surfaces, whose structure is a ventilated one, and inside, in the areas with geometric discontinuities, emphasizing the following important aspects:

a. The route and the influence of the linear thermal bridges determined by pillars and girders (figure 2).

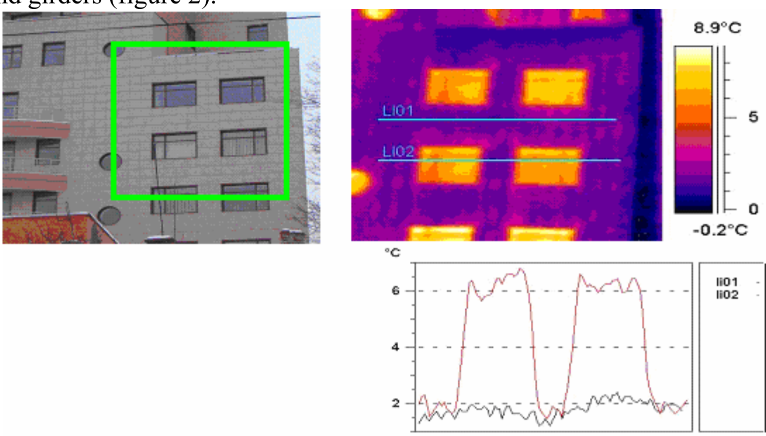

Figure 2 Thermographic image; Northern façade

It may be noticed that by a certain treatment of the thermal bridges, by putting on the exterior of a mineral wool layer of 10 cm thickness, reduces the temperature differences compared with the façade current field (ΔT = 1.5°C). The relative image is an emphasizing of the heat losses through the glazed areas.

b. The geometric discontinuities influence (figure 3). The presence of a console in contact with the exterior air on 3 sides determines a temperature difference of 4°C compared with that of the field.

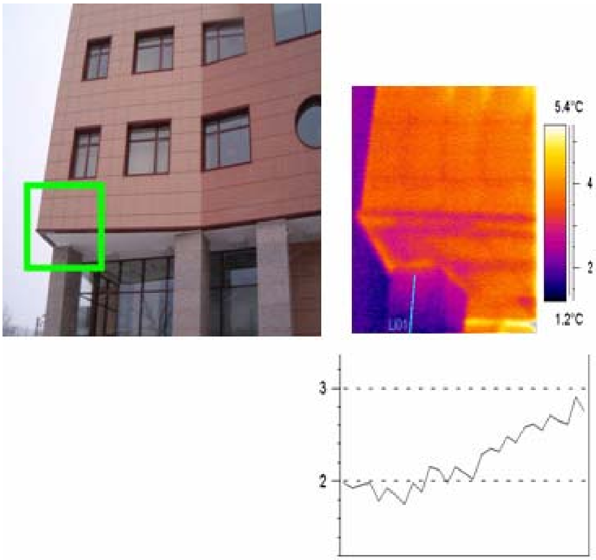

Figure 3. Thermographic image of a geometric discontinuity

The punctual thermal bridges determined by the presence of fixing devices of the ceramic plates against the structure (figure 4).

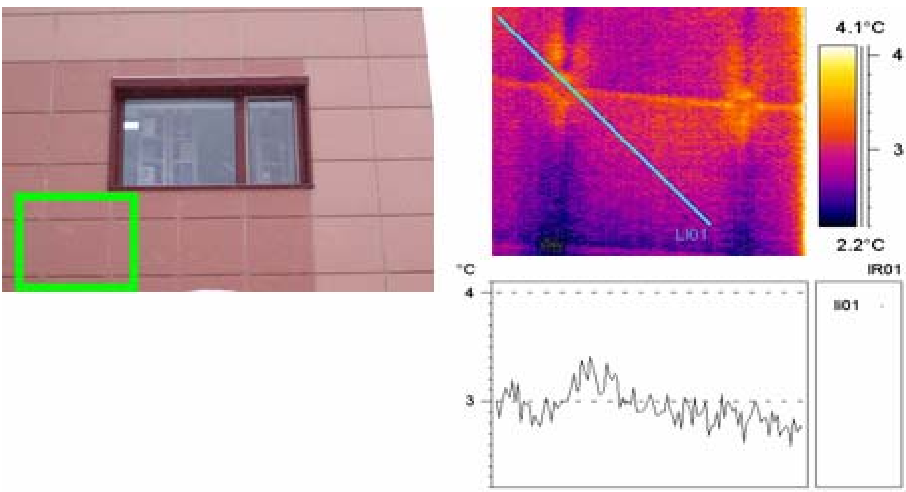

Figure 4. The influence of the punctual thermal bridges

The decreasing of the thermal resistance determined by all these is not important (a difference of 0.5°C compared with that of the field) because of the thickness and the continuity of the mineral wool layer.

d. The presence on the interior walls surfaces of some low temperatures because of the geometric configuration.

4. NUMERICAL SIMULATION

The IR thermography does not allow the effective assessment of the thermal insulation of the façade, because the surface temperature depends not only by the heat flux that traverses the wall, but also by the heat transfer between the surface and the exterior. Besides that, a ventilated façade appeared always as being cold, because the veneering is permanently ventilated by the exterior air. An assessment of the thermal performance by means of numerical simulation is required.

It was adopted as performance criterion the adjusted mean thermal resistance on the façade, R', respectively the adjusted thermal transfer coefficient, U', determined with the relations:

$$U' = \frac{1}{R} + \frac{\sum(\Psi \cdot l)}{A}$$

Where:

R is the thermal resistance on a single direction in the façade areas unaffected by thermal bridges;

Ψ is a correction coefficient of the linear thermal bridges (figure 4).

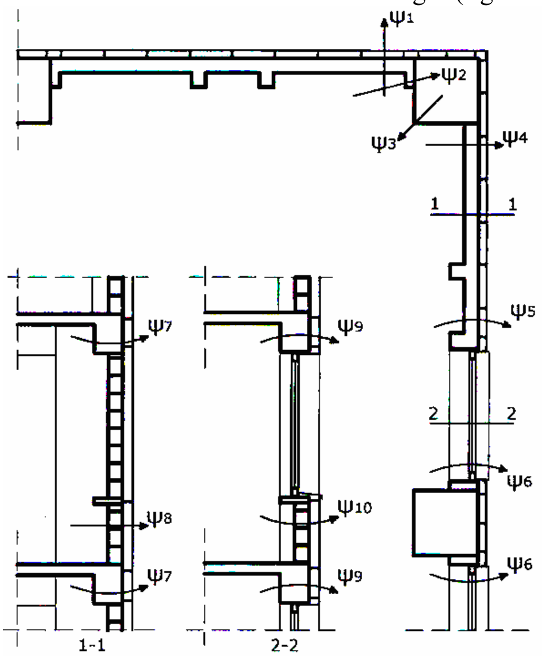

Figure 4. Areas of thermal bridges; adjusted coefficients

A is the area of thermal transfer

The correction coefficient related to the thermal bridges, ψ, have been determined on a simulation base of the plan thermal field in the thermal or geometric discontinuity areas (figure 5).

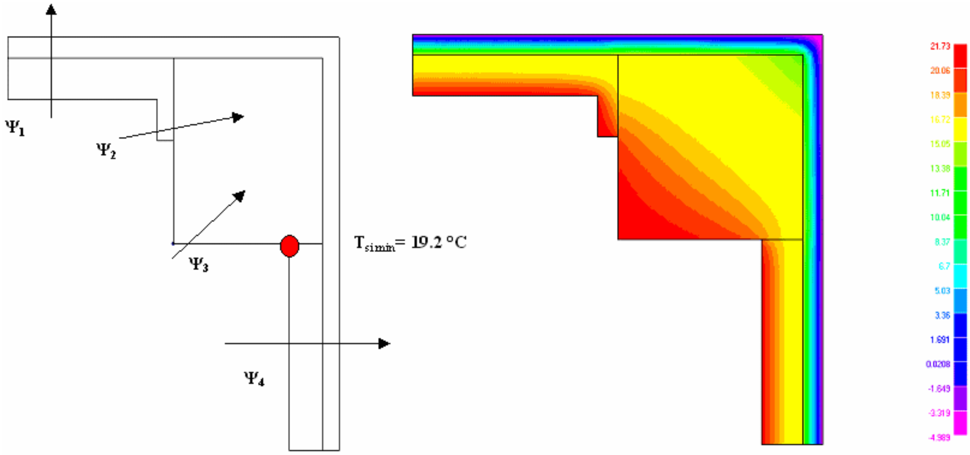

Figure 5. Numerical simulation results; example

The effect of the ventilated air layer has been taken into account through a low value of the thermal transfer coefficient by convection and radiation on the exterior surface. The calculus has been made for the following situations:

- opaque corner bay;
- corner bay with window;
- opaque current bay;
- current bay with window.

The results are presented in the table 1.

Zone (bay)	A m^2	U W/m^2K	ΔU W/m^2K	U' W/m^2K	R' m^2K/W
Opaque corner bay	21.00	0.293	0.100	0.393	2.540
Corner bay with window	18.11	0.293	0.410	0.703	1.420
Opaque current bay	21.00	0.293	0.081	0.374	2.670
Current bay with window	18.11	0.293	0.404	0.697	1.434

Table 1. The thermal performance of the ventilated façade

The results emphasize a mean thermal resistance value of the façade that is superior of the specific normal values in Romania (1.4 m^2K/W for the opaque walls). A big contribution has the ventilated air layer that diminishes the air change by convection.

5. CONCLUSION

The assessment of the energy performance of buildings implies an analysis for each element of the envelope, the façade having the most important role.

The thermographic and numerical simulation of the temperatures plan field reveals a high thermal insulation level of the façade, because of the air layer included in the structure, too.

Acknowledgements

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TENTH NATIONAL AND
FOURTH INTERNATIONAL
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NEW CLOTHINGS FOR OLD AND DEGRADATED HABITATION BUILDINGS

Summary: *The rehabilitation and modernization of the habitation buildings in prefabricated great panels is badly needed, if structural stability is assured, considering their great incidence in the economy of urban habitation in Roumania. This is due to the current status of envelopes, the condense-and misuse-related degradations, their functional rigidity and the disagreeable architectural plastics, as well as to the huge energetic consumption during exploitation. Our paper reviews advantages and potential inconveniences of various modernization solutions aimed to substantially reduce the energetic costs - the double skin facades -that also represent an attractive solution from the point of view of volumetric and plastic remodeling of these buildings, whose aspect is currently a significant urban stress factor.*

NOVE OBLOGE ZA STARE I DOTRAJALE ZGRADE ZA STANOVANJE

Rezime: *Rehabilitacija i modernizacija zgrada za stanovanje sa velikim prefabrikovanim panelima je neophodna ako je osigurana stabilnost konstrukcije, uzimajući u obzir njen uticaj na ekonomiju urbanih sredina u Rumuniji. Ovo je zbog trenutnog stanja omotača zgrada, kondenzacije i oštećenja usled zloupotrebe, funkcionalne krutosti, i neslaganja sa arhitektonskim detaljima, kao i zbog velike energetske potrošnje tokom eksploatacije. Naš rad daje pregled prednosti i potencijalnih poteškoća pri različitim modernim rešenjima radi smanjenja troškova energije - dupla površinska fasada - koja predstavlja atraktivno rešenje sa aspekta zapreminskog i plastičnog re-modeliranja ovih zgrada, koje trenutno predstavljaju značajan urbani faktor stresa.*

In most important Romanian cities, there are numerous habitation buildings built with prefabricated great panels. According to the Background of the B12 Study of „Urban habitation”, afferent to the General Urban Development Plan of Iasi, the tall and medium collective habitation buildings represent the most common type, in respect to the number of apartments –88.9% of habitable spaces, respectively, and 22% of the total buildings counted. Out of the 2220 blocs, 1684 are built with concrete frames, diaphragms and great panels and only 552 of bricks. Out of the former ones, approximately 70% have prefabricated panel structure (4, 9).

A critique analysis of structure and behavior of these buildings led to the following observations concerning the security requirements and the space-volumetric structures:

Security requirements are, mostly, correctly satisfied. Building stability or that of components are insured. Due to the structure conception, there are no records of progressive collapse phenomenon, not even after large-scale earthquakes or in the case of explosions and fires, the observed degradations and fissures not being severe enough to threaten the safety of the buildings (3, 7).

With regard to habitation safety, due of the difficulties caused by the water accumulation and sometimes due of the users habits or overpopulation, there are to be notified dysfunctions that might endanger the health of the habitants. The higrothermic behavior is the main vulnerability of this building type (1).

Due to economic reasons and the quasi-ignorance of necessity of energy preservation, the thermal protection and the exterior concrete layer of the walls were dimensioned, for longtime, without the concern to ensure a minimum yearly consumption of the building using a proper thermal protection of the envelopes (6).

Calculation of the global coefficient of thermal insulation for one of the often used sections in Iasi (exterior three layered precast walls - armed concrete - mineral wool - armed concrete – terrace roofs with granulite thermal insulation) indicates that the medium thermal resistance of the outer cover (0,58 m2K / W) does not even satisfy the exigencies of the standards for 1984, much less the recent changes in the law (13). The same calculation also reveals that, out of the total surface of the exterior walls, the opaque surface, having the aforementioned constitution, with a thermal resistance of 2,023 m2K/W, represents a mere 40%, while the areas that represent thermal bridges (linking ribs, R=0,203 m2K/W -22,3%, horizontal fixed joining, R=0,755 m2K/W – 8,2%, vertical fixed joining, R =1,530 m2K/W-12,3%) –42,8%, whereas the glass areas have an R = 0,49 m2K /W- 17,2%. Thus, the constructive solution ignored, at the very moment of completion, the exigency of saving energy.

The flats made out of large panels have suffered, over significant periods of time, from the condensations of vapors created by circumstances related to closures, climactic variation and the usage of the flats overpopulation, lack of heating, insufficient ventilation) (7). This explains the dissatisfaction of the flat owners and their desire to bring improvements to their apartments (8).

The errors committed in calculating the risks of condensation, through design (the closing up of ventilation) or through the usage of the flats, have had serious consequences. Uninspired interventions (the replacing of carpentry that was ensuring an acceptable level of ventilation with an air-tight system) have increased the effects of condensation and as well as the risks to the health of the inhabitants (10).

Excessive exposure to the sun is a serious deficiency in the case of these constructions whose facades, lacking balconies and protective elements against the sun prove a real reason for discomfort during the summer months (2, 5).

Given the reduced dimension of the rooms in these structures, the lighting exigencies are satisfied, even if the empty spaces seem reduced in terms of their surface in relation to the economic of the outer layer, due to the necessity for reducing the number of typical-dimensions of the windows.

The exigencies concerning phonic isolation in living space are only partially satisfied, given the weight of the components, that in the light of the transportation logistics and assembly, has to be reduced in the case of this type of construction. The capacity for phonic isolation of the concrete floor panels and the interior panels to impact noise (in many instances 14 cm wide) is insufficient, as is the capacity of phonic isolation from aerial noise of carpentry attached to the exterior wall panels. The supplementary thermal isolation and the improvement of tightness would undoubtedly also generate an amelioration of the acoustic comfort (1).

Based on psychological principles, the exigencies of the interior space rely in the dimensions of the rooms. As much as this was allowed by the spatial regulations, the standardized sections have coherent functional structures (12). The advantaged of lacking pillars and grinds is tempered by the small sizes of cells within the diaphragms and by the impossibility of a flexible functionality, which allow the adaptation of the living space to the evolution of the owner families.

The esthetics of the prefabricated great panel buildings is by far the major source of criticism. The reduced number of tipo-dimensions which imposed to the producers by the installation necessities of prefabricated great panels, the cvasi-imposibility of acceptable refinishing of panels damaged during transportation and installation, are the main causes of the bad renowned of the great panel buildings within architects and users. Even if such an attitude is exaggerate, that great panel buildings are very resistant to any adaptation because of the platitude of the facades. Therefore, one has to admit that hard prefabrication does not allow a high quality in architecture. Alvar Aalto named great panel building “industry aggression over individual taste”, an invasion of ugliness that may result in perverted perceptions and a source of daily stress (3).

Sociological exigencies, such as intimacy, communication, adaptation to different life styles are the most hurt by the great panel building ensembles. In general, these are groups of cheap social flats, built in short time and careless. This type of buildings, present on important street axes, is a source of depression for the community.

A potential long-term approach to highro-thermic and rehabilitation modernize of these buildings might consist of using double-skin facades, a contemporary interpretation of some traditional solutions. The advantages of this approach consist of increases in phonic insulation, increase in the ventilation potential of windows, and overall increased comfort (11, 14).

Double skin facades are based on multistratification, consisting of an external façade, an intermediate space and an internal façade. The external layer will contribute to protection against climate conditions and will improve the acoustic insulation. It also contains openings that allow the ventilation of the intermediate layer and of interiors. Air circulation in the intermediate layer is activated by the ascending thermic forces induced by sunlight and wind effects. To obtain a better adaptability of climate changes, external layer wholes can be closed if needed (Figure 1)

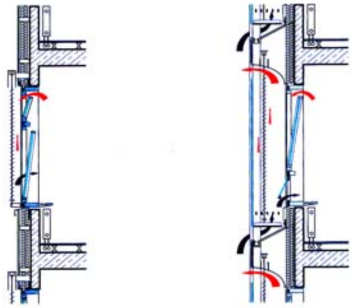

Fig.1 Comparison between single and double skin facades

The external layer of the double skin facades generally consists of a highly resistant glass layer. An adjustable protection system is generally installed in the intermediate layer to protect the internal layer from insolation. The internal layer consists of a double carpentry that protects against heat loses during the winter period. In the majority of cases, the interior layer can be open to allow natural ventilation.

Double skin façades can be classified based on the segmentation of the intermediate layer to obtain different ventilation patterns.

There are 4 types of double skin facades: box-window, shaft-box window, shaft façade and multilevel façade (14). Of these, two can be considered in order to modernize the existent facades that do not satisfy the comfort and economic contemporary exigencies: box-window and shaft-box. There two variants also have the advantage of an easy installation.

Box-window façade is the oldest form of double layer façade. The box-window consists of a sheath with two layers of windows with interior opening. The external window has a single layer of glass and contains openings that allow ventilation of both intermediary layer and interior spaces. The cavity between the two layers is horizontally divided along the building axes at the levels of inferior and superior cotes of the rooms. On a vertical section, the divisions are placed at the level of separating walls as well as at the level of each window. The continuous separation elements prevent transmission of sound and odors between cavities and between rooms (Figure 2).

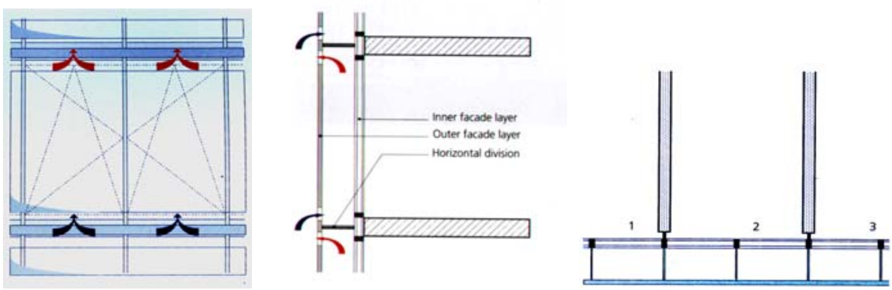

Figure 2. Box-window-elevation, section and plan

Box-windows are currently used when there is a strict regulation of external noise and when there are special needs for isolation between adjacent rooms. This is the only system that can achieve this function on conventional facades, with rectangular openings (14). Each element of the box-window needs its own whole for air entering and elimination. These outer openings have to be considered at the time when the external façade is projected.

Shaft-box window is a special form of box-window construction based on the "double-face" principle and consisting of a system of box-window with a continuous vertical shaft encompassing several floors thus realizing a chimney effect. The façade will consist of an alternance of windows with chimney segments. At each floor, the vertical shafts are connected with the adjacent box through an opening that allow air flux (Figure 3).

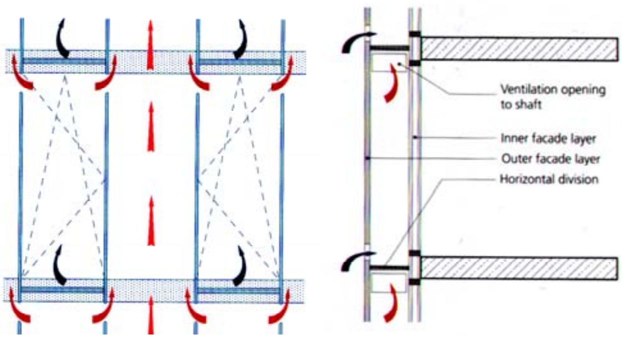

Fig.3.Shaft-box window - elevation, section and plan

Due to the chimney effect, the convection flux is conducting the air from the windows in the vertical shaft and upper, where it is eliminated. In addition, the ascension of warm air can be mechanically facilitated.

The shaft-box facades use less outer openings due to the use of the chimney effect (14). This results in a better phonic insulation. In practice, the height of the shafts is necessarily limited, this type of approach being adaptable to small and medium buildings. An aerodynamic adjustment is necessary if all the window-boxes connected to a given shaft need to be equally ventilated.

The double skin facades can be conceived in these cases in which the buildings are exposed to extremely intense aerial noises or severe winds, such as those cased in which inhabited buildings are rehabilitated (1). These solutions can be used for tall, medium and small buildings. If buildings are to be ventilated most of the year, double skin facades are of choice.

Double skin facades have a special esthetics that can be widely exploited by architects. The visual impression of transparency and depth, corroborated to the external layer structural form, opens large avenues for diverse designs. Obviously, this approach can solve the disorder and platitude issues of the facades of prefabricated great panel buildings, without any trouble for the inhabitants.

Figure 4. Existent façade versus modernized double skin facade

If the modernizing intervention is aimed at transforming the terrace in a garden-terrace or a habitable attic, the global coefficient of thermal insulation can achieve levels concordant with the current standards. The over basement bridging, as well as the sideways (doubled by air absorption crush barriers) are also included in the modernization process, being needed by the double skin facades.

Building a new layer on the façade involves additional outlay. These outlays are balanced by better insulation that reduce the heating and cooling costs. Thus, a double skin façade may reduce the use of air conditioning, the window ventilation being sufficient, at least for some of the warm months of the year. Moreover, the external layer bring significant amelioration of the phonic insulation, allowing a very simple configuration of the internal layer (i.e., dual layer wood carpentry).

Studies have shown that the long-term costs for a double skin façade with mechanical ventilation are lower than those for a single skin façade and interior air conditioning.

The energetic savings calculated for double skin facades is the more impressive as the initial level of insulation of the single skin structure was low. Some of the energy savings are diminished by the fact that the light transmission is poorer due to the addition of a new layer of glass. In these cases, a larger surface of the opening can compensate this inconvenient.

The improvements in phonic insulation by double epidermal depends on the magnitude and position of the openings. Moreover, the phonic insulation may be influenced by absorbent surfaces of the intermediate layer. Considering a total opening surface of 10%, the quality of phonic insulation of the internal layer will be improved by 3-6 dB. If the openings represent only 5%, the improvement will be of 10 dB.

Finally, this type of facades can result in a short-term reduction of wind-induces fluctuations. This is facilitated by the buffer effect of the intermediate layer.

Conclusions

The analysis of the configuration and exploitation of habitation buildings built with prefabricated great panels generated the following conclusions:

1. The decision of using this type of buildings was, at the time of their achievement, justified by the advantages of prefabrication (building of a large number of apartments in a short time, high productivity, low costs, low qualified personnel).

2. Current status of envelopes, degradation produced by condensation and vicious usage, functional rigidity, low energetic effective power, all suggest, at an initial appraisal that the service of these buildings is exhausted and that, in principle, the best approach would be a radical solution consisting of their replacement with habitable buildings more adapted to current needs. Due to their large number, this solution is currently not feasible, a large part of the big city population depending of this type of habitats. Therefore, rehabilitation is compulsory. This rehabilitation should be organized in a manner aimed to prevent any aggravation of social pathology related to this major issue.

Structural stability is achieved, therefore consolidation approaches are not necessary. The current interventions should aim transforming the envelope in a manner that can reduce substantially energetic costs, a very important issue, especially when considering the social character of this type of habitats. The double skin facades may constitute an acceptable solution of modernization. The energy savings, a better phonic insulation, a better air circulation are some of the advantages of this approach that can justify the financial efforts towards modernizing structures otherwise obsolete. Last but not least, double skin facades may improve the aesthetics of a large number of buildings and initiate a needed change of the aspect of urban areas.

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EASTERN HARBOUR DISTRICT AND IJBURG – REDISCOVERING THE CITY

Summary: *The aim of this paper is to emphasize importance of typo morphological researches, connection between type and morphology and their mutual relationship, as integral part in process of urban planning and architectural design. It will explore how continuous variation and reinvention of existing urban elements became one of key elements in innovative planning of Eastern Harbour District (Borneo and Sporenburg) and IJburg (Haveneilanden and Rieteilanden). Both analyzed districts introduce variability which is achieved by continuous variation of existing urban structures. This is the characteristic that distinct them from average VINEX district and introduce multi-layered periphery.*

Key words: architectural typology, urban morphology, urban planning, housing, Amsterdam

EASTERN HARBOUR DISTRICT I IJBURG – PONOVNO OTKRIVANJE GRADA

Rezime: *Cilj ovog rada je da istakne značaj tipomorfoloških istraživanja, veze između arhitektonske tipologije i arhitektonske morfologije i njihovih međusobnih odnosa,kao sastavnog dela procesa planiranja i projektovanja. Rad će istražiti kako stalne varijacije i ponovna otkrivanja urbanih elemenata postaju jedan od ključnih elemenata u inovativnom planiranju Eastern Harbour District-a i IJburg-a. Obe analizirane četvrti se odlikuju raznolikošću koja je postignuta varijacijama postojeće urbane strukture. Upravo ova karakteristika čini da se ove četvrti razlikuju od klasičnih VINEX naselja.*

Ključne reči: arhitektonska tipologija, urbana morfologija, urbano planirananje, stanovanje, Amsterdam

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1. INTRODUCTION

City form is changing constantly. Interaction between that which was once built, and how it has been changed by use and rebuilding, became one of key element in process of urban planning and architectural design. In decades after war, we have been witnesses of discontinuity and radical break in planning and designing the city. Urban block has been changed, perimeter block disappeared, streets became traffic arteries, and, as result, the relations between private and public domain became totally different. That obvious discontinuity between traditional city and modern city became source of problems. Transformations of housing as predominant architectural type have strongest influences on urban form and other architectural types. Because of that, today, we can see contemporary housing as tool in effort to re – evaluate traditional urban form, and use those values as source of inspiration in design process.

2. AMSTERDAM – REDISCOVERING THE CITY

The history of architecture and urbanism in Amsterdam show us how expansion, renewal and continuity can go hand in hand. After arrival of Housing Act in 1901, H.P.Berlage plan for South Amsterdam presented in 1916, inject high – powered structure in inner city of Amsterdam. Until that moment division of parcels of land into building lots was the model for structuring urban block. Growth of the city based on small – scale private initiatives was no longer sufficient in the beginning of 20th century. Berlage's plan introduces new type of urban block in which dwelling or flat subordinate to whole of the block. Collective stair and entrance was introduced. But at the same time architects of the Amsterdam School tried to preserve that each dwelling has entrance for the street level whenever it was possible. The «Hague porch» and «front door battery» were examples of continuum of private entrance, as one of the important elements of 17th throughout 19th century city.

Picture 1. Housing block with «Hague porch» units

Picture 2. «front door battery» housing

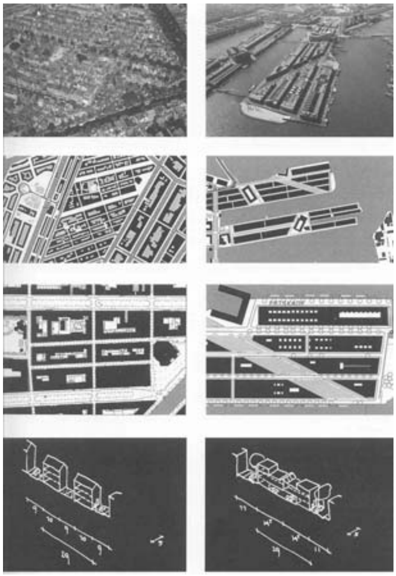
The plans for transformations and expansions of the city of Amsterdam since 1990s reveal that they are inspired by existing city. They reveal that relations between historic tissue and the new tissue is complex. Contemporary demands set new basic conditions and produce variations in urban forms, dwelling types and styles. KNSM island part of Eastern Harbour District planned by Jo Coenen is sort of elongated variant of Berlage's Amsterdam Plan – South. But demands to accomodate big number of dwelling units, narrow site lead to more like grand manner 19th century plan, with monumental blocks and broad boulevard in the middle of the site.

Picture 3.KNSM island and Java island

Sjoerd Soeters, on the other hand, for his design of the Java island as an inspiration took 17th century ring canals, which looks like better choice for the narrow site. North and south side, longer ones, are occupied by apartment buildings with collective stairwell with a lift. Long facades are intersected by four small canals, along which individual houses were built. Elements such individual houses with back yards, sidewalk zones, stairs, differences in the level, bridges over the canals, designed for only one reason to create old part Amsterdam streetscape. Urban structure of Borneo and Sporenburg (Eastern Docklands) and IJburg with differentiated urban blocks composed of individual lots is mixed with a layer of large blocks standing out like monuments. Both these plans reinvent urban structure of 17th century Amsterdam with dense urban tissue with public buildings as architecture entities.

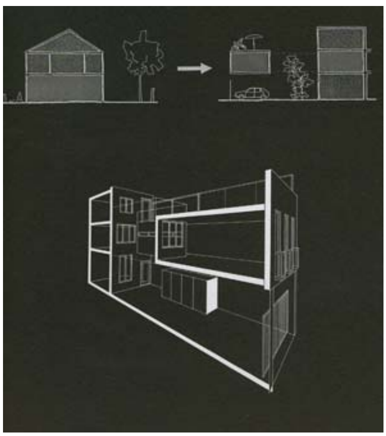
2.1. Borneo and Sporenburg

The peninsula Borneo and Sporenburg were built between 1996 and 2000 as last part of Eastern Docklands «archipelago». Plan made by Adriaan Geuze from West 8 combine low – rise and high – density buildings and large blocks. Because of narrow site, high density of 100 housing units pre hectare and need for individual housing units Geuze proposed long rows of back to back dwellings with patios or roof terraces and three large residential blocks. Comparing this remarkable urban plan with urban structure of 17th century district Jordaan in Amsterdam, we will find great similarities between these to urban structures. Just as in 17th century city public spaces consists of quays, streets and cross streets, the block is built by individual units, large public buildings on squares transformed to sculptural apartment blocks.



Picture 4. Comparative overview of Jordaan and Borneo/Sporenburg

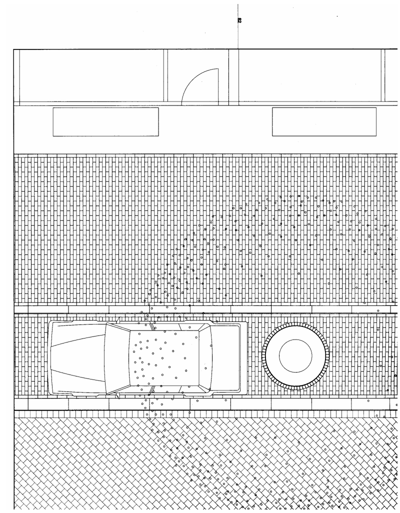
The biggest assignment was to design individual dwellings. Houses share both its side and back walls with its neighbours. The type that was developed can be read as interpretation of 17th century front and back house. In both cases main living floor lies on the first floor of dwelling.



Picture 5 .Metamorphosis of 17th century house into new type of dwelling- patio house

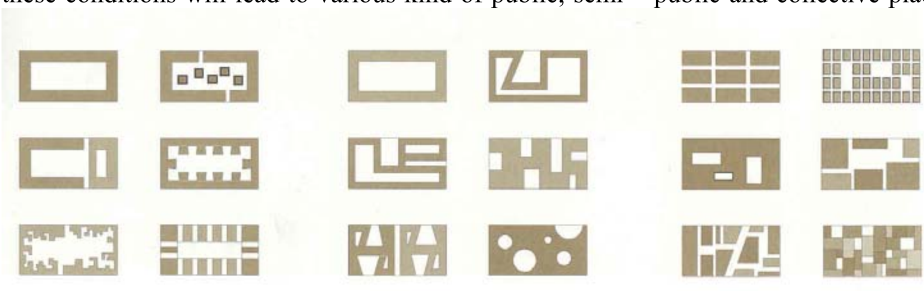
2.2. IJburg – Haveneiland and Rieteilanden

The urban design for Haveneiland and Rieteilanden in the IJburg archipelago consists of grid of urban blocks and variety of streets and side streets, boulevards, canals and squares. Designers Felix Claus, Frits van Dongen and Ton Schaap introduce grid plan where is the contrast between street and block essential. Streets are planned to adopt structural characteristics of 19th century street. Streets are form 30 meters to 22 meters wide. Housing on the street is to be in closed perimeter block, street is formal and has high of three stories and higher, houses on the street should have entrance on the street side, blocks have to be surrounded by a margin of 1.2 meters as they are along canals as transition between public and private. The streets are public spaces in Amsterdam in sense of term, living is lifted half meter or more above ground like on the Amsterdam canal rings.



Picture 6 . 19th century Amsterdam street   *Picture 7. New planned street in IJburg*

The grid of streets defines city blocks. Programme of the blocks consist of a mix of dwelling categories, commercial and general facilities. Architects should find solutions to accommodate all the functions. All these rules should contribute to continuity of the street scene (formal) and different structures of block (informal).Designers believe that these conditions will lead to various kind of public, semi – public and collective places.



Picture 8 . Possible strategies for accommodating the programme in the block

3. CONCLUSION

The newly developed urban district in Amsterdam is based on reintroduction of old city urban profiles. They are not copies but reinvented structures made by architects who take old tissue which fascinate them and produce new tissue of their own. Those designs as exercises for development of new designing methods which will introduce new types of public spaces, urban structures, dwelling types, and show how change can go with continuity.

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FLUID - DAM INTERACTION UNDER DYNAMIC EFFECTS

Summary: *When dam reservoir system is subjected to dynamic loads the dam together with the banks and river bad start vibrating. These motions induce into the fluid compression and dilatation waves. The inertia forces of agitated water are well known as hydrodynamic forces. The intencity of the hydrodunamic pressure depends on the geometry and the shape of the structure and the canyon as well as deformability of the whole system. This phenomena was explained and for the first time treated by Westergaard, Zangar, Napetvaritze and Clough. However their solution is under the assumption that the motion of the fluid is a stationary phenomenon whereat the dam and the reservoir are rigid while the effect of the hydrodynamic pressure is applied on the structure by means of virtual or added mass. In this paper more sofisticated solution is given using boundary element method. The aplication of this metod results in fast and more acurate caluculation of these pressures while their intencities are dependon on the developed accelerations during the dynamic responce of the dam-reservoir system. Based on the boundary element method, the software package BEL3 has been coded.*

Key words: *hydrodynamic pressure, dam reservoir interaction,boundary element*

INTERAKCIJA FLUID - BRANA U DIMAMICKIM USLOVIMA

Rezime: *Kada je sistem brana-rezervoar pod dejstvom dinamičkih sila, brana zajedno sa obe obale i korito reke počinju da vibriraju. Ova pomeranja indukuju u fluidu kompresivne i dilatacione talase. Inercijalne sile u uzburkanoj vodi dobro su poznate kao hidrodinamičke sile. Intenzitet hidrodinamičkog pritiska je zavistan od geometrije i oblika konstrukcije brane i kanjona, kao i od deformabilnosti celog sistema. Ovaj fenomen bio je objašnjen i prvi put obrađen od Westergaard, Zangar, Napetvaritze i Clough. Ipak njihovo rešenje je pod pretpostavkom da je proces stacionaran i da je brana nedeformabilno telo pri čemu se efekat hidrodinamičkog pritiska uzima u analizu prema principu virtuelnih ili dodatnih masa. U ovom radu obrađeno je sofisticiranije rešenje koristeći metod graničnih elemenata. Primena ove metode daje brži i tačniji proračun ovih pritisaka, gde su njihovi intenziteti u zavisnosti od pojave akceleracija za vreme dinamičkog odgovora sistema brana – rezervoar. Koristeći teoretske principe metode graničnih elemenata kompjuterski program BEL3 je razrađen.*

Ključne reči: *hidrodinamički pritisak, interakcija brana fluid, granični elementi.*

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1. INTRODUCTION

In conditions of dynamic effects, an interaction between the water fluid and the dam takes place resulting in occurrence of additional hydrodynamic pressure upon the dam body. The intensity of the hydrodynamic pressure is variable in the course of the seismic effect. This phenomenon has been treated since the time of Westergaard, who gave, for the first time, a physical and mathematical explanation of the phenomenon based on certain assumptions. Later, this phenomenon has been created by a number of authors among whom Clough and his collaborators, who updated the concept of additional mass that was originally proposed by Westergaard. All the solutions are mainly conceptualized based on the assumptions that the water fluid is ideally incompressible and that the motion of the fluid in the reservoir is a stationary phenomenon. Since the water mass is considerably more inert in respect to the ground motion, the motion of the water particles is characterized by relatively small velocity amplitudes wherefore it can be treated as a kind of a stationary wavy motion. Unlike the motion of the water fluid, the motion of the dam body is characterized by considerably higher values of total accelerations and relative velocities. If it is assumed that the dam body represents an ideally rigid structure, then all the points in the dam body are characterized by the same total accelerations as those of the soil. However, there is no ideally rigid body in reality, i.e., the dam body has a certain flexibility so that the total accelerations in individual zones of the dam have different values and are defined as a sum of the ground acceleration and the relative accelerations that take place as a result of the dynamic response of the dam.

For the stationary type of motion of the incompressible fluid, the main equation describing the process of motion is the Laplace-s differential equaion of the second order, with the functions of the boundary conditions:

$$\frac{\partial^2 W}{\partial x^2}+\frac{\partial^2 W}{\partial y^2}+\frac{\partial^2 W}{\partial z^2}=0 \qquad \Omega=\Omega(x,y,z) \qquad 1.1$$

$$\frac{\partial W(x,y,z)}{\partial n}=\frac{\partial \overline{W}(x,y,z,t)}{\partial n} \in \Gamma_2 \qquad W=W(x,y,z,t) \quad \in \Gamma_1 \qquad 1.2$$

The Laplace's equation can be solved by applying the finite difference method, the finite element method and the boundary element method. This paper deals with solution of the Laplace's equation by using the boundary element method. The concept of realization of fluid – dam interaction is proposed. Namely, the boundary element method is applied for relatively fast solving of equation 1.1, with the corresponding boundary conditions 1.2. The principle of unit total acceleration in the direction of the normal to the surface at selected point M is used and a matrix of fluid-dam interaction is formed. This matrix participates in the differential equation of motion of the system. Presented in the paper is a brief review of the theoretical assumptions of the solution by interpretation of a typical equation of boundary elements for point M that belongs to the number of points located on surfaces Γ_1 and Γ_2 of the model of boundary elements. Based on the typical equation of boundary elements, the software package BEL3 has been coded.

1.1. Model of Boundary Elements with Description of Boundary Conditions

The equation of motion of the fluid is expressed through the function of the hydrodynamic pressure of the fluid $W = W(x, y, z, t)$ and presents the distribution of the hydrodynamic pressure in space Ω. The solution of equation 1.1 is not possible without defining the functions of the boundary conditions. There are two different types of boundarty conditions. These are the following:

- Boundary conditions of the "essential type", when the values of the function of the hydrodynamic pressure $W = W(x, y, z, t)$ are assigned for some of the contours Γ_1 of domain Ω , Fig(1).
- Boundary conditions of the "natural type", when the values of the derivations of the function of the hydrodynamic pressure are assigned for some of the contours Γ_2 of domain Ω, Fig(1) in the direction of the normal to the surface.

$$\frac{\partial W(x,y,z)}{\partial n} = \frac{\partial \overline{W}(x,y,z,t)}{\partial n} \qquad 1.3$$

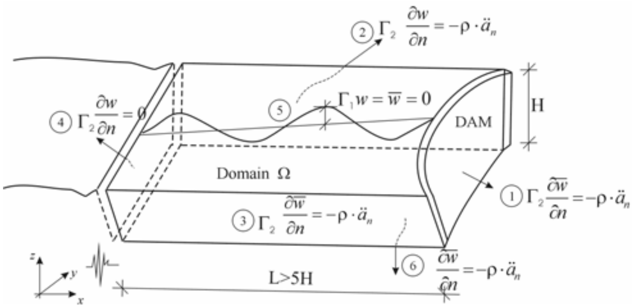

Fig.(1) Model of boundary elements

The derivation of the function of the hydrodynamic pressure along the normal at each point of the considered boundary area of the domain is expressed by the following equation:

$$\frac{\partial \overline{W}(x,y,z)}{\partial n} = -\rho \ddot{a}_t(x,y,z) \qquad x, y, z, \in \Gamma_2$$

1.4

Where : ρ – fluid density $\rho = \dfrac{\gamma}{g}$ [kNsec^2/m^4]

γ [kN/m3] – bulk density of the fluid
g [m/sec2] – ground acceleration

$\ddot{a}_t$ [m/sec2] – total acceleration in the direction of the normals to the points belonging to surfaces Γ_2

Contour 1, i.e., the dam has a boundary condition of the type of Γ_2 , because at each moment of the dynamic response of the dam, the points that belong to it have known values of total accelerations in the direction of their normal. So the following equation holds:

$$\frac{\partial \overline{W}(x,y,z,t)}{\partial n} = -\rho \ddot{u}_t(x,y,z,t) \quad x,y,z,\in \Gamma_2 \qquad 1.5$$

Contours 2, 3 and 6, i.e., the left and the right bank and the bottom of the reservoir also have a boundary condition of the type of Γ_2 , i.e., :

$$\frac{\partial \overline{W}(x,y,z,t)}{\partial n} = -\rho \ddot{u}_g(x,y,z,t) \quad x,y,z,\in \Gamma_2 \qquad 1.6$$

In this expression, $\ddot{u}_g$ represents ground acceleration.

On contour 4, there exist boundary conditions of the type of Γ_2. This contour is formed as an assumed vertical cross-section through the water fluid at a relatively greater distance from the dam body. On this contour, the total accelerations of the fluid particles have a low value so that it can be considered that the following condition is satisfied:

$$\frac{\partial \overline{W}(x,y,z,t)}{\partial n} = -\rho \ddot{u}_t(x,y,z,t) \cong 0 \;\; x,y,z,\in \Gamma_2 \qquad 1.7$$

Acceleration $\ddot{u}_t$ tends to zero with distance from the dam, i.e., theoretically the acceleration is zero when distance L tends to infinity. However, the length of the reservoir is not infinite and from practical reasons length L is considered to be 5 to 10 times greater than the dam height H.

If it is considered that the velocities of motion of the fluid particles are low due to its innertness and if the pressure upon the free surface of the fluid to be generated due to the wave height is neglected, then the function of the hydrodynamic presseure has zero value at all the points upon the fluid surface, i.e., contour 5 (water fluid surface) has boundary condition of the type Γ_1.

$$W(x,y,z,t) = \overline{W}(x,y,z) = 0 \qquad x,y,z,\in \Gamma_1 \; . \qquad 1.8$$

2. SOLVING OF THE LAPLACE-S DIFFERENTIAL EQUATION

2.1. Transformation of the Laplace's Equation

The Laplace's equation contains derivations of the hydrodynamic pressure function of the second order. To apply the digital BE method, it is necessary to transform the derivations of the second order into the first order, i.e., to decrease the order of the Laplace's differential equation. Applying the weighted residual method upon the Laplace's differential equation and the boundary conditions, the following is obtained:

$$J = \int_{\Omega} \left(\frac{\partial^2 W}{\partial x^2} + \frac{\partial^2 W}{\partial y^2} + \frac{\partial^2 W}{\partial z^2} \right) \cdot p d\Omega + \int_{\Gamma 1} \left(\frac{\partial W}{\partial n} - \frac{\partial \overline{W}}{\partial n} \right) \cdot p d\Gamma_2 + \int_{\Gamma 1} \left(W - \overline{W} \right) \cdot \frac{\partial p}{\partial n} d\Gamma_1 = 0$$

2.1

where $p = p(x,y,z)$ is the weight function, while $\overline{W} = \overline{W}(x,y,z)$ and $\frac{\partial \overline{W}}{\partial n} = \frac{\partial \overline{W}(x,y,z)}{\partial n}$ are known, i.e., assigned values.

Integrating only the volume integral from expression 2.1 by partial integration, the following is obtained:

$$J = \int_x \int_y \int_z \left(\frac{\partial^2 p}{\partial x^2} + \frac{\partial^2 p}{\partial y^2} + \frac{\partial^2 p}{\partial z^2}\right) W dxdydz + \int_{\Gamma 1}\left(\frac{\partial W}{\partial n} p - \overline{W}\frac{\partial p}{\partial n}\right)\cdot d\Gamma_1 +$$

$$+ \int_{\Gamma 2}\left(\frac{\partial \overline{W}}{\partial n} p - W\frac{\partial p}{\partial n}\right)\cdot d\Gamma_2 = 0 \qquad 2.2$$

According to the weighted residual principle, the weight function $p = p(x,y,z)$ can be any function that is valid only in the $\Omega = \Omega(x,y,z)$ domain. This means that one can select a weight function that satisfies the Laplace's differential equation:

$$\frac{\partial^2 p}{\partial x^2} + \frac{\partial^2 p}{\partial y^2} + \frac{\partial^2 p}{\partial z^2} = 0 \qquad 2.3$$

Whereat, equation 2.2 obtains the following form:

$$\int_{\Gamma 1}\left(\frac{\partial W}{\partial n} p - \overline{W}\frac{\partial p}{\partial n}\right)\cdot d\Gamma_1 = \int_{\Gamma 2}\left(W\frac{\partial p}{\partial n} - \frac{\partial \overline{W}}{\partial n} p\right)\cdot d\Gamma_2 = 0 \qquad 2.4$$

Finally, after all the performed transformations equation 2.4 is obtained as a starting equation for application of the boundary element method.

Furthermore, while applying the boundary element method, another form of transformation of equation 2.4 has to be performed as well. That transformation finally leads to another form of equation 2.4 whereat a typical equation is obtained characteristic for any point M whose coordinates x,y,z belong to surface Γ_1 or surface Γ_2.

Such a transformation will be performed if weight function p=p(x,y,z) is selected such that it satisfies the Laplace's equation for all the points in domain Ω but not for the discrete point $M(x_m,y_m,z_m)$ at which the function has a unit value. Fig 2 shows the position of point M on contours Γ_1 and surface Γ_2.

If one adopts a weight function *pp* which represents a sum of the so called "fundamental" solution of the Laplace's equation and one Dirac-Delta fundction Δ^m, the following is obtained:

$$pp = \frac{\partial^2 p}{\partial x^2} + \frac{\partial^2 p}{\partial y^2} + \frac{\partial^2 p}{\partial z^2} + \Delta^m = 0 \qquad 2.5$$

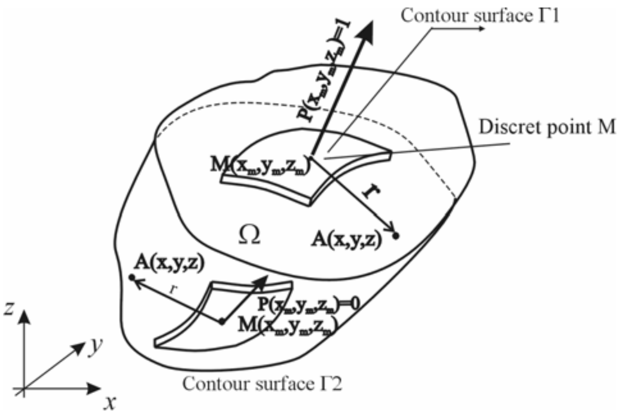

Fig 2. Position of point M on contours Γ1 and surface Γ2

Where Δ^m is the Dirac-Delta function that has a unit value at point M with coordinates x_m, y_m, z_m and a zero value at all the remaining points. Taking into account equation 2.5, the triple interval in equation 2.2 obtains the following form:

$$\int_x\int_y\int_z \Delta^m W dxdydz = -W_m \qquad 2.6$$

Where the value of W_m is the hydrodynamic pressure at the discrete point M. Now equation 2.4 obtains the following form:

$$W_m = \int_{\Gamma 1}\left(\frac{\partial W}{\partial n}p - \overline{W}\frac{\partial p}{\partial n}\right)d\Gamma_1 + \int_{\Gamma 2}\left(\frac{\partial \overline{W}}{\partial n}p - W\frac{\partial p}{\partial n}\right)d\Gamma_2 \qquad 2.7$$

After the performed transformations, equation 2.7 obtains the following form taking into account the singularity at point M.

$$\boxed{\frac{1}{2}W_m = \int_{\Gamma 1}(\frac{\partial W}{\partial n}p - \overline{W}\frac{\partial p}{\partial n})d\Gamma_1 + \int_{\Gamma 2}(\frac{\partial \overline{W}}{\partial n}p - W\frac{\partial p}{\partial n})d\Gamma_2} \qquad 2.8$$

Expression 2.8 represents a typical equation referring to one discrete point M located along the contours of Γ_1 or Γ_2 type. It is the basis for the application of the boundary element method in solving the Laplace's equation. The digital form of equation 2.8 is obtained if surfaces of the type of Γ_1 and Γ_2 are represented as ensembles of smaller surfaces, which will be referred to surfaces of the boundary elements:

$$\frac{1}{2}W_m = \sum_{e=1}^{NEL}\int_{\Gamma 1}(\frac{\partial W}{\partial n}p - \overline{W}\frac{\partial p}{\partial n})dBE + \sum_{i=1}^{NEL}\int_{\Gamma 2}(\frac{\partial \overline{W}}{\partial n}p - W\frac{\partial p}{\partial n})dBE \qquad 2.9$$

Analyzing eqaiton 2.9, a conclusion can be drawn that the integrals in the Γ1 and Γ2 areas are represented as sums of integrals over the surfaces of the boundary elements. Fig. 3 shows the discretization of Γ1 and Γ2 areas in the form of enseble of boundary elements. Each boundary element existing in the space is defined by the coordinates of its nodal points in respect to the global coordinate system. The boundary elements can be defined by 4 or 8 nodal points. The local numeration of the nodal points (1,2,3...8) should be given in a sequence starting with one node, for example node number one, Fig. 3, and going round the element clockwise or counter clockwise depending on the direction to which the vector of the normal at the centroid of the boundary element is to be directed. Fig. 3 shows the direction of the normal at the centroid of the element depending on the adopted directions of local numeration of the element. The nodes of the boundary elements should be marked in such a way that the normal to the elementary surface be always directed beyond domain Ω.

3. CONCEPT OF UNIT ACCELERATION FOR DEFINITION OF HYDRODYNAMIC EFFECT

The influence of hydrodynamic pressure could be included in the basic differential equation of dynamic motion, as follows:

$$M\ddot{U} + C\dot{U} + KU = M\ddot{a}(t) - HDF(t) \qquad 3.1$$

The hydrodynamic forces HDF(t) according to the concept of the effect of unit total acceleration at point M upon the remaining points on the dam surface is obtained from the following equation:

$$HDF(t) = H_p \ddot{a}_t \qquad 3.2$$

where Hp is the matrix evaluated in such a way that a unit acceleration is applied as boundary condition at point M on the dam surface in order to obtain the effect of point M upon the remaining points, whereat the boundary condition at those points is $\frac{\partial W_i}{\partial n} = 0$.

Vector $\ddot{a}_t$ in equation 3.2 is the vector of total accelerations at the discrete points on the dam face at time moment t. Fig. 3 shows the network of boundary elements on the dam face in contact with the fluid and an illustration of point M. The network of boundary elements has N points.

By application of the programme package BEL3 for the boundary conditions on the dam face illustrated in Fig. 3, a solution of pressure of the water fluid upon the discrete points of the network on the dam face is obtained, i.e., a row of matrix H referring to point M is obtained. Matrix H is obtained by iteration of the described process for all the nodes of the network of the dam face. If matrix H is multiplied by the vector of total accelerations along the normal at points i = 1, 2….N, one can obtain the vector of hydraulic pressure upon the dam body which is a variable category per time t. Since H is the matrix of effect of hydrodynamic pressure, and to obtain matrix Hp that defines the forces, the following well known expression is applied:

$$H=\begin{bmatrix} H_{11} & H_{12} & H_{13} & \bullet & \bullet & \bullet & H_{1N} \\ H_{21} & H_{22} & H_{23} & \bullet & \bullet & \bullet & H_{2N} \\ H_{31} & H_{32} & H_{33} & \bullet & \bullet & \bullet & H_{3N} \\ \bullet & \bullet & \bullet & \bullet & \bullet & \bullet & \bullet \\ H_{M1} & H_{M2} & H_{M3} & \bullet & \bullet & \bullet & H_{MN} \\ H_{M+1,1} & H_{M+1,2} & H_{M+1,3} & \bullet & \bullet & \bullet & H_{M+1N} \\ H_{1N} & H_{2N} & H_{3N} & \bullet & \bullet & \bullet & H_{NN} \end{bmatrix} \qquad 3.3$$

$$H_P=\int_A N^T H dA \qquad 3.4$$

Matrix Hp is evaluated irrespective of the process of solution of the equation of motion for the system 3.1 and it is stored in the computer memory. This means that matrix Hp is to be read only at the beginning of the process of solution of equation 3.1 and at each time moment t, it should be multiplied by the vector of total accelerations in the direction of the normal to the nodes of the dam face by which the hydrodynamic forces at those nodes at moment t will be obtained.

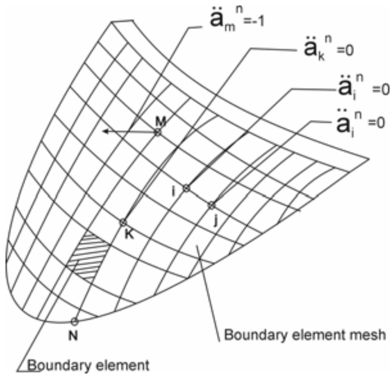

Fig. 3 Presentation of part of the model of boundary elements referring to the dam face with illustration of point M

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ALTERNATIVE METHODOLOGY FOR THE DETERMINATION OF THERMAL TRANSMITTANCE COEFFICIENT Ψ AND X

Summary:*During recent years a great deal of effort has been made to obtain a better insulation quality of the building envelope. In most countries insulation regulation came into force, which were accompanied by the information necessary to ensure correct thermal insulation.*

A special attention is given to thermal bridges, which can be taken as parts of the building envelope which are not correctly insulated. Here, the determination of the thermal transmittance coefficients ψ and χ represents the most difficult task regarding the calculus of thermal insulation. In the Romanian normative, inspired by the European regulation, for the determination of these quantities there are use derived formulas from those in definition of ψ and χ coefficients. These relations are accompanied by a set of rules and simple drawings for the main types of thermal bridge. This complicates the process of understanding the method and, on the other hand, the calculus is more difficult.

The alternative methodology proposed in this work uses directly the relations of definition for ψ and χ coefficients, more simple than those used in the normative and a simplified set of rules, which leads to a faster method understanding, and to a simple application.

Key words*: thermal insulation, thermal bridges, thermal transmittance coefficients ψ and χ,*

ALTERNATIVE METHODOLOGY FOR THE DETERMINATION OF THERMAL TRANSMITTANCE COEFFICIENT Ψ AND X

Rezima: *Poslednjih godina sve veća pažnja pridaje se podizanju izolacionih svojstava omotača zgrada. I većini zemalja donosi se tehnički propisi za izolaciju, sa pratećim informacijama potrebnim za obezbeđenje zadovoljavajuću toplotnu izolaciju.*

Posebna pažnja poklanja se termalnim mostovima koji su deo omotačai koji nisu dobri izolatori. Određivanje termičkog koeficijenta prenosa ψ i χ predstavlja najteži deo zadatka pri proračunu termičke izolacije. U regulativi Rumunije koja je pod uticajem Evropskih normi, za određivanje ove vrednosti koristi se izvedena formula iz one koja definiše vrednosti ψ i χ koeficijente. Ovim odnosima se pridružuje set pravila i primeri detalja crteža glavnih tipova termičkih mostova. Ovo komplikuje proces razumevenja metodologije i s druge strane otežava proračun.

Alternativna metodologija koja je predložena u radu koristi direktnu odnose i definicije koeficijenata ψ i χ, znatno je jednostavniji nego oni koji se koriste u tehničkim propisima i uprošćava set pravila koji dovode do bržeg razumevanja problema i lakše primene.

Key words*: termička izolacija, termički mostovi, koeficijenti prenosa toplote ψ i χ,*

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1. THE METHODOLOGY RECOMMENDED BY THE ROMANIAN NORMATIVE

For the calculus of the linear coefficient of the thermal transmittance ψ and for the punctual one χ, in the Romanian normative C 107/3 is used the relations:

$$\psi = \frac{\Phi}{\Delta T} - \frac{B}{R} \quad (1) \qquad \chi = \frac{\Phi}{\Delta T} - \frac{A}{R} \quad (2)$$

where: Φ – the thermal flux for a surface with the width B and the length of 1 m (W);
ΔT – the total difference of temperature (K or ºC);
B – the width considered for computation of the temperature field, according Fig. 1, 2, 3, and 4 (m);
R – the unidirectional thermal resistance (m^2K/W);
A – the area permeated by the thermal flux (m^2).

The „B" dimension from the relation (1) is considered for the interior surface of the elements. In Fig. 1, 2, 3 and 4 are represented a few usual situations in which occurs linear thermal bridges and the way of estimation the dimensions of the domain considered in the calculus.

The normative C 107/3 recommends to consider the widths of approximately 1,0 m (0,8...1,2 m) to the left and to the right of the bridge, supposed to cover the influence of any type of thermal bridge.

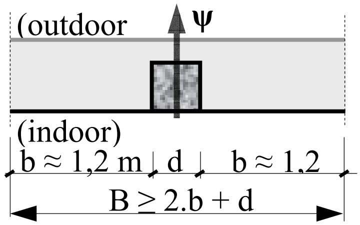

Fig. 1. *Thermal bridge in a built in column. The definition of the term „B" from relation (1)*

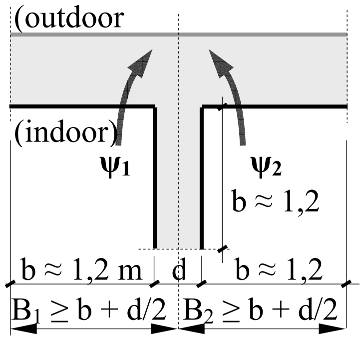

Fig. 2. *Thermal bridge at the intersection of an exterior wall with an interior one. The definition of term „B" from the relation (1)*

2. THE SIGNIFICANCE OF Ψ AND X COEFFICIENTS

For the linear ψ and the punctual χ coefficients of thermal transmittaSnce, there are no definitions in the C 107/3 Normative, which only specifies, quite vaguely, that they „bring a correction to the unidirectional calculus, considering the presence of the thermal bridges...".

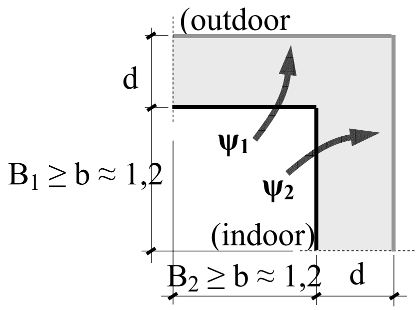

Fig. 3. *Thermal bridge from the intersection of two exterior walls - outer corner. The definition of term „B" from relation (1)*

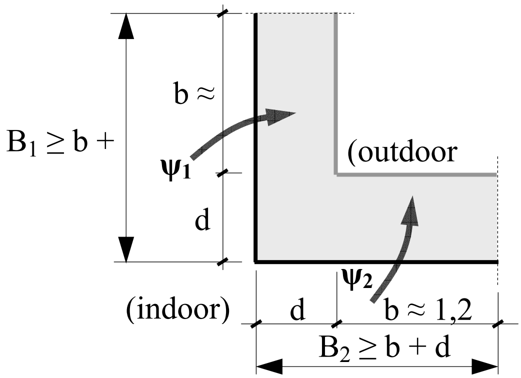

Fig. 4. *Thermal bridge situated at the intersection of two exterior walls - inner corner. the definition of point „B" from relation (1)*

For the disclosure of the physical significance of these coefficients, it can begin from the relation of definition for the corrected coefficient of thermal transmittance U':

$$U' = \frac{q'}{\Delta T} \tag{3}$$

where: q' – the thermal flux density (the unitary thermal flux) (W/m^2);
ΔT – the total dumping of the temperature (the difference between the temperature of the indoor air and the temperature of the external air)

For an area of a building element that contains only one linear bridge (Fig. 5), the density of the thermal flux can be expressed as the sum between the density q_u in an unidirectional field (without considering the thermal bridge, as if it would be missing) and a supplementary flux density Δq caused by the bridge: $q' = q_u + \Delta q$.

Relation (3) becomes:

$$U' = \frac{q_u + \Delta q}{\Delta T} \tag{4}$$

Expression (4) can be written as it follows:

$$U' = \frac{q_u + \Delta q}{\Delta T} = \frac{q_u}{\Delta T} + \frac{\Delta q}{\Delta T} = U + \frac{\frac{\Delta\Phi}{A}}{\Delta T} = \frac{1}{R} + \frac{\Delta\Phi}{A.\Delta T} = \frac{1}{R} + \frac{\Delta\Phi}{\lambda.\Delta T} \cdot \frac{\lambda}{A} \tag{5}$$

where: U – coefficient of linear thermal transmittance (W/m^{2}K);
R – linear thermal resistance (m^{2}K/W);
ΔΦ – additional flux due to the thermal bridge (W);
A – the area crossed by the thermal flux; according to Fig. 5: $A = B.\ell$ (m^2).

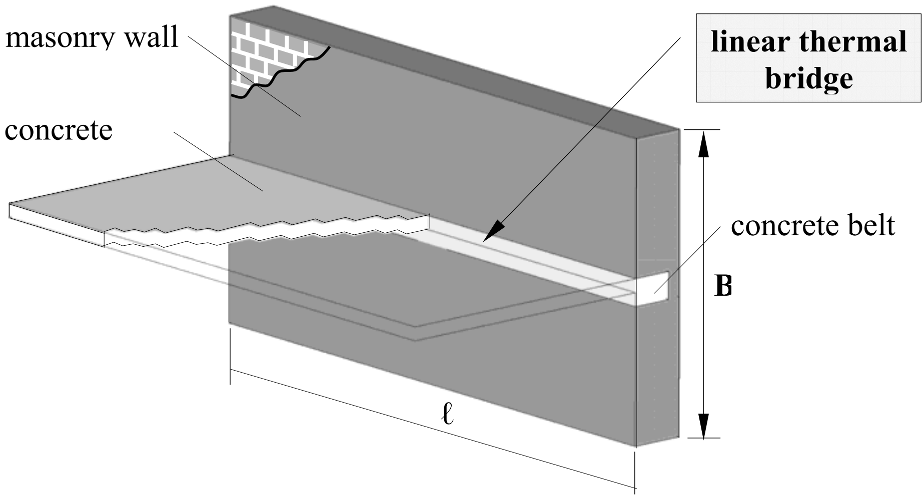

Fig. 5. *Element with a single linear thermal bridge*

With the notation:

$$\frac{\Delta\Phi}{\lambda.\Delta T} = \psi \tag{6}$$

relation (5) can be written as:

$$U' = \frac{1}{R} + \frac{\psi.\lambda}{A} \tag{7}$$

Expression (7) represents the well known relation for the estimation of the corrected thermal transmittance coefficient, singularized for the situation where there is only one linear thermal bridge.

Expression (6) constitutes the relation of definition for the linear coefficient of thermal transmittance ψ. By means of this expression the coefficient ψ can be defined as the supplementary flux $\Delta\Phi$ due to a linear bridge, reported to the length ℓ of the bridge and to the total dumping of temperature ΔT or, in other words, the linear coefficient of thermal transmittance represents the supplementary thermal flux which crosses a bridge with the length of 1 m, for a temperature drop of 1 ºC or 1 K.

In the same manner, it can be demonstrate that the relation of definition for the punctual thermal transmittance coefficient χ is:

$$\chi = \frac{\Delta\Phi}{\Delta T} \tag{8}$$

The coefficient χ represents the supplementary thermal flux which crosses a punctual thermal bridge, concordant to a temperature difference of 1 ºC or 1 K.

3. ALTERNATIVE METHODOLOGY FOR THE ESTIMATION OF THE COEFFICIENTS Ψ OR X

As it was demonstrated above, the relations of definition for the linear, ψ, and punctual coefficients, χ, coefficients of thermal transmittance are:

$$\psi = \frac{\Delta\Phi}{\lambda . \Delta T} \qquad (9) \qquad\qquad \chi = \frac{\Delta\Phi}{\Delta T} \qquad (10)$$

where: $\Delta\Phi$ – supplementary flux due to the thermal bridge: $\Delta\Phi = \Phi' - \Phi_u$ (W);
Φ' – the thermal flux that crosses the domain (the part of the element that includes the thermal bridge) (W);
Φ_u – unidirectional thermal flux, that crosses the same domain, but in the absence of the thermal bridge (W);
ℓ – the length of the linear thermal bridge (m);
ΔT – the total dumping of temperature (the difference between the temperature of the indoor air and the temperature of the external air) (K or ^{o}C).

The relations (1) and (2), used in the Romanian regulations, have been deducted from the expressions (9) and (10) respectively, but there is no any obstruction for the direct use in the practical calculus of the relations of definition.

Therefore, the effective calculus of the coefficients ψ and χ can be done using the expressions (9) and (10), following the stages exposed below:

a. the construction (on the computer) of the numerical model of the thermal field, for the plane field defined by the transversal section through the linear bridge (usually a vertical or a horizontal section) in the case of ψ coefficient, or for the spatial domain in the case of χ coefficient and the determination of the thermal flux Φ' which crossbeams the element;

b. the determination of the linear thermal flux Φ_u for the domain defined at paragraph a, in the absence of the thermal bridge (the calculus can be done manually);

c. the estimation of the difference between the two fluxes $\Phi' - \Phi_u = \Delta\Phi$ and the adjustment of data to the length of the bridge and to the temperature dumping (for the ψ coefficient), or just to the temperature dumping (for the χ coefficient).

The problem is how extended must be the considered domain. Practically, in case of the linear thermal bridges must be considered sections on the one side and the other of the bridge, extended enough so it can exceed the limits of the influence area of the bridge, limits which varies mainly consequently to the bridge structure. According to normative C 107/3 and other regulations, an approximate width of 1,2 m of the two adjacent areas could be considered enough to cover the case of every type of bridge.

For the situations presented in Fig. 1, 2, 3 and 4, the modelled domains (for the assessment of the flux Φ') are taken according to Fig. 6.a, 7.a, 8.a and 9.a, and the correlated domains with the eliminated bridges (for the assessment of flux Φ_u) according to Fig. 6.b, 7.b, 8.c and 9.c.

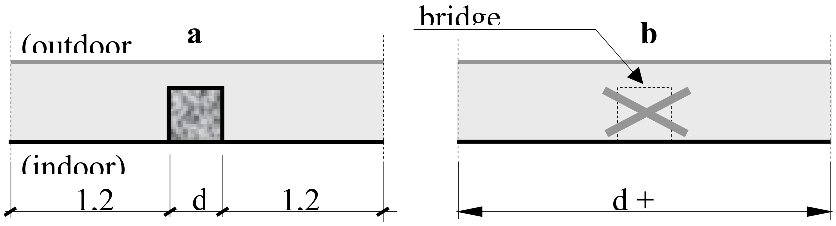

Fig. 6. *Thermal bridge near a built in column (horizontal section)*
a. 2D domain numerical modelled; b. domain without thermal bridge (unidirectional calculus)

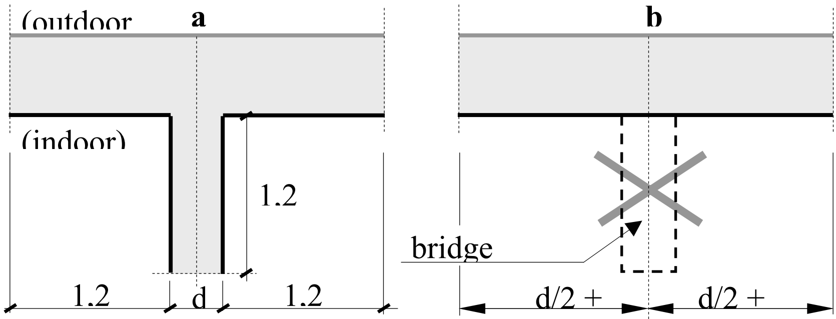

Fig. 7. *Thermal bridge situated at the intersection of one external wall with a internal one*
a. 2D domain numerical modelled; b. domain without thermal bridge (unidirectional calculus)

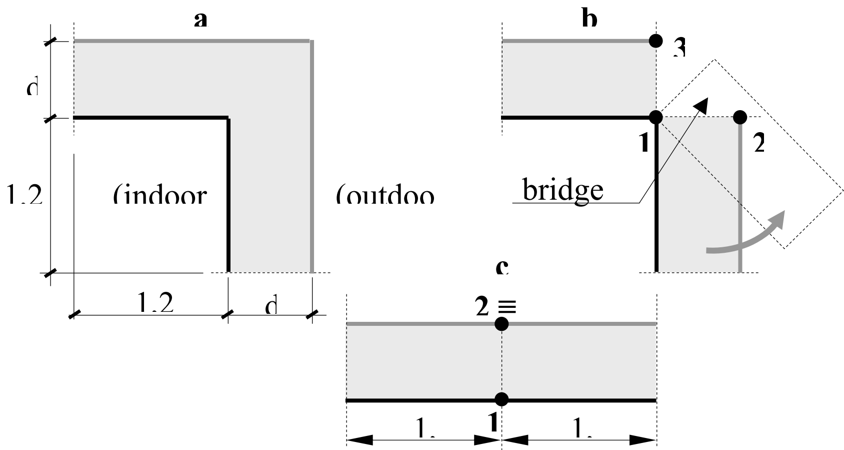

Fig. 8. *Thermal bridge situated at the intersection of two exterior walls – outer corner*
a. 2D domain numerical modelled; b.the way of thermal bridge „removal”;
c. domain without bridge (unidirectional calculus)

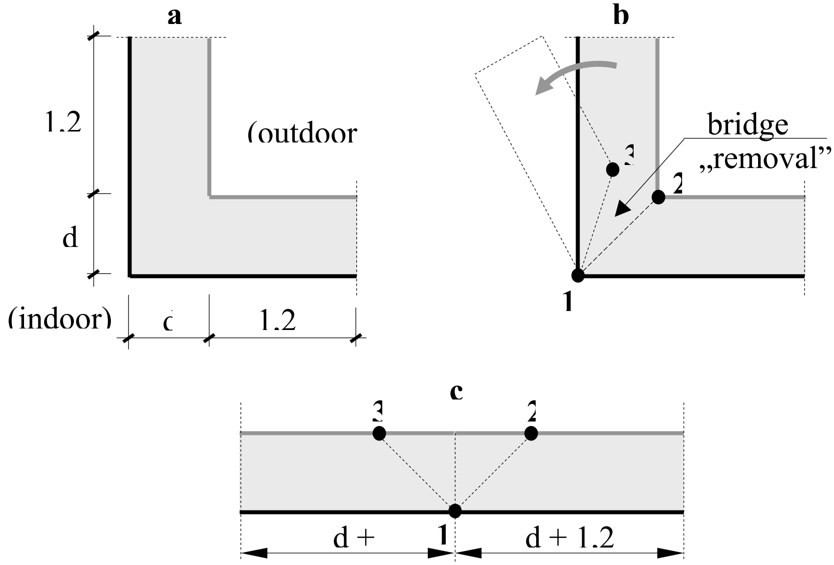

Fig. 9. *Thermal bridge situated at the intersection of two exterior walls – inner corner*
a. 2D domain numerical modelled; b. the way of thermal bridge „removal”;
c. domain without bridge (unidirectional calculus)

4. CONCLUSIONS

The equations (1) and (2) on one side, and (9) and (10) on the other side, represents two ways to determine the ψ and χ coefficients of thermal transmittance. Both ways imply the same volume of computation, but the second, based on the relations of definition, has the following advantages:

- it uses simpler expressions for the transmittance coefficients ψ and χ;
- emphasize the physical significance of the coefficients ψ and χ, leading to a more transparent, coherent and easy to understand computation manner; relations (1) and (2) wrap up the logic of the method, especially because in the romanian Normative C 107/3 there are no definitions for these coefficients;
- avoids the handle of the term „B” in relation (1) by applying the rules of the thermal bridges removal, showed in Fig. 6, 7, 8 and 9.

Regarding the thermal flux Φ’ that pass through each thermal bridge, this must be calculated either by the determination of the thermal field in the plane domain defined by the transversal section through the linear thermal bridge, or by the determination of the thermal field on the spatial domain corresponding to the punctual bridge. In both situations it is necessary to use a computation program capable to solve thermal field problems, usually programs based on the finite elements method.

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TENTH NATIONAL AND
FOURTH INTERNATIONAL
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APPLICATION OF THE RISK THEORY TO MANAGEMENT RELIABILITY OF THE PIPELINE

Summary: *The leakless distant transportation of large amounts of hazardous matters (for example natural gas) through a pressurized pipelines is a serious challenge. The pipeline should be able to withstand the full range of ambient conditions as well as the Eath's crust movements, varying soil chemical state at present and after months or years of exploitation. During the previous years the formal risk assesments were carried out on the base of the failure intensity. Since that time the procedure of the risk assesment has become more sophisticated. At present risk management is being more and more mandated by regulations.*
Key words: *risk management, pipeline, reliability.*

PRIMENA TEORIJE RIZIKA ZA UPRAVLJANJE POUZDANOŠĆU CEVOVODA

Summary: *Neprocurivanje tokom dugog transporta velikih količina opasnih materija (na primer prirodnog gasa) kroz cevovode pod pritiskom je ozbiljan izazov. Cevovod treba da izdrži sve ambijentalne uslove, kao što su pomeranje zemljine kore i različite hemijske uticaje iz tla, sada i nakon meseci i godina eksploatacije. Tokom prethodnih godina formalna procena rizika je donošena na osnovu intenziteta oštećenja. Od tada procedura procene rizika je postala mnogo sofisticiranija. Sada se upravljanje rizikom sve više uređuje propisima.*
Key words: *upravljanje rizikom, cevovod, pouzdanost.*

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APPLICATION OF THE RISK THEORY TO MANAGEMENT RELIABILITY OF THE PIPELINE

The leakless distant transportation of large amounts of hazardous matters (for example natural gas) through a pressurized pipelines is a serious challenge. The pipeline should be able to withstand the full range of ambient conditions as well as the Eath’s crust movements, varying soil chemical state at present and after months or years of exploitation. During the previous years the formal risk assesments were carried out on the base of the failure intensity. Since that time the procedure of the risk assesment has become more sophisticated. At present risk management is being more and more mandated by regulations.

In general risk is defined as the probability of an event causing damage and the potential amount of the sustained loss.

Basic concepts Hazard

Underlying the definition of risk is the concept of hazard. We typically define a hazard as a characteristic or group of characteristics that provides the potential for a loss. Flammability and toxicity are examples of such characteristics.

It is important to make the distinction between a *hazard* and a *risk* because we can change the risk without changing a hazard. When a person crosses a busy street, the hazard should be clear to that person. Several methodologies are available to identify hazards and threats in a formal and structured way. A hazard and operability (HAZOP) study is a technique in which a team of system experts is guided through a formal process in which imaginative scenarios are developed using specific guide words and analyzed by the team. Event-tree and fault-tree analyses are other tools. Such techniques underlie the identified threats to pipeline integrity that are presented in this book. Identified threats can be generally grouped into two categories: time-dependent failure mechanisms and random failure mechanisms, as discussed later. *Risk* is most commonly defined as the probability of an event that causes a loss and the potential magnitude of that loss. By this definition, risk is increased when either the probability of the event increases or when the magnitude of the potential loss (the consequences of the event) increases. Transportation of products by pipeline is a risk because there is some probability of the pipeline failing, releasing its contents, and causing damage (in addition to the potential loss of the product itself).

The most commonly accepted definition of risk is often expressed as a mathematical relationship:

Risk=(event likelihood) x (event consequence)

As such, a risk is often expressed in measurable quantities such as the expected frequency of fatalities, injuries, or economic loss.

A complete understanding of the risk requires that three questions be answered:

1. What can go wrong?
2. How likely is it?
3. What are the consequences?

By answering these questions, the risk is defined.

Failure

Answering the question of "what can go wrong?" begins with defining a pipeline failure. The *unintentional release of pipeline contents* is one definition. *Loss of integrity* is another way to characterize pipeline failure.

Failure occurs when the structure is subjected to stresses beyond its capabilities, resulting in its structural integrity being compromised. Internal pressure, soil overburden, extreme temperatures, external forces, and fatigue are examples of stresses that must be resisted by pipelines. Failure or loss of strength leading to failure can also occur through loss of material by corrosion or from mechanical damage such as scratches and gouges.

The answers to what can go wrong must be comprehensive in order for a risk assessment to be complete. Every possible failure mode and initiating cause must be identified. Every threat to the pipeline, even the more remotely possible ones, must be identified. Theory detail possible pipeline failure mechanisms grouped into the four categories of *Third Party, Corrosion, Design,* and *Incorrect Operations.* These roughly correspond to the dominant failure modes that have been historically observed in pipelines.

Probability

By the commonly accepted definition of risk, it is apparent that probability is a critical aspect of all risk assessments. Some estimate of the probability of failure will be required in order to assess risks. This addresses the second question of the risk definition: "How likely is it?"

That is, "real" probability estimates arise only from statistical analyses—relying solely on measured data or observed occurrences. In reality, there are no systems beyond very simple, fixed-outcome-type systems that can be fully understood solely on the basis of past observations—the core of statistics. Almost any system of a complexity beyond a simple roll of a die, spin of a roulette wheel, or draw from a deck of cards will not be static enough or allow enough trials for statistical analysis to completely characterize its behavior. Statistics requires data samples—past observations from which inferences can be drawn. As systems become more complex, more variable in nature, and where trial observations are less available, the frequency approach will often provide answers that are highly inappropriate estimates of probability.

Underlying most of the complete definitions of probability is the concept *of degree of belief.* Ideally, the degree of belief could be determined in some consistent fashion so that any two estimators would arrive at the same conclusion given the same evidence. A statistic is not a probability. Statistics are only numbers or methods of analyzing numbers.

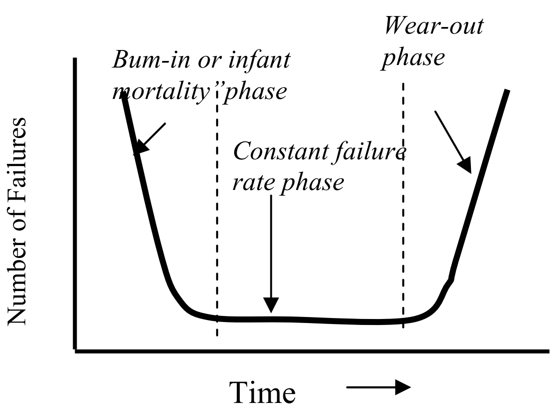

Figure 1. Common failure rate curve.

Some pieces of equipment or installations have a high initial rate of failure. This first portion of the curve is called the *burn-inphase* or *infant mortality phase/* As these defects are eliminated, the curve levels off into the second zone. This is the so-called *constant failure zone* and reflects the phase where random accidents maintain a fairly constant failure rate. Components that survive the burn-in phase tend to fail at a constant rate. Failure mechanisms that are more random in nature—third-party damages or most land movements for example—tend to drive the failure rate in this part of the curve. Far into the life of the component, the failure rate may begin to increase. This is the zone where things begin to wear out as they reach the end of their useful service life. Where a time-dependent failure mechanism (corrosion or fatigue) is involved, its effects will be observed in this *wear-out phase* of the curve. An examination of the failure data of a particular system may suggest such a curve and theoretically tell the evaluator what stage the system is in and what can be expected (1).

BEGINNING RISK MANAGEMENT

Step 1: Acquire a risk assessment model

A pipeline risk assessment model is a set of algorithms or "rules" that use available information and data relationships to measure levels of risk along a pipeline. A risk assessment model can be selected from some commercially available models, customized from existing models, or created "from scratch" depending on requirements.

Step 2: Collect and prepare data

Data preparation are the processes that result in data sets that are ready to be read into and used by the risk assessment model.

Step 3: Devise and implement a segmentation strategy

Because risks are rarely constant along a pipeline, it is advantageous to first segment the line into sections with constant risk characteristics (dynamic segmentation) or otherwise divide the pipeline into manageable pieces.

Step 4: Assess the risks

After a risk model has been selected and the data have been prepared, risks along the pipeline route can be assessed. This is the process of applying the algorithm—the rules—to the collected data. Each pipeline segment will get a unique_risk score that reflects its current condition, environment,~and the operating maintenance activities. These relative risk numbers can later be converted into absolute risk numbers. Risk assessment will need to be repeated periodically to capture changing conditions. -

Step 5: Manage the risks

This step consists of determining what actions are appropriate given the risk assessment results.

Model design and data collection are often the most costly parts of the process. These steps can be time consuming not only in the hands-on aspects, but also in obtaining the necessary consensus from all key players. The initial consensus often makes the difference between a widely accepted and a partially resisted system. Time and resources spent in these steps can be viewed as initial investments in a successful risk management tool. Program management and maintenance are normally small relative to initial setup costs.

Risk assessment models.

Three general types of models, from simplest to most complex, are matrix, probabilistic, and indexing models. Each has strengths and weaknesses, as discussed below. Although each risk assessment method discussed has its own strengths and weaknesses, the indexing approach is especially appealing for several reasons:

• Provides immediate answers
• Is a low-cost analysis (an intuitive approach using available information)
• Is comprehensive (allows for incomplete knowledge and is easily modified as new information becomes available)
• Acts as a decision support tool for resource allocation modeling
• Identifies and places values on risk mitigation opportunities

Third-Party Damage Index

A.	Minimum Depth of Cover	20%
B.	Activity Level	20%
C.	Aboveground Facilities	10%
D.	Line Locating	15%
E.	Public Education Programs	15%
F.	Right-of-Way Condition	5%
G.	Patrol Frequency	15%
		100%

This table lists some possible variables and weightings that could be used to assess the potential for third-party damages to a typical transmission pipeline.

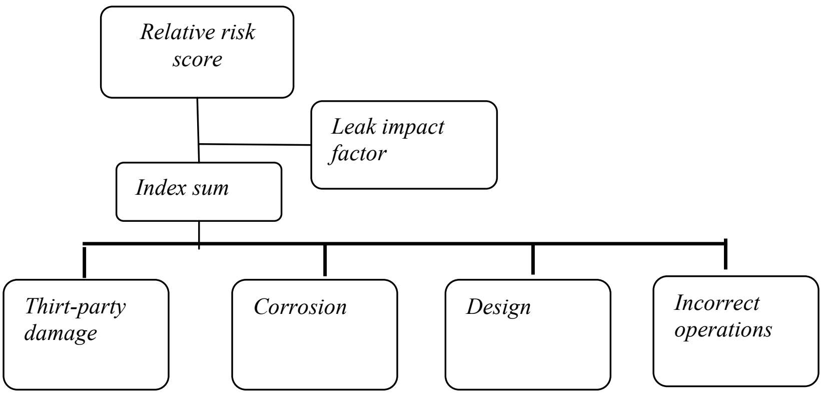

Figure 2. Basic risk assessment model.

Of course, the ideal risk model will be accurate, but accuracy may only be verified after many years. Reproducibility is another characteristic that is sought and immediately verifiable. If multiple assessors examine the same situation, they should come to similar conclusions if our model is acceptable.

The scientific method requires both inductive reasoning and deductive reasoning. Induction or inference is the process of drawing a conclusion about an object or event that has yet to be observed or occur on the basis of previous observations of similar objects or events. In both everyday reasoning and scientific reasoning regarding matters of fact, induction plays a central role. In an inductive inference, for example, we draw conclusions about an entire group of things, or a population, on the basis of data about a sample of that group or population; or we predict the occurrence of a future event on the basis of observations of similar past events; or we attribute a property to a non observed thing on the grounds that all observed things of their burial 2.5 to 3 feet deep.

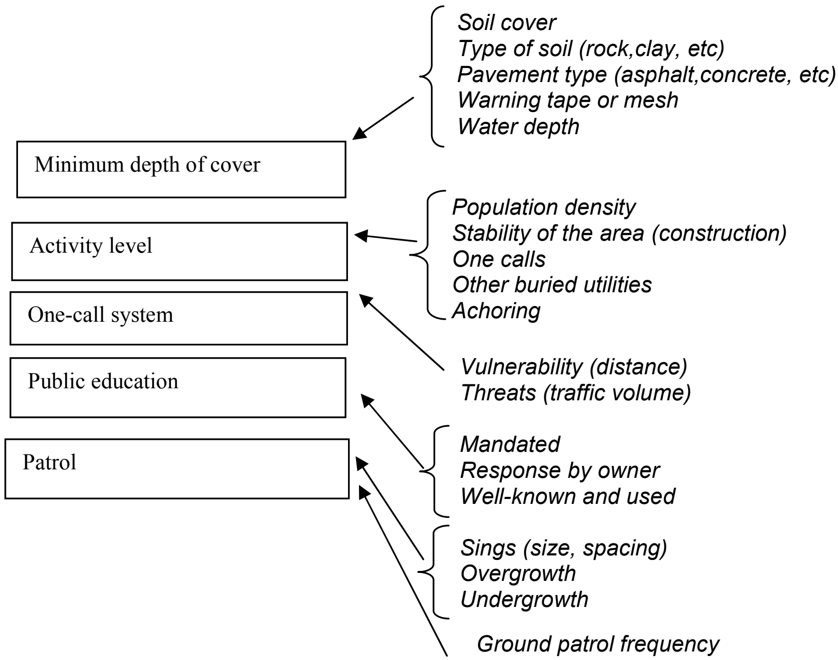

Figure 3. Assessing third-party damage potential: sample of data used to score the third-party index.

However, encroachments of population and land development activities are routinely threatening many pipelines today.

Risk variables

The pipeline designer and, perhaps to an even greater extent, the operator can affect the probability of damage from third-party activities. As an element of the total risk picture, the probability of accidental third-party damage to a facility depends on

- The ease with which the facility can be reached by a third party
- The frequency and type of third-party activities nearby.

Possible offenders include
Excavating equipment
Projectiles
Vehicular traffic
Trains
Farming equipment
Seismic charges
Fence posts

Factors that affect the susceptibility of the facility include
Depth of cover
Nature of cover (earth, rock, concrete, paving, etc.)
Man-made barriers (fences, barricades, levees, ditches, etc.)
Natural barriers (trees, rivers, ditches, rocks, etc.)
Presence of pipeline markers
Condition of right of way (ROW)
Frequency and thoroughness of patrolling
Response time to reported threats.
The activity level is often judged by items such as:
Population density
Construction activities nearby
Proximity and volume of rail or vehicular traffic
Offshore anchoring areas
Volume of one-call system reports
Number of other buried utilities in the area.

Serious damage to a pipeline is not limited to actual punctures of the line. A mere scratch on a coated steel pipeline damages the corrosion-resistant coating. Such damage can lead to accelerated corrosion and ultimately a corrosion failure perhaps years in the future. If the scratch is deep enough to have removed enough metal, a stress concentration area could be formed, which again, perhaps years later, may lead to a failure from fatigue, either alone or in combination with some form of corrosion-accelerated cracking.

Several variables are thought to play a critical role in the threat of third-party damages. Measuring these variables can therefore provide an assessment of the overall threat. Note that in the approach described here, this index measures the potential for third-party damage—not the potential for pipeline failure from third-party damages. This is a subtle but important distinction. If the evaluator wishes to measure the latter in a single assessment, additional variables such as pipe strength, operating stress level, and characteristics of the potential third-party intrusions (such as equipment type and strength) would need to be added to the assessment.

Failure scenarios *I*

There are an infinite number of possible failure scenarios encompassing all possible combinations of failure parameters. For evaluation purposes, nine different scenarios are examined involving permutations of three failure (hole) sizes and three possible pressures at the time of failure. These are used to represent the complete range of possibilities so that all probabilities sum to 100%. Probabilities of each hole size and pressure are assigned, as are probabilities for ignition in each case. For each of the nine cases, four possible damage ranges (resulting from thermal effects) are calculated. Parameters used in the nine failure scenarios are shown in Table 14.38.

The system is about 100 km long and consists of a 1420 mm buried trunk line, a station for pressure regulation, gas metering points, and transfer connection to other pipelines and several fittings and valves.

This pipeline is used to transport dry gas at a nearly constant pressure. Table 1 summarizes the basic data of the pipeline.

Table 1

Pipeline system data	
Description	**Value**
Outer diameter	1420 mm
Wall thickness	18.4 mm
Material	10G2FB
Area	Onshore, low population density
Cover depth	>0.9 m
Length of system	900 m

The input dates for an evaluation of a condition of the pipeline are determined in pipeline inspection reports; observations during maintenance; the analysis of a cartographical material.

Probabilistic analysis

For calculation the expectancy of function of a reserve of strength $\overline{S}$ and its dispersion the formula (1) were used.

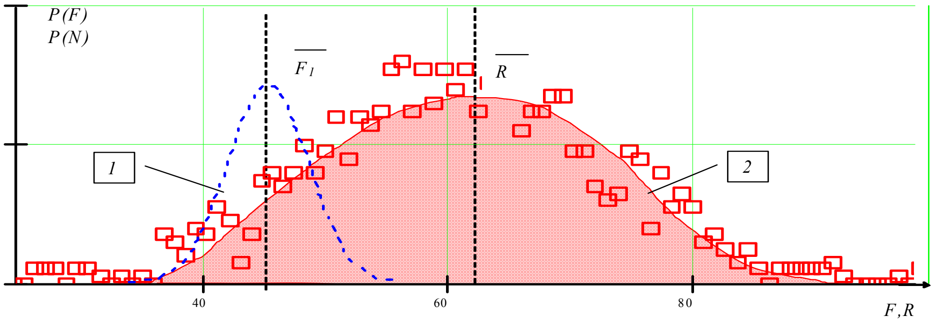

Figure.4 Graphs of statistical distribution of longitudinal force N_y - 1 (in MPa) and bearing capacity (2 - longitudinal critical force for first of the form of loss a stability in MPa) gas transmission line segment.

The author offers the engineering approach to an evaluation of a reliability of complexes systems (on an example, gas transmission line segment), permitting sharply to reduce number of experiments at statistical modeling (simulation) (up to 2^{n-1} , where n - number of taken into account parameters of a condition) (1,2).

The basic task of probability calculation is the evaluation of characteristic (reserve) of strength $\widetilde{S}$. If to accept, that the probability of realization of an inequality $\widetilde{S} = \widetilde{R} - \widetilde{F} > 0$, is probability indestructibility of a construction, the probability of destruction - spikes (exceeding of the boundary of area in allowable state) is determined by formula:

$$P_f = \int_{-\infty}^{0} P_s(S)dS \qquad (1)$$

where $P_S(S)$ - function of density (the strength reserve).

At any laws of distribution $\widetilde{F}$ and $\widetilde{R}$: $\quad \overline{S} = \overline{R} - \overline{F}; \quad \sigma_S = \sqrt{\sigma_R^2 + \sigma_F^2}$.

This method was designed to identify safety margins under present conditions and offers an easy way to find admissible changes in operating limits given the predefined system performance of special importance are the benefits achievable under capacity control variations attributable, in part, to potential operating flaw’s.

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TENTH NATIONAL AND
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DEVELOPMENT OF STEEL SPATIAL STRUCTURES IN BULGARIA AND ITS APPLICATION IN HIGH EARTHQUAKE ACTIVITY REGIONS

Summary: *This paper presents the application of steel spatial construction system, development in Bulgaria for buildings with double-layer grids roofs structures. Attention is drawn to the possibilities for using the system in regions with high earthquake activity. The results of seismic analysis of several buildings in Bulgaria, which are interesting with their architecture are presented and analyzed.*

Key words: *spatial structures, double- layer grids, earthquake .*

RAZVOJ PROSTORNIH ČELIČNIH KONSTRUKCIJA U BUGARSKOJ I NJIHOVA PRIMENA U OBLASTIMA SA VISOKOM SEIZMIČKOM AKTIVNOŠĆU

Rezime: *U radu je prikazana primena prostornog konstrukcijskog čeličnog sistema, razvijenog u Bugarskoj za objekte sa krovnim konstrukcijama od dvoslojne mreže. Posebno su istaknute mogućnosti primene sistema u oblastima sa visokom seizmičkom aktivnošću. Prikazani su i analizirani rezultati seizmičke analize nekoliko objekata u Bugarskoj, koji su interesantni zbog svoje arhitekture.*

Ključne reči: *prostorne konstrukcije, dvoslojna mreža, zemljotres.*

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1. INTRODUCTION

The requirements for greater expressiveness of the modern forms of architecture based on frequently repeated members brought forth framework structures of new type called spatial structures.

They are with considerable advantages, which make possible to raise the economic efficiency of construction work as compared to the results with the traditional methods obtained so far.

In this sense the advantages involve:

Spatial behaviour of the structures, which is the reason for the increased reliability in respect of unexpected collapse of single joint members or bars;

Less overall height of the roof construction;

Spanning of greater support distances;

Planning and design of: overhead transport, suspended ceilings;

Screens with various decorative effects;

Lighting units and other fixtures as arrange in interior would contribute to the artistic set ups and apprehension of the roof construction and of the space as a whole;

Flexible solutions in case of complicated layouts and vertical compositions;

Prefabrication of the steel structures;

Lower transport expenditures and easier transportation to place difficult for asses;

Assembling and erection of the construction on ground level and lifting up large size block elements, etc.

In geometrical composition the spatial structures are formed of individual unitised modules, which are called structural crystals, fitted in space so as their peaks would from two even surfaces- upper and lower- in such a way that the system nodes formed by the upper surfaces are in positions as to the lower surfaces arranged specific modules (fig.1).

In general the structural crystal are 3D members, which can be with different geometrical forms- inclined prisms with triangular, square, rectangular or another cross-section; wedge on rectangular or square base, ect.

The choice of the form of the structural crystal depends upon the type and size of the load as well as the nodal connections.

The first spatial structures appeared by the end of the forties of the XX century. Within the next 20 years a great number of space structures were introduced all over the world in USA, Germany, Czech Republic (Lederer F., 1960.), Spain (Martinez A., 1989), Japan, Macedonia and other countries.

Of particular importance among all spatial structures is the structures developed at the Moscow Architectural Institute in Russia and KNIPIAT “Glavproject”- “SLS’ 85-MArchI”. It is manufactured at factory “Montex” at the town of Montana, Bulgaria since 1975.

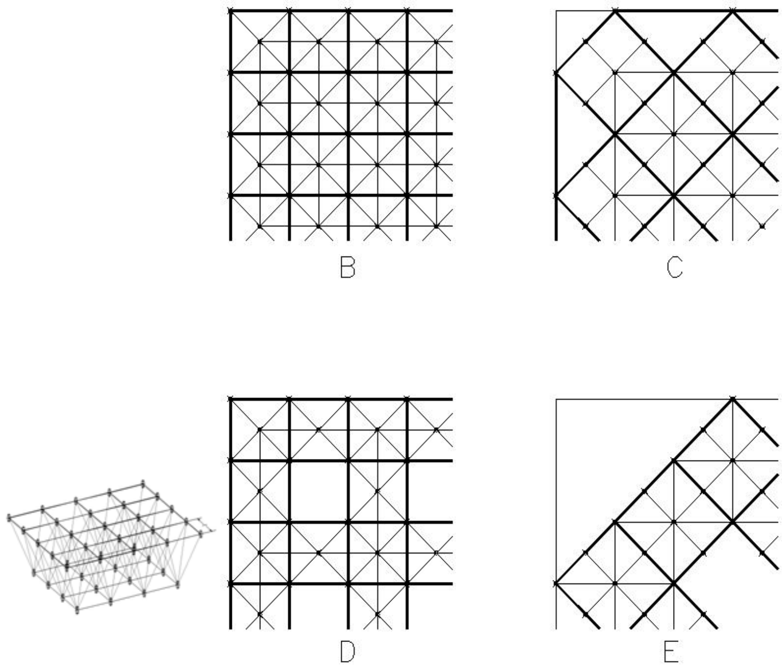


Figure 1 Double-Layer Grids Systems

2. DEVELOPMENT OF THE CONSTRUCTION SYSTEMS

Basically the geometrical composition of the spatial frame type MArchI consists repeatedly used pyramids with square or rectangular contours, which made up of rods and nodal members (fig.1).

The rods are made of steel pipes of different diameters. At the ends of the rods are welded inserts fitted with axial bolts and moving hexagon special sleeves. The rods are with coordinating lengths of 1.5 m, 2.0 m, 3.0 m, 4.5 m, (fig.2).

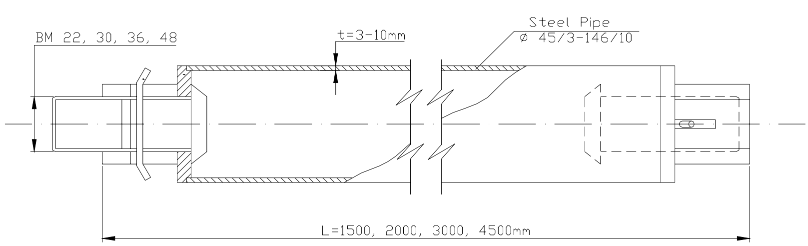


Figure. 2. Main structural element

The nodal members represent polyhedrons inscribed in spherical surfaces. On the flat side of the polyhedrons are drilled holes, which are threaded on the inside. Parts of the nodal members are used to connect horizontal rods at an angle of 90° and diagonal rods at 45° (fig.3). Predominantly they are used for buildings with spatial structures composed pyramids with square basis.

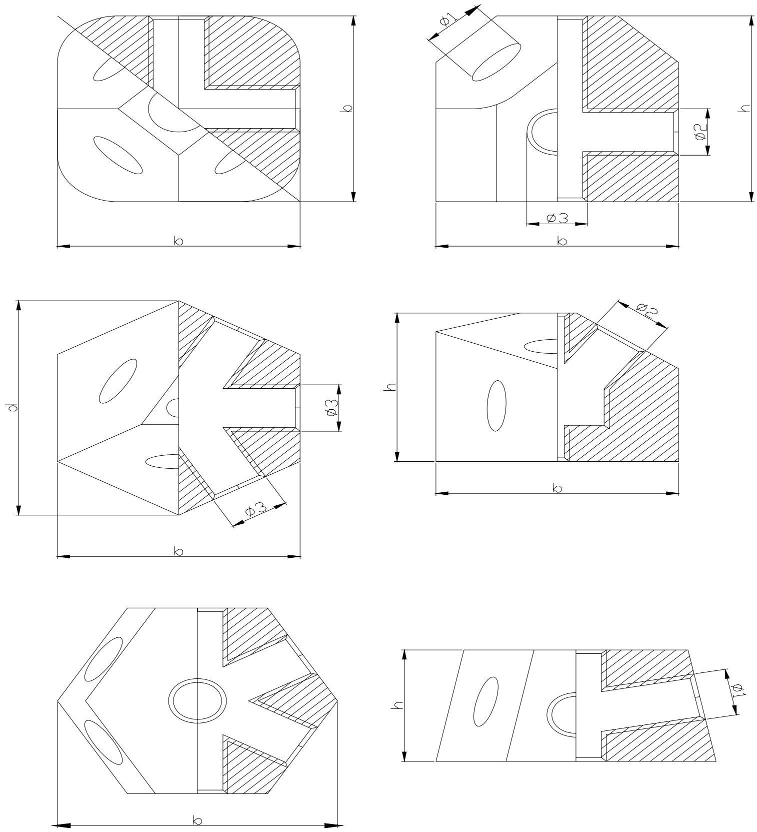


Figure 3. Nodal members

When rods are connected by means of nodal members with holes drilled at 90°, then upper and lower rectangular and even lattices are formed with the size of the orthogonal cells 1.5×1.5 m; 2.0×2.0 m; 3.0×3.0 m or 4.5×4.5 m depending on the coordinating rod lengths. The rods of the upper lattices are shifted in respect of the lower ones by half a cell module. In the space nodes of the upper and lower lattices are connected by diagonal rods, which are arranged at angles of 45° in relation to the horizon. Their coordinating lengths are equal to the lengths of the chord rods. By connecting the members in the space (rods and nodal members) it is ensured that the horizontal and vertical loads acting upon the roof construction are transferred to the column heads.

The rods and nodal members are connected with the aid of high strength bolts, which are built at the tubular members. The bolts are driven into the nodal members by the rotary- translation movement when rotating the special fixed to the bolts with retention pins. Driving of the special sleeves continue until their lower surfaces would get in touch with the retention surfaces of the nodal members.

The tightness of the connections is ensured through the initial tensions formed in the bolts while the tensioning forces depend upon the pin bearing capacities.

For the construction system there are nodal members in which holes are drilled for chord rods at 60° and for diagonal rods at 54°. With these elements it is possible to form pyramids with rectangular bases and then construct space frames either triangular or hexagonal in planning. Also, nodal members are developed which are making possible to construct cylindrical space frames (fig.3).

In terms of geometrical composition building system “MArchI” represents a “non-limited” system (term, used in typification of structures determining universally and serviceability of building systems), since it enable design of various in configuration and dimensions (in plan and elevation) different spatial skeletons with considerable large borders of architectural performance.

From the point of view of flexibility, the system satisfies in practice arbitrary kind of buildings or structures accounting an appreciable range of design loadings and a seismic impact. An important advantage of that system is its manufacture in every plant or workshop equipped according to standard. Likewise a basic prerequisite for its application is the possibility of permanent perfection, reaching high technical and economical indices. In comparison to traditional solutions structures manufactured after the “MArchI” system consume 20-25% less of metals. The reduction of labour of 20% is due to the adopted flow-line production and assembling of large size blocks. In practice it shortens building terms up to three times.

It is obvious from the foresaid that the efficiency of spatial structures at the present day of their development is not confined by the only criterion for example materials or labour expenditure but a range of criteria as reliability, durability, universality, unification possibility, architectural expressiveness etc. Structures after “MArchI” response in the highest degree to that combination of criteria therefore they are in great prospects in our building practice, i.e. sports centers (gymnasium, athletics halls, swimming pools, covered skating rings etc.), for trade objects (supermarkets, covered bazaars), for multifunctional buildings (youth clubs, educational buildings), exhibition halls, industrial and agricultural buildings.

The unified used by now assortment of rod and nodal members envelops the most advantageous economical solutions of the great majority available so far. Cross-sections of rods have been selected according to the definite gradation of stresses, where the min. dimension satisfies the ultimate value of the corresponding compressed rod, the max. dimension corresponds to the max. stress undertaken by the heat-treated bolt.

System columns are worked out in two versions:

Pipes, providing an elevation from 4.8 m to 8.4 m to the bottom chord;

Rolled rectangular or square steel angles, providing an elevation of 10.8 m to the bottom edge of the spatial structure.

At the lower end columns are equipped with supporting traverses and there are special supporting heads at the top, where the spatial construction treads. Steel columns are fixed to the foundations by means of anchor bolts.

The elements included in the building system permit erection of the main construction of one-storey blocks with dimensions in plan from 12×12 m to 90×120 m without intermediate supports.

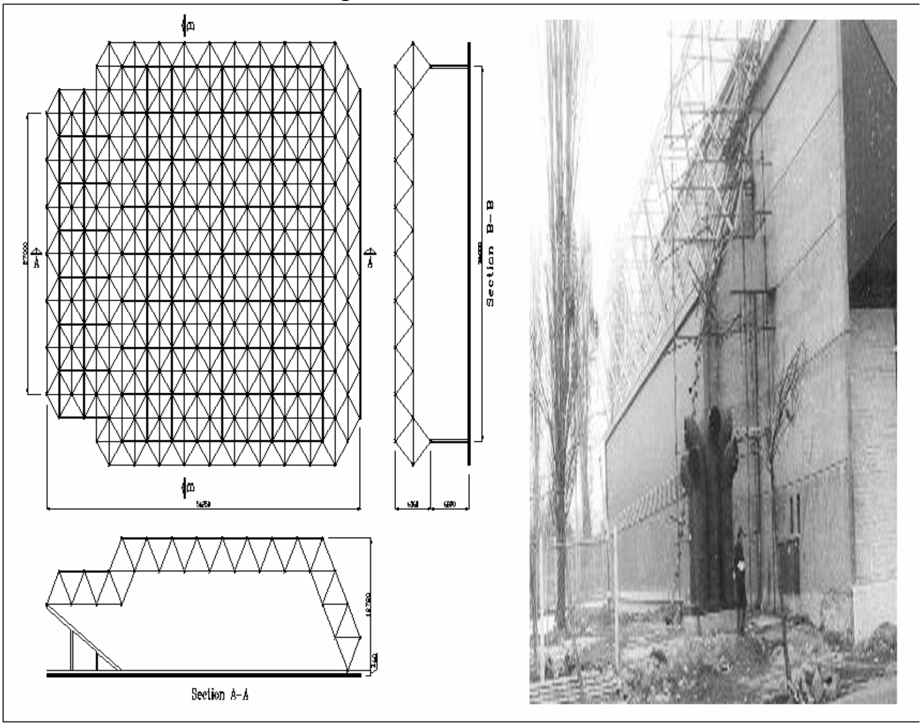
Maximum vertical loading, for which spatial structures are studied and performed with the help of the system elements is 3.70 kN/m2 and minimum loading- 2.0 kN/m2 (including snow load- 0.7 kN/m2, suspended transportation load- 32 kN, seismic load- level 9, according to MSK.

With building structures manufactured in our country after the system "MArchI" various projects of one million square meters and more have been built of very good economic and social effect.

3. APPLICATION OF DOUBLE-LAYER CONSTRUCTION SYSTEM "MARCHI" FOR BUILDINGS

3.1. Tennis playground hall in Plovdiv

The basic dimensions of the hall is 36×54 m, with clearance height of 10 m, measured from level 0.00 to the lower belt of the structure (fig.4). Two-layer framework is used composed of cells with dimensions 4.5×4.5 m and 3.18 m in height. In longitudinal direction the spatial structure is supported by 16 tubular columns of ∅ 402×9 mm. Transversely the spatial structure is supported along the left edge by the steel stand and in opposite direction continues as a sloped space wall. In order to maintain maximum load capacity in sense of tensile strength at bolt M30, which is 370 kN, the space frame is prestressed at the respective locations. The total amount of steel used for the roof structure is 38.7 tons. The building is constructed in 1983.



3.2. Training hall for ice-hockey in Sofia

The training hall is designed for ice-hockey but can be used for other sports as well (basket ball, volley ball, hand ball, tennis etc. The spatial structures with dimensions 51×30 m and 2.121 m height handed up on ten inclined columns, which are situated on the contours of the building (fig.5). The inclination of the ten columns is really effective by reason of lessening the bulk and the span of the building.

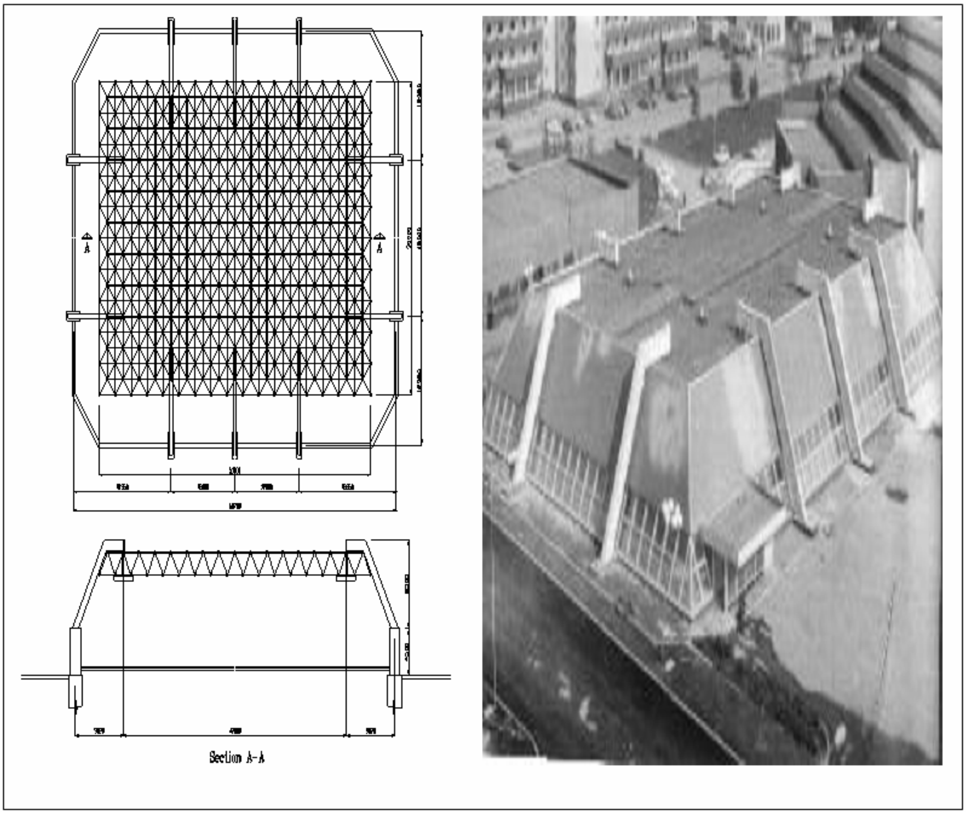


Figure 5. Ice-hockey Hall

TENTH NATIONAL AND
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NUMERICAL ANALYSIS OF COMPOSITE STEEL-CONCRETE BEAMS, REGARDING RHEOLOGY

Summary: *The paper presents analysis of the stress changes due to creep in statically determinate composite steel-concrete beam.. The mathematical model involves the equation of equilibrium, compatibility and constitutive relationship, i.e. an elastic law for the steel part and an integral-type creep law of Boltzmann – Volterra for the concrete part. For determining the redistribution of stresses in beam section between concrete plate and steel beam with respect to time "t", Volterra integral equations of the second kind have been derived, on the basis of the theory of the viscoelastic body of Arutyunian–Trost-Bazant. Numerical method, which makes use of quadrature formulae for solving these equations, is proposed. Example with the model proposed is investigated. The creep functions is suggested by the "CEB-FIP" models code 1970. The elastic modulus of concrete $E_c(t)$ is assumed to be independ of time 't'. Our results are compared with the corresponding results of Effective modulus method (EMM), Rate of creep method (RCM), Improved Dischinger method (IDM) and Trost method (TM) and European Code EC4) data.*

Key words *: composite steel-concrete beam, Volterra integral equations, rheology.*

NUMERIČKA ANALIZA KOMPOZITNIH GREDA ČELIK-BETON U POGLEDU REOLOGIJE

Rezime: *U radu je prikazana analiza promene napona usled tečenja u statički određenim kompozitnim gredama čelik-beton. Matematički model je uključio jednačinu ravnoteže, kompatibilnost i konstitutivne veze, odnosno zakon elastičnosti za čelični deo i integrakni zakon tečenja Bolcman-Volter za betonski deo. Za određivanje preraspodele napona u preseku grede između betonske ploče i čelične grede uzimajući u obzir vreme "t", Volterova jednačina druge vrste je izvedena, na osnovu teorije viskoelastičnog tela Arutyunian–Trost-Bazant. Predložen je numerički metod, koji omogućava upotrebu kvadratne formule za rešavanje ovih jednačina. Istražen je primer sa predloženim modelom. Funkcije tečenja su predložene u "CEB-FIP" model kodu iz 1970. Pretpostavljeno je da je modul elastičnosti betona $E_c(t)$ nezavistan od vremena 't'. Naši rezultati su upoređeni sa odgovarajućim rezultatima metode efektivnog modula (EMM),metode brzine tečenja (RCM), naprednog Dischinger-ovog metoda (IDM) i Trost-ovog metoda (TM) i podataka iz Evrokoda EC4..*

Ključne reči: *kompozitne grede čelik-beton, Volterova integralna jednačina, reologija*

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1. INTRODUCTION

The problem of investigating the statically determinate composite plate beam in the time t has for 60 years drawn the attention of engineers who were dealing with the problems of their design. This problem has, however, received a certain currency in the past few years, due to the new facts gathered about the rheological qualities of concrete. It is known that while in the steel beam, under the effect of the service ability loads, we see only elastic deformations, in the concrete plate during the time significant plastic deformation takes place as a consequence of creep and shrinkage of concrete. As a result of these deformations and because of the stiff connection between the two elements of the composite plate beam, in every cross-section subjected to the effect of constantly operating outside bending moment M_0 in the time t the arises a new additional group of forces and moments $N_{c,r}(t)$, $M_{c,r}(t)$, $N_{s,r}(t)$, $M_{s,r}(t)$ (fig.1.). The influence of this group of forces and moments over the general stress conditions of the statically determinate composite plate beam is expressed by the decrease of the stresses in the concrete plate and in the increase of stresses in the steel beam. The papers dealing with the solution of the problem of finding the unknown normal forces $N_{c,r}(t)$, $N_{s,r}(t)$ and the bending moments $M_{c,r}(t)$, $M_{s,r}(t)$ are numerous and diverse. Their chronological analysis shows from the first aspect the aspirations of various authors to penetrate further into the actual behavior of the structures, which will eventually lead to the creation of more accurate calculation methods; and from the second aspect it shows the aspirations to replace the complicated methods by simple ones for practical usage.

The first works, which give the answer to this problem are based on the Law of Dischinger[3,4], who had first formulated a time-dependent stress-strain differential relationship for concrete.

These books and papers connected with the names of Frohlich[6], Esslinger [6], Kloppel [6], Sonntag [6], Kunert [6], Muller [6], Dimitrov [6] and Mrazik [6], represent one independent group for which it is characteristic that for the unknown quantities $N_{c,r}(t)$ and $M_{c,r}(t)$ a system of simultaneous differential equations have been derived and solved. All these methods have been collected and analyzed by Sattler [6] and by the first author of this paper [6]. In parallel with the developed numerical methods, Blaszkowiak [6], Fritz [6] and Wippel [6] have developed approximate methods, which use Dischinger's idea for applying in the calculation the ideal (fictitious) modulus of elasticity [3,4]. Another method of the estimate design calculation as described in [6] has been based on the creep fibre method by Busemann [6]. With Wippel's methods [6] the first stage of the development of the analytical methods is based entirely on the works of Dischinger [3,4], has been completed.

Further development of rheology as a fundamental science and its application to concrete as well as a great number of investigations in the field of creep of concrete have led to new formulations of the time-dependent behavior of concrete [6]. This new formulation that gives the relationship between $\sigma_c(t)$ and $\varepsilon_c(t)$ is an integral equation which gives the basis of the theory of linear viscoelastic bodies. However, in order to avoid the deduction of the integral equations of Volterra for treating the problem connected with the creep of concrete structures, Trost [7] and Zerna [8] have revised the integral relationship into new algebraic stress-strain relationship. Departing from the same considerations another revision of integral relationship into new algebraic stress-strain relationship have been made by Kruger [6] and Wolff [6]. On the basis of the new laws methods have been developed connected with the names Wolff [6], Trost [6] and Heim [6], for solving the problem raised by Frohlich [6]. In parallel with the methods developed on the basis of the theory of linear viscoelastic bodies, Sattler [6], Haenzel [6], and Profanter [6] have recently developed new methods, which are based on the 'modified theory' of Dischinger, called also the theory of Rusch-Jungwirt [5]. It turned out, however, that the theory of Rusch-Jungwirt [5] has been subjected to serious criticism in the works of Alexandrovski-Arutyunyan [1] and Bazant [2].

2. THEORY

The theory implies the following assumptions to be true:

a) Bernoulli's concerning plane strain of cross-sections.

b) Concrete is not cracked $\sigma_c \leq (0.4 \div 0.5) R_c$.

c) Hooke's law applies to steel as well as to concrete under short-time loads.

d) In the range of service ability loads concrete behaves in a way allowing to be treated as a linear viscoelastic body. The stress-strain behaviour of concrete can be described with sufficient accuracy by the integral equations (1) by Trost [7]

$$\varepsilon_c(t) = \frac{\sigma_c(t_0)}{E_c(t_0)}\left[1 + \varphi(t - t_0)\right] + \int_{t_0}^{t} \frac{d\sigma_c(\tau)}{d\tau} \frac{1}{E_c(\tau)}\left[1 + \varphi(t - \tau)\right] d\tau, \tag{1}$$

where $\varphi(t-\tau) = \varphi_N K(\tau) f(t-\tau)$ is the so called the creep function and φ_N the ultimate value of creep coefficient $K(\tau)$ depends on the age increase of concrete. It is called the function of aging, and it characterizes the process of the aging. The increase of τ makes $K(\tau)$ monotonously decrease. The function $f(t-\tau)$ - (where *t* is the time interval during which the structure is under observation, τ is the running coordinate of time) – characterizes the process of creeping.

e) The modulus of concrete elasticity is invariant in time *t* i.e.

$$E_c(\tau) = E_c(t_0) = E_c \tag{2}$$

f) According to a proposal by Sonntag [6], the influence of the development of the bending moment $M_{c,r}(t)$ in the concrete member, upon the redistribution of the normal force of concrete $N_{c,r}(t)$ can be neglected.

Let us denote both the normal forces and the bending moments in the cross-section of the plate and the girder after the loading in the time $t = 0$ with $N_{c,0}$, $M_{c,0}$, $N_{s,0}$, $M_{s,0}$ and with $N_{c,r}(t)$, $M_{c,r}(t)$, $N_{s,r}(t)$, $M_{s,r}(t)$ a new group of normal forces and bending moments, arising due to creep and shrinkage of concrete.

For a composite bridge girder with $J_c = \dfrac{A_c(nI_c)n}{A_s I_s} \leq 0.2$ according to the suggestion of Sonntag [6] we can write the equilibrium conditions in time *t* as follows

$$N(t) = 0; \quad N_{c,r}(t) = N_{s,r}(t); \tag{3}$$

$$\sum M(t) = 0; \quad M_{c,r}(t) + N_{c,r}(t) r = M_{s,r}(t), \tag{4}$$

Due to the fact that the problem is a twice internally statically indeterminate system, the equilibrium equations (3), (4) are not sufficient to solve it.

It is necessary to produce two additional equations in the sense of compatibility of deformations of both steel girder and concrete slab in time *t*.

These conditions are as follows :

Strain compatibility on the contact surfaces between the concrete and steel members of composite girder

$$\varepsilon_{sh}(t_0) f(t - t_0) + \frac{N_{c,0}}{E_c(t_0)A_c}\left[1 + \varphi(t - t_0)\right] - \frac{1}{E_c(t_0)A_c}\int_{t_0}^{t} \frac{dN_{c,r}(\tau)}{d\tau}\left[1 + \varphi(t - \tau)\right] d\tau +$$
$$\frac{N_{s,0}}{E_s A_s} - \frac{1}{E_s A_s}\int_{t_0}^{t} \frac{dN_{s,r}(\tau)}{d\tau} = \frac{M_{s,0}}{E_s I_s} r + r\frac{1}{E_s I_s}\int_{t_0}^{t} \frac{dM_{s,r}(\tau)}{d\tau} d\tau \tag{5}$$

Compatibility of Curvatures when $\tau = t$

$$\frac{M_{c,0}}{E_c(t_0)I_c}\left[1 + \varphi(t - t_0)\right] - \frac{1}{E_c(t_0)I_c}\int_{t_0}^{t} \frac{dM_{c,r}(\tau)}{d\tau}\left[1 + \varphi(t - \tau)\right] d\tau =$$
$$\frac{M_{s,0}}{E_s I_s} + \frac{1}{E_s I_s}\int_{t_0}^{t} \frac{dM_{s,r}(\tau)}{d\tau} d\tau \tag{6}$$

After integrating the two equations by parts and using the (3) and (4) for assessment of normal forces $N_{c,r}(t)$ and bending moment $M_{c,r}(t)$ two linear integral Volterra equations of the second kind are derived.

$$N_{c,r}(t) = \lambda_N \int_{t_0}^{t} N_{c,r}(\tau)\frac{d}{d\tau}\left[1 + \varphi_N K(\tau) f(t - \tau)\right] d\tau + \lambda_N N_{c,0}\varphi_N K(t_0) f(t - t_0) + \lambda_N N_{sh} f(t - t_0)$$

, (7)

$$M_{c,r}(t) = \lambda_M \int_{t_0}^{t} M_{c,r}(\tau)\frac{d}{d\tau}\left[1 + \varphi_N K(\tau) f(t - \tau)\right] d\tau + \lambda_M M_{c,0}\varphi_N K(t_0) f(t - t_0) - \lambda_M \frac{E_c I_c}{E_s I_s} N_{c,r}(t) r$$

(8) in which

$$\lambda_N = \left[1 + \frac{E_c A_c}{E_s A_s}\left(1 + \frac{A_s r^2}{I_s}\right)\right]^{-1}, \quad \lambda_M = \left[1 + \frac{E_c I_c}{E_s I_s}\right]^{-1} \tag{9)-(10}$$

$$\varepsilon_{sh}(t_\infty) = 20.10^{-5},\ N_{sh} = \varepsilon_{sh}(t_\infty) E_c A_c \tag{11}$$

In each of these equations the functions

$$N_{c,0}\varphi_N K(t_0) f(t-\tau),\ M_{c,0}\varphi_N K(t_0) f(t-\tau),\ \frac{d}{d\tau}\left[1+\varphi_N K(\tau) f(t-\tau)\right]$$

are given.

3. NUMERICAL METHOD

The integral equations are solved by a numerical method using quadratic formulas. These methods represent a replacement of the integral equations by approximate linear equations with triangle matrix related in view of a discrete value of the unknown function. The replacement is achieved on the basis of approximation of the integral equation operator by quadratic formulas. The increase of the parameter τ is related to the growth of the descretizating points, so that the application of certain quadratic formulas of Simpson, Gauss, Markoff, Chebishev is rather troublesome. That is why integrals are approximated with quadratic formulas of trapeze.

4. NUMERICAL RESULTS

A demonstration of the numerical method is implemented in MATLAB code. Several practical examples in bridge construction have been solved. One of the examples had its parameters cross-section values and initial section forces are shown in fig. 1.

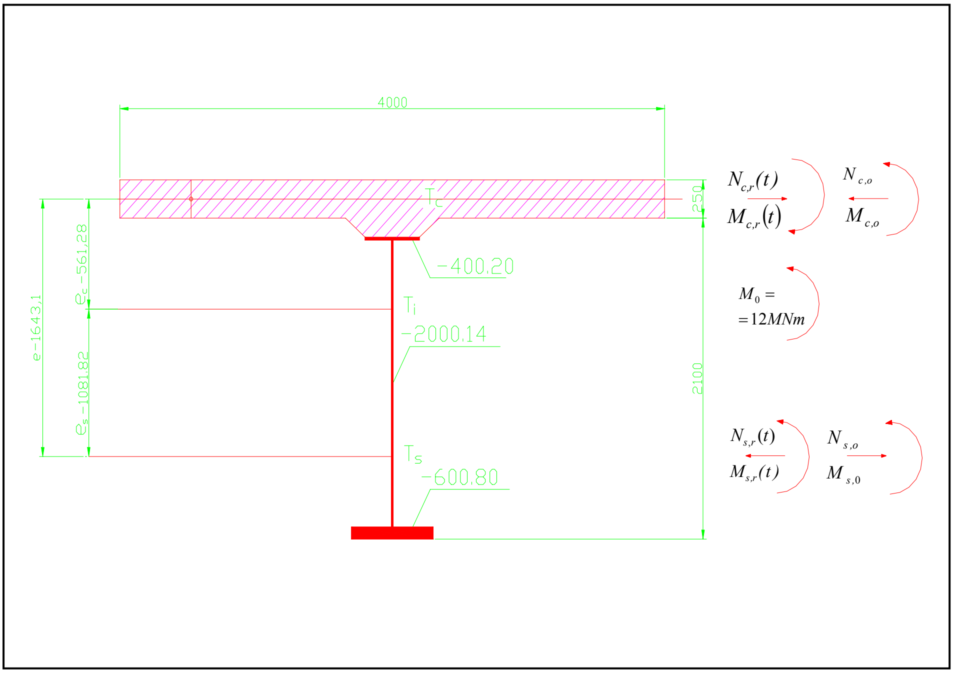

Figure 1. Composite beam with cross-section characteristic

$$E_c = 3{,}4.10^4 \ MPa,\ E_s = 2{,}1.10^5 \ MPa,\ A_c = 10000cm^2,\ A_s = 840cm^2, n = \frac{E_s}{E_c} = 6{,}176$$

$$I_c = 520833cm^4,\ I_s = 4859650cm^4,\ r_c = 56{,}128cm,\ r_s = 108{,}182cm,\ r = 164{,}31cm,$$
$$A_i = 2453{,}05cm^2,\ I_i = 19875408\,cm^4.$$

On the basis of numerous solved examples the optimal step of three days for solving the integral equations (7) and (8) is found. Creep coefficient $\varphi_N = 2.5$ and the functions $K(\tau)$ and $f(t-\tau)$ are taken according to the recommendations of CEB-FIP(1966-1970). For both functions $K(\tau)$ and $f(t-\tau)$ the approximations of Wolff [6] are chosen:

$$K(\tau) = \begin{cases} \dfrac{10.28}{5+\sqrt{\tau}} & for \quad \tau \le 857 \\ 0.3 & for \quad \tau > 857 \end{cases} \tag{12}$$

and

$$f(t-\tau) = 1 - e^{\left[-0.6\left(\frac{t-\tau}{30}+0.0025\right)^{0.4} - 0.091\right]} \tag{13}$$

The variations of the inner normal forces $N_{c,r}$, $N_{s,r}$ and $M_{c,r}$, $M_{s,r}$ in the example for solving the creep problem are found for $t_0 = 28,\ 90,\ 180, 365$ and 730 days till respectively 6025, 6087, 6177, 6362 and 6727 days are shown on figure 2,3 and 4.

The variations of the normal stresses in time t_∞ in concrete plate σ_c^{up} and in steel girder σ_s^{up}, σ_s^{down} in the example for solving the creep problem are found for $t_0 = 28,\ 90,\ 180, 365$ and 730 days till respectively 6025, 6087, 6177, 6362 and 6727 days are shown on figure 5,6,7.

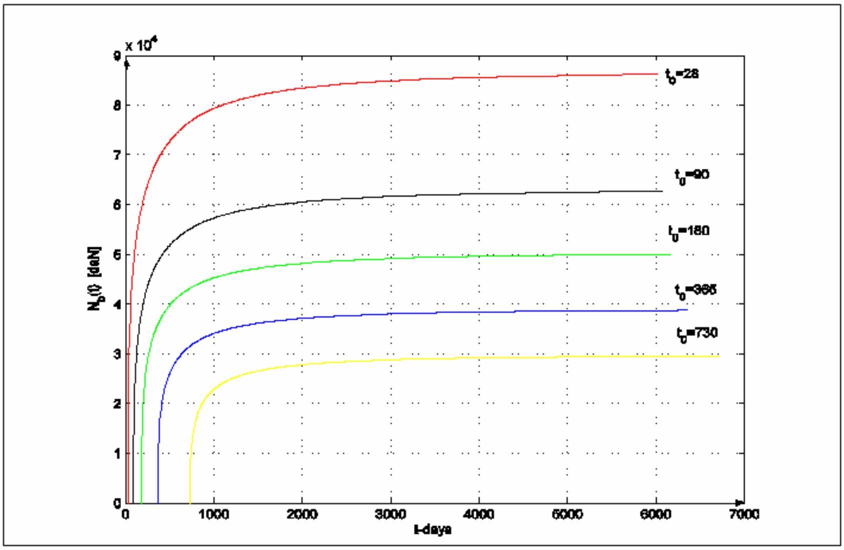

Fig. 2. Values of normal forces $N_{c,r}(t) = N_{s,r}(t)$ *in time* t *when loading is applied in time* t_0*=28,90,180,365 and 730 days*

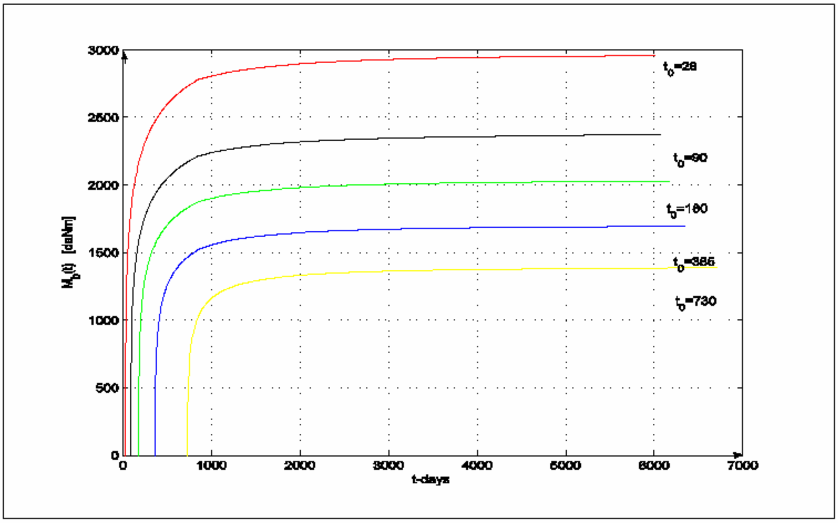

Fig. 3. Values of bending moments $M_{c,r}(t)$ in time t when loading is applied in time t_0=28,90,180,365 and 730

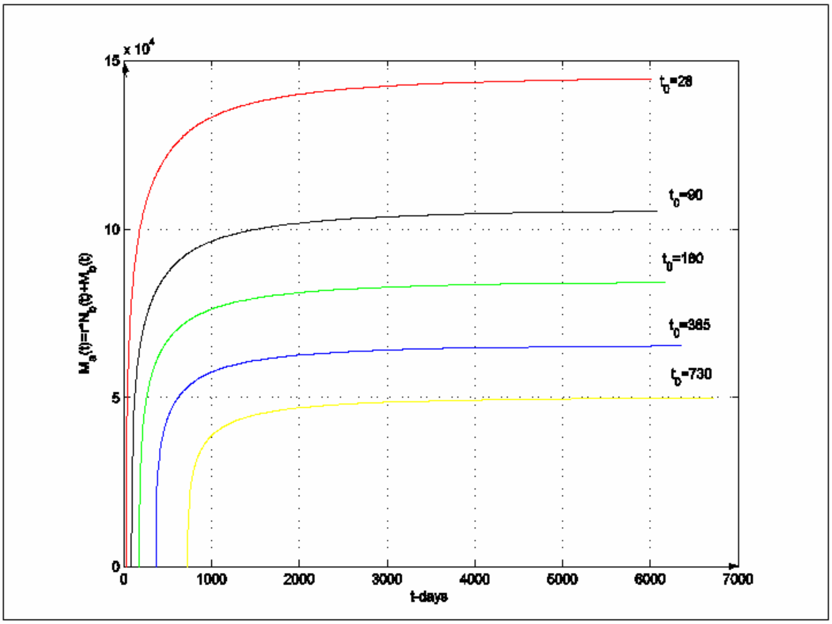

Fig 4. Values of bending moments $M_{s,r}(t) = r * N_{c,r}(t) + M_{c,r}(t)$ in time t *when loading is applied in time* t_0*=28,90,180,365 and 730 days.*

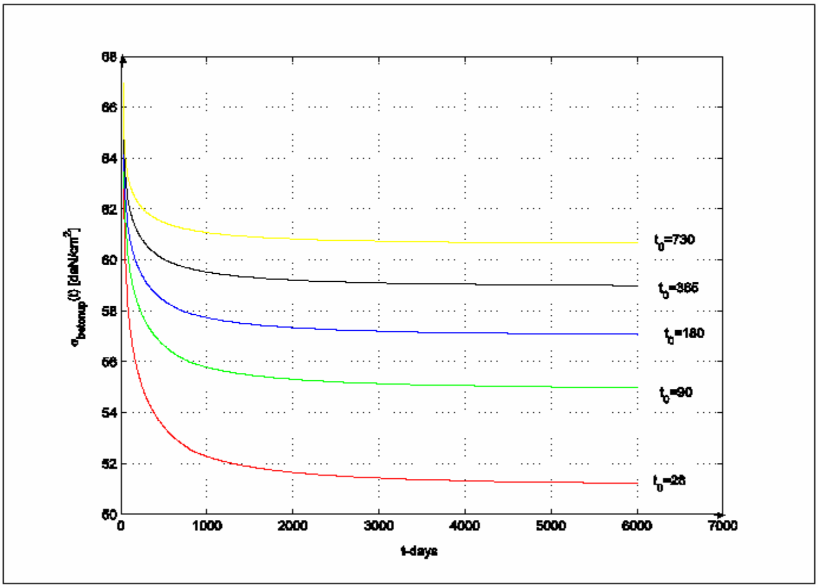

Fig. 5 Values of normal stresses in upper fiber of concrete plate $\sigma_c^{up}(t)$ *in time* t_∞ *when loading is applied in time* t_0*=28,90,180,365 and 730 days*

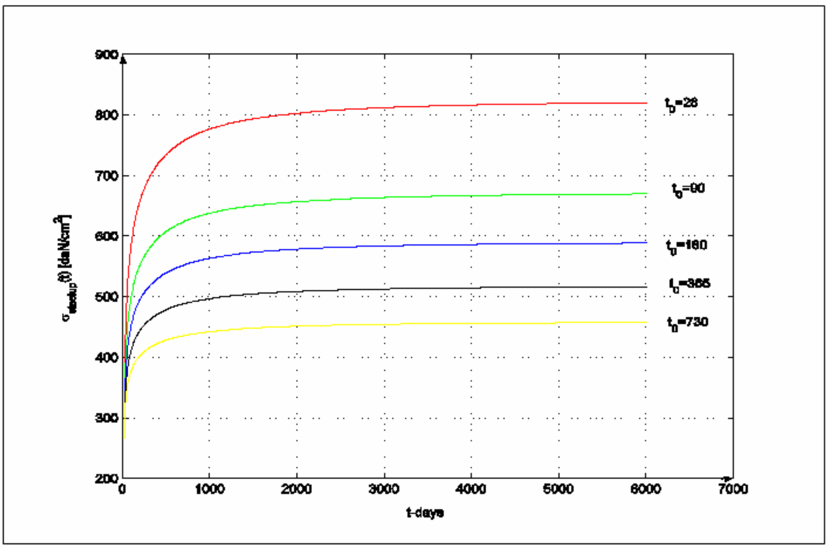

Fig. 6. Values of normal stresses in upper fiber of steel girder $\sigma_c^{up}(t)$ *in time* t_∞ *when loading is applied in time* t_0*=28,90,180,365 and 730 days*

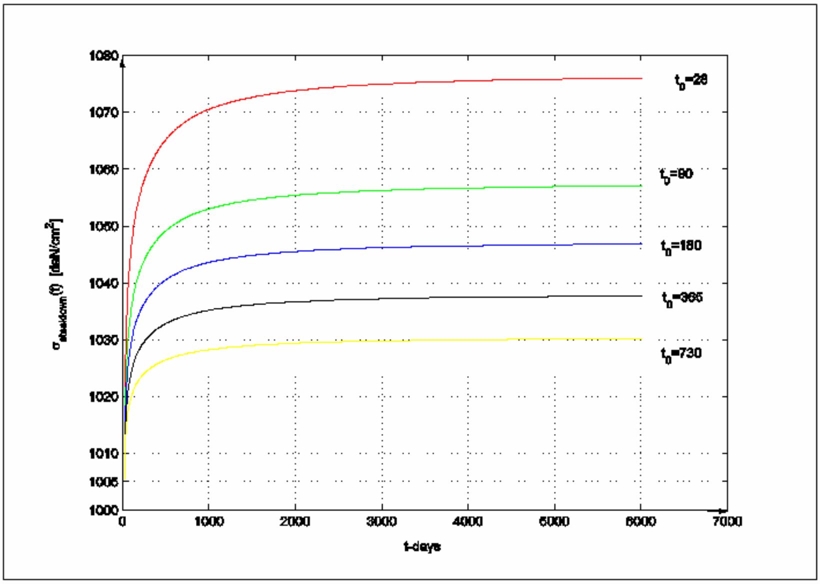

Fig. 7. Values of normal stresses in down fiber of steel girder $\sigma_s^{down}(t)$ *in time* t_∞ *when loading is applied in time* t_0*=28,90,180,365 and 730 days days*

5. SUMMERY AND CONCLUSION

Bearing in mind the creep effect, and using the integral equations (7) and (8) an universal calculating method has been elaborated for statically determinate bridge composite plate girder. This method allows the use of a perfect linear theory of concrete creep i. e. the theory of the viscoelastc body of Maslov-Arutyunyan-Trost-Bazant.

Comparing the results calculated by the numerical method and the methods of Trost and Wolff, we can say, that the numerical method gives very reasonable results from a practical point of view. So we can draw the conclusion, that the Trost's method is quite applicable and up to date. That is why, as to a new relaxation coefficient, calculated from the integral equation is not proposed, this method for a long time will be the only one for solving the formulated problem (tab. 1.).Dealing with the methods [5] based on the 'modified' Dischinger's theory, we can say, on the basis of the difference in the results, calculated according to the methods of both groups (confirm Bazant's comment [2]) – that as to the Rusch-Jungwirth's theory no improvement has taken place, especially regarding the approximation of the creep function. The latter should not be recommended for practical use.Consequently, it should be said that the numerical method gives more reasonable value of the composite girder behavior, under sustained service and leads to smaller dimensions of the steel beam.

	t_0	Normal forces and moments	Partov-kantchev's method	Partov's methods [6]	Trost's methods	Wolf's methods	Haensel's methods	Sontag's methods
Creep problem	28	$N_{c,r}(t)=N_{s,r}(t)$ [N]	861815	981728	992410	994650	934720	1033930
		$M_{s,r}(t)=N_{c,r}*r+M_{c,r}(t)$ [Nm]	1445628	1644517	1661340	1665980	1567560	1735180
		$M_{c,r}(t)$ [Nm]	29579	31438	30710	31660	31790	36330
	90	$N_{c,r}(t)=N_{s,r}(t)$ [N]	626343	718010	763970	724950	777870	795320
		$M_{s,r}(t)=N_{c,r}*r+M_{c,r}(t)$ [Nm]	1052853	1205901	1282500	1217730	1306960	1339890
		$M_{c,r}(t)$ [Nm]	23708	26138	27200	26550	28920	33090
	180	$N_{c,r}(t)=N_{s,r}(t)$ [N]	499991	575429		580330	679970	696330
		$M_{s,r}(t)=N_{c,r}*r+M_{c,r}(t)$ [Nm]	841821	968041		976750	1143960	1175390
		$M_{c,r}(t)$ [Nm]	20286	22853		23210	26760	31240
	365	$N_{c,r}(t)=N_{s,r}(t)$ [N]	387575	447601			581340	605450
		$M_{s,r}(t)=N_{c,r}*r+M_{c,r}(t)$ [Nm]	653771	754943			979410	1024010
		$M_{c,r}(t)$ [Nm]	16946	19489			24270	29180
	730	$N_{c,r}(t)=N_{s,r}(t)$ [N]	295393	342109				
		$M_{s,r}(t)=N_{c,r}*r+M_{c,r}(t)$ [Nm]	499226	578368				
		$M_{c,r}(t)$ [Nm]	13865	16248				

Table 1. Results obtained by numerical methods and exiting methods (EMM-Sontag), (RCM-Wolf), (IDM-Haensel) and (TM-Trost).

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TENTH NATIONAL AND
FOURTH INTERNATIONAL
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NOVI SAD, NOVEMBER 22-24,2006.

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A PRACTICAL SYSTEM FOR THE CONSTRUCTION OF THE BUILDING ENVELOPE ELEMENTS WITH HIGH TECHNOLOGICAL AND THERMAL EFFICIENCY

Summary: *Concepts as sustainable development or green building are growing in popularity with homeowners and builders, with more and more programs on these topics being established across the world by associations as: The International Centre for Trade and Sustainable Development (ICTSD), L'Institut Européen du Développement Durable (IEDD), World Business Council for Sustainable Development (WBCSD) etc.*

This paper presents the result of one of these programs, which consists in a new and efficient technological system for the construction of the reinforced concrete load bearing walls and roof slabs. The system propose the use of high density expanded polystyrene insulating concrete forms for the external concrete walls and roofs, providing in this way both the strength and the hygrothermal resistance requirements in one technological step.

Key words: *Sustainable development, polystyrene shuttering system, high thermal insulation, technological efficient.*

PRAKTIČAN SISTEM ZA GRADNJU ELEMENATA OMOTAČA ZGRADA VISOKE TEHNOLOŠKE I TERMIČKE EFIKASNOSTI

Rezime: *Koncepti kao što su održivi razvoj i zelene zgrade postaju sve popularniji kod vlasnika kuća i graditelja, sa povećanjem programa iz ove oblasti koje su širom sveta uspostavile asocijacije: The International Centre for Trade and Sustainable Development (ICTSD), L'Institut Européen du Développement Durable (IEDD), World Business Council for Sustainable Development (WBCSD) itd.*

U ovom radu su prikazani rezultati jednog od ovih programa, koji se sastoji od novog i efikasnog tehnološkog sistema za gradnju armiranobetonskih nosećih zidova i krovnih ploča. Sistem predlaže upotrebu oplate za spoljne betonske zidove i krovove od ekspandiranog polistirena velike gustine, obezbeđujući na ovaj način zahteve za higrotermičku otpornost i čvrstoću u jednom tehnološkom koraku.

Ključne reči: *Održivi razvoj,sistem oplate od polistirena,visoka termička izolacija, tehnološka efikasnost.*

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1. INTRODUCTION

The interest for subjects as rational and sensible administration and use of the natural resources and fossil energy deposits, local material and cultural heritages protection and development, environmental friendly constructing and living have been integrated in concepts as s*ustainable development* or *green building*. This concepts pay attention of more and more specialists in architectural and urban planning, local economic development, research and educational field.

In order to put in practice the principles and ideas, derived from the general concepts mentioned above, all over the world are founded organizations and associations as: *The International Centre for Trade and Sustainable Development (ICTSD)* in Geneva*, L'Institut Européen du Développement Durable (IEDD)* in France*, World Business Council for Sustainable Development (WBCSD), United Nations Division for Sustainable Development* in US etc. These organizations in collaboration with research centers and education centres systematize concepts, elaborate and submit helpful resolution practicable in building industry and other economical branches of the society.

The constructive system presented in this paper is proposed by „*Insulating Concrete Form Association* " a group of producers and dealers as: *Amazon Forms – LLC, American Polysteel, Arxx Building Products, Energy Efficient Wall Systems* from US*, Quad-Lock Building Systems* from Canada*, PlastBau* from Germany, *Plast Edil* from Italy etc. and represent the result of researches in the domain of buildings with low energy consumption.

The central idea of this constructive system is to build the main reinforced concrete structural elements of the edifice envelope (external walls, top floor slab) using lost shuttering of thermal insulating materials. Thereby for the external shear walls and the floor slabs over the top level (or under the roof) the requirements of mechanical and thermal resistance are fulfilled in the same technological step.

2. THE DESCRIPTION OF THE SYSTEM

Shuttering for the walls – Using semi-rigid expanded polystyrene shuttering plates (EPS) and high density polyethylene connecting ties an insulating shuttering / formwork can be assembled. This formwork (for reinforced concrete monolith walls) is adaptable for the different requirements of reinforcing on horizontal or vertical direction.

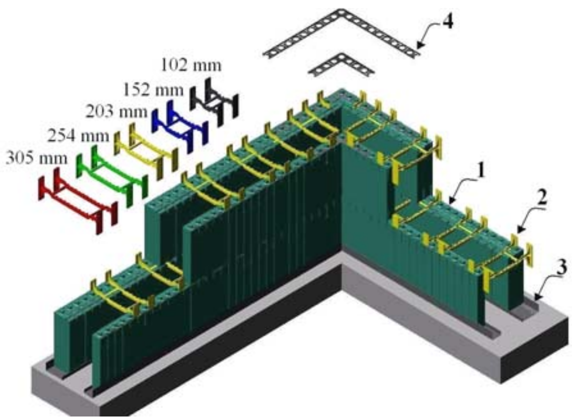

Figure 13. Wall shuttering of thermal insulting material plates:

1- expanded polystyrene plate,

2 - high density polyethylene connecting tie,

3- sheet metal profile for the alignment and the fixing of the shuttering base,

4 - aluminium angle bracket.

The 120x30 cm expanded polystyrene plate represents the formwork face and in the same time the thermal insulation of the concrete wall. The dimensions of the plate allow an easy manipulation and a great adaptability to different architectural planes. The thickness of the polystyrene plate is 57 mm fort the standard type and 108 for pus type. The polystyrene plates take the form of foamed plastic blocks separated by plastic or carbon-fibbers spacers, which lock together in the manner of Lego bricks constructions. The shuttering plates from the both sides of the wall are fixed to a constant distance by polyethylene connecting elements which bring support for the horizontal reinforcement bars of the concrete wall and works as ties against the pressure of the fresh concrete on the formwork faces. The system has also some metallic elements that gives the alignment of the wall base and provide the stiffness of the scaffolding at the corners or wall intersections.

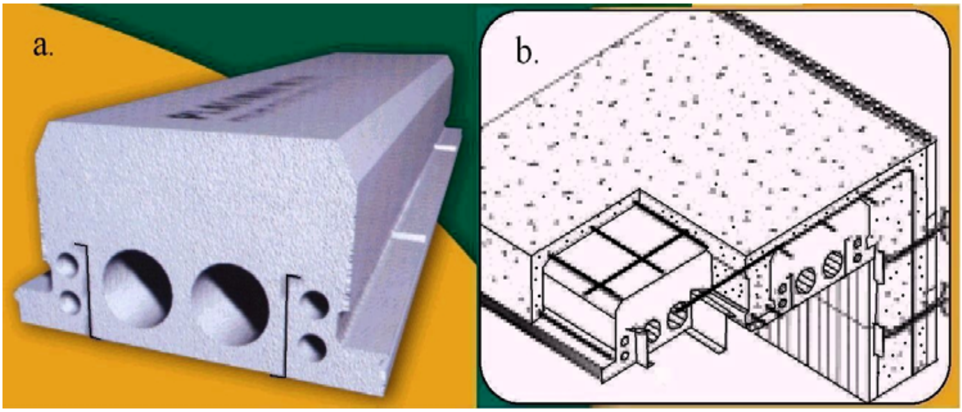

Figure 14. Insulated forming systems for floors under roof:

a *- all the expanded polystyrene floor panels are supported and reinforced with two integral steel beams moulded into the product from end to end and are provided with channels electrical or other installation systems path,*

b *– floor and wall joint shuttering,*

c *– the floor polystyrene panels are easy to manipulate although it covers large openings.*

Shuttering for the floors under roofs – in combination with the insulating shuttering for wall can be used insulating polystyrene deck forming system for floor. The expanded polystyrene floor panels are 60 cm wide, 17,5 to 34,5 cm high and their length can reach 12 m. The rigidity along the deck is given by two Z form integral steel beams moulded into the polystyrene element from end to end. Four tubular channels (two with the diameter ϕ=12 cm and two with ϕ=12 cm) placed along the deck allow for the placement of plumbing, electrical, telephone, and ventilation systems without having to remove any of the forming material. The joints between the panels are designed as a tongue-and-groove joint at the base of a 12 cm wide cavity that forms the concrete joist of the floor. The necessary number of rebar is placed in each cavity, and than the concrete is cast into the form to create joist with the dimensions cross of 12x13 cm2 to 12x25 cm2 function of the panel deck high and the level of the concrete slab poured on top of the panel.

3. THE SYSTEM IMPLEMENTATION

The insulating concrete forms system described above was used for the construction of an individual dwelling building in a new residential area near our city Iasi. From the collaboration of the authors of this paper, to the construction of the building, results some aspects regarding the implementation of this system that will be presented below.

3.1. *System benefits*

a. Flexibility for building design and site handling

- the system is presents flexibility and adaptability to complex architectural plans and site handling.
- The elements are designed to be easy to handle in the site. Once the sheet metal profile have been well balanced and fixed to the concrete base the polystyrene plates are easily fixed by gliding in the profile or pressing on the top of the under plate line.
- The elements are lightweight and just one person to be carried on short distances and two persons to be balanced and fixed on the final position.
- The reinforced concrete shear walls that results from this technology are appropriate to be used for cellular structures with large internal spaces.
- The resultant structure permits to use a reduced number of internal shear walls and for this reason indoor natural ventilation and lighting can be provided.
- The resultant elements can be easily finished using plasterboards that can be fixed in directly the polyethylene connecting ties of the wall of metal profile of the floor panels.

b. Resistance and security

- The resultant reinforced concrete monolith walls provide good strengths and stability (mechanic rigidity) to the building structure.
- A good fire resistance (about 3…4 hours) is grant due to the concrete structure.
- The materials are resistant to the water action and are unattractive for mice different types of microorganisms.

c. Energy consumption

- The external walls and floors under the roof constructed with this system offer high thermal resistance, according the type of polystyrene plates used the value of the resistance can be 3.52…3.64 m2K/W in case of two standard plates, between 4,99 and 5,12 m2K/W when are used one standard and one plus, and 6,46...6,58 m2K/W for two plus plates, for the floors under the roof (considering the concrete joists influence) the mean thermal resistance is 5,91m2K/W.

- The joints between the walls and the top floors are designed to provide the continuity of the thermal insulation in that part of the building.
- The expenses for heating of the “EPS building” in the cold season, are strongly reduced comparing to those from the buildings constructed using traditional systems that are prevalent at the moment in the individual dwellings construction industry.

d. Ecological qualities

- The panels are not toxic and present good stability along the time.
- The expanded polystyrene plates are 100% recyclable.
- This constructive system do not need to use timber in the constructive process, and in the manufacturing require a reduced quantity of water and energy than the ceramic building materials.
- Due to the high thermal resistances conferred the large surfaces building envelope elements, a substantial reduction of the heating energy consumption is achieved and implicitly the reduction of the pollutant emissions that often results in the energy fabrication process.

e. Reduced costs

- Reduced expenses to provide an optimal indoor temperature both in cold or excessive hot seasons.
- Because the panel elements are lightweight and compact the costs for transportation and storage are reduced.
- The dimensional stability and construction accuracy permitted by this system bring to economies in time and costs implied by eventuality further corrections.
- The shuttering emplacement do not demands special equipments.

f. Comfort

- The high thermal insulation achieved by this system brings about the premises of optimal thermal comfort conditions for the indoor spaces.
- The building elements presents also good sound insulation qualities.

3.2. Disadvantages

- The system is not recommended for the construction of intermediate floor slabs or load bearing walls placed in heated spaces where the thermal insulation function conferred by this technology is not required and the supplementary costs are not justified.
- The thermal insulation placed on the internal face of the wall forbid the massive reinforced concrete wall or floor slab to act as a thermal flywheel in order to reduce the temperature oscillations in the case of batch heating process or for periodical natural ventilation of the indoor spaces (if there are not used conditioned air systems).
- In the constructive process the system requires pneumatic pumping devices for the wet concrete placement in the forms.
- Because the concrete is considered in the common conscience as a cold and unfriendly material, the idea of dwelling made of reinforced concrete can be unattractive for the possible investor in dwelling buildings.

- The shear wall constructive system can be expensive in case of a ground storey dwelling because it is used under the optimal possibility of the bearing capacities.
- Due to the low resistance against mechanic shocks of the polystyrene boards, at list on the lower part of the external wall, the façade must be protected against mechanic actions.

4. OPINIONS REGARDING THE APPLICATION OF THE SYSTEM IN OUR COUNTRY

The system, with an actual technological and hygrothermal efficiency, may be used especially in regions where is a lack of local construction materials, so that the reduced expenses of transport and storage implied by this system will determine an accession of the economic efficiency.

The system is recommended in seismic regions, with the condition to place the reinforcement bars in concordance with the specific regulations and standards for the seismic zone, according to an adequate design of each individual building.

5. CONCLUSIONS

Created in the last years, the expanded polystyrene shuttering (EPS) system, such as other new constructive systems or elements (Porotherm ceramic elements, Isover, Swisspor, Baumit insulating façade systems etc.) take aim to achieve optimal hygrothermal and economic efficiency. For this reason the system deserve the attention of the constructors and investor in building industry.

The manner how the system is used can be optimised taking into account several economic criteria.

Until the elaboration of some standard projects or design regulations for this type of expanded polystyrene shuttering (EPS) system, each objective constructed with this system must be designed separately in concordance with the specific regulations our country.

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TENTH NATIONAL AND
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ARE PARTNERSHIP, TRUST AND BONDING ESSENTIAL ELEMENTS OF SUPPLY CHAIN MANAGEMENT IN THE AUSTRALIAN CONSTRUCTION INDUSTRY RESIDENTIAL SECTOR?

Summary: *Concepts and definitions of supply chain management in achieving and sustaining a competitive advantage for industrial companies diverge from logistics, management of distribution channels from suppliers to end users, to good relationship with business partners. The aim of the paper is to highlight an understanding and application of supply chain management in the Australian residential sector. The research design applied a case study analysis based on interviews conducted with eight selected companies. For all companies approached supply chain management is understood as a partnership relation to suppliers. From the point of view of trust and bonding, a respect to suppliers developed through experience was indicated as essential. The paper highlights the importance of having good relationship with suppliers in terms of developed partnership, trust and bonding in order to remain competitive.*
Key words: *supply chain management, partnership, trust, bonding*

DA LI SU PARTNERSTVO, POVERENJE I POVEZIVANJE OSNOVNI ELEMENTI LANCA MENADŽMENTA PONUDE U STAMBENOM SEKTORU AUSTRALIJSKOG GRADJEVINARSTVA?

Summary: *Koncepti i definicije lanca menadžmenta ponude u sticanju i održavanju kompetitvne prednosti industrijskih kompanija variraju od logistike, menadžmenta distribucionih kanala od snabdevača do krajnjih korisnika, do dobrih odnosa sa poslovnim partnerom. Cilj rada je da ukaže na shvatanje i primenu lanca menadžmenta ponude u australijskom stambenom sektoru. Primenjena metodologija je case study analiza bazirana na intervjuima u osam odabranih kompanija. U svim kompanijama lanac menadžmenta ponude je shvaćen kao partnerstvo sa snabdevačem. Sa aspekta poverenja i povezivanja, pokazalo se da je odlučujuće uvažavanje snabdevača. Rad ističe značaj dobrih odnosa sa snabdevačima kroz partnerstvo, poverenje i povezivanje sa ciljem održavanja organizacijske konkurentnosti.*
Ključne reči: *Lanac menadžmenta ponude, partnerstvo, poverenje, povezivanje*

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1. INTRODUCTION

Building and Construction Industry (BCI) has characteristics that distinguish it from other industries. Apart of the physical nature of the product that is mostly large and expensive, three separate groups of people - client, designer and contractor - are involved in a construction process. Client is an initiator of the building and construction process. Designer is a planer of principal activities. Contractor executes the building and construction job in residential sector, non-residential sector and engineering construction area (Australian Bureau of Statistics, 1993).

The work system in Building and Construction Industry (BCI) is based on projects. Each project incorporates several organisations subcontractors that operate with their own objectives and pressures. In order to organise a building and construction process to function smoothly, the project manager has to control overall costs, time and quality of actions undertaken. Project management activity is temporary, but exposed to a constant pressure of time and cost constraints, competitive tendering and a practice of awarding contracts to the lowest bidder (Holmse, et al,1999).

Last two decades witnessed rapid growth in the concept of supply chain management (SCM being) adopted by organisations in all types of industries and has been defined in a number of different ways. Cooper and Ellram (1993) described SCM as a philosophy to manage distribution channel from supplier to the ultimate user, while Christopher (1997) and Saunders (1994) viewed it as procurement and logistics. Kiefer and Novack (1998) define supply chain as an integrated collection of organizations that manage information, product, and cash flows in order to maximize consumption satisfaction with minimal total costs. SCM is also understood as a network of organizations in delivery channels that produce value for customers, and contributes to achieving and sustaining a competitive advantage (Cox, 1999a, 1999b, Towill et al, 2000

There are also opinions that SCM should be defined differently in different industries. According to Titus and Bröchner (2005), SCM in the BCI deals with management of materials, but also with relationships between contractors, suppliers, and distributors.

Relationships between parties, or partnerships, are seen in the literature as a commitment between the client and supplier, cooperating to meet "separate but complementary objectives" (Blayse and Manley, 2004).

Partnerships can take many forms. According to Welling and Kamann (2001) supply chain partnerships in BCI can be at the firm level or the project level. At the firm level supply chain partners can have stable and long-term working arrangements among a limited number of firms. At the project level relationships are more temporary as they are usually established for the duration of the project (Dubois and Gadde,2002). Koskela and Vrijhoef (2001) argue that temporary partnership is an inhibitor to innovation. This opinion is supported by Dulaimi et al (2002), who in looking at the need for integration and innovation in construction, note that the Australian BCI was criticised for its low investment in research and having fragmented supply chains which affected its capacity to compete internationally.

Although partnership plays an important role of SCM in BCI, according to Moberg and Speh (2003) development of trust and commitment is also significant.

Trust as the willingness to rely on an exchange partner in whom one has confidence (Moorman *e*t al., 1992) has many dimensions (Smith and Barclay, 1997). It indicates that partner can be trustworthy in one dimension but not in another. Wang et al

(2004) suggest that traditionally, trust develops from specific events and repeated interaction, in relationship development. Thus, trust is strongly associated with commitment and loyalty. Another way of developing trust is a cue-based trust, or trust generated by an initial encounter with a stimulus, usually a symbol or sign, which is associated in the viewer's mind with trust.

Wood, et al (2002:4) explore the issues of the ethical benefits of trust-based partnering and state "that engaging in trust-based partnering encourages parties to adopt higher ethical standards, and achieve improved ethical performance in all their business dealings". It is intimated from this statement that trust could affect the efficiency and effectiveness of a firm's operations, depending on the partners they choose.

Lane and Bachmann, (1996) note that within a supply chain, inter-organisational trust is important in maintaining a competitive advantage. If trust is developed through contacts between parties, it then becomes a bond or a tie that brings partners together.

Bonding can take two forms: structural and social (Wilson, 1995). Structural bonds are those economic and strategic ties that link buyers and sellers, such as legal contracts and agreements. On the other hand, social bonds are made up of personal and social ties between individuals in organisations.

Wilson (1995) notes that structural bonds are important in any business relationship, but those organisations that have strong social bonds generally have a greater commitment in their relationships. Social bonds focus more on the interpersonal relationships between and in organisations; they may include creating a family type atmosphere and consideration for the other party when making decisions.

The paper aims to find out what does SCM mean in selected BCI organisations and what role partnership, trust and bonding play in improving the organisations competitive advantage. After the introduction an explanation of the methodology is presented, followed by the findings, discussion and conclusions with suggestions for future research.

2. METHODOLOGY

The qualitative research undertaken included in depths interviews conducted with eight selected companies in the Australian BCI residential area.

Table 1: BCI total value in 2005

Value of work done	AUD $m	%
Residential Building	8728	38
Non-Residential Building	4831	22
Engineering Construction	9168	40
Total Value	22797	100

Residential sector (See Table 1) makes 38 per cent of total value of the Australian BCI. The sector has remarkable annual increase of 9 per cent notified within last few years (Australian Bureau of Statistics, 2006).

Residential building area can be defined as the construction of new dwellings, but also the alterations, additions, renovations and general repairs to such buildings. In our research we have concentrated on the new buildings sector. Much of the activity in this sector is conducted by both private and public organisations, with the private sector playing the significantly larger role.

The research design was a semi-inductive approach aimed at learning from the field research rather than testing existing variables (Yin, 2003). An interview protocol with open-ended questions was developed setting out the statements upon which the discussions were based.

Initial requests for interviews were made to the Managing Directors of each company requesting them to nominate managers that were more appropriate respondents for issues to be covered in the study. In depth face-to-face interviews were held by a researcher who had background in construction engineering. The interviews lasted one-and-half to two hours and were held at the respondents' offices.

According to De Vaus (1996), the choice of the sample for case study research needs to reflect the characteristics of the group from which it is drawn. Thus, eight case companies were selected to cover the various sub-categories of the residential BCI in the Greater Melbourne area.

Table 2: **Companies Involved in the Research**

Company	Company Description	Interviewee's Position
A	- Small size - Middle to high size	Director/Builder
B	- Medium size - Low income	Administration Manager
C	- Large franchise - Low income	Operations Manager
D	- Leading construction and development firm -Specially or architecturally designed homes	Building Manager of one of firm within the group
E	-Corporate property development group -Specially or architecturally designed homes	Cost Planning Manager
F	-Corporate developer and builder -Middle to high income	Supply Manager
G	-Multi-national project management and construction firm -Units, apartments or cluster houses	Senior Site Manager
H	-International construction firm -Units, apartments or cluster houses	Senior Site Coordinator

Table 2 provides details of the companies involved in the interviews. For a variety of reasons, the company names are not identified and are simply referred to here as Company A to Company F. All interviews were tape-recorded and transcribed.

The companies were classified as

- low-income housing construction ($100,000 to $300,000)
- middle to high income construction ($300,000 and above)
- specially or architecturally designed homes (also included green houses)

units, apartments or cluster houses.

The questionnaire consisted of the following parts:

- What does SCM mean for selected company?
- What is the role of partnership and trust in the SCM?
- What is the role of bonding in the SCM?

3. FINDINGS

The research findings are presented by the questionnaire parts.

3.1. Meaning of SCM

All approached companies base their work on project management. Accordingly, supply chain partnership is at the project level facing only networks between the company and its suppliers. All interviewees indicated that supply chain is a system to manage goods and services in order to ensure that projects were completed on time and within budget.

Operating the supply chain differs across the companies. Very small companies tend to have multi-task staff members. In contrast, larger organisations (Companies C-H) have standardised processes with an established list of suppliers, through formal contracts.

Irrespective of size, all organisations agree that supply chain arrangements are important to offer an attractive price to the client and high quality of service and products.

3.2. Partnerships and Trust

While all companies regarded relationships as important, small firms tended to have a more “personal” relationship with their suppliers. The messages, however, were mixed on the degree of trust shown by firms towards suppliers. Nearly all interviewees claimed to trust their suppliers. Nearly half of the firms, spread across small and large categories, specifically referred to their respect for their suppliers.

3.3. Bonding

Both forms of bonding, structural and social, as identified by Wilson, (1995), were found in our study to varying degrees, across all companies. The structural bonds, such as legal contracts and agreements, tended to be found in larger firms, although the actual structure varied. The situation in the smaller firms varied, ranging from Company A which knew the suppliers personally for five to ten years to Company B who knew the suppliers but always checked the prices each time.

There were two aspects of social bonding activity identified – supplier initiated and firm initiated activities. All firms noted that such activities occur to some extent.

Supplier activities varied, ranging from low-key social gathering to going to Christmas lunch and suppliers tents at the races (Company B). All the construction firms realised that being entertained by suppliers had a business outcome.

In terms of company's initiated activity it was noted that 70 percent of firms, large and small, claimed to maintain a healthy relationship with suppliers, especially those suppliers who could affect their bottom line, through limited reciprocity.

4. DISCUSSION

For all companies we have approached the SCM is understood as a partnership relation to suppliers. That obviously indicate an understanding of SCM close to Titus and Bröchner (2005), but still different as it does not include relationship wit distributors. Therefore, SCM in BCI seems to be one sided.

Because of the nature of the work in BCI based on tripartite relation: client, contactor and designer, of which we have researched the contractor part – it is clear that the only contractor supplier side was relevant for understanding the importance of SCM to improve organisational competitiveness. None of the approached companies had supply chain partnership at the firm level. In stead, in all eight cases supply chain partnership was based on the project level. According to Welling and Kamann (2001), relationships with partners are temporary, established for duration of the project. Our findings, however, indicate that in some cases (Company H and Company G) partnership was not temporary.

It was found, overall, that the processes in supply arrangements are similar across the BCI residential sector. However, the relationships between the suppliers and the BCI firms underpinning these arrangements appear to range from a "personal" approach in the case of smaller firms to a more "contractual" or "business-like" approach with larger organisations. This latter situation is closer to a more conventional supply chain process, with a focus on what the literature refers to as collaborative strategic alliances.

It seems that because temporary partnership prevails in the approached companies of our research project, it may become an inhibitor to innovation in BCI, as noted by Koskela and Vrijhoef (2001). Further, according to Dulaimi et al (2002) it may increase research in the industry and through supply chain influence the industry competitiveness. In an unexpected result it was found that all firms indicated they relied, to varying degrees, on suppliers to provide advice on research and innovation. This support, both reactive and proactive, was found to be especially evident with larger firms, covering all aspects of the supply chain.

The factors of trust and bonding in particular emerged as key areas of relationship development, consistent with the findings of Wang et al (2002). Again, consistent with the findings of these authors, it was noted that companies specifically mentioned respect for their suppliers as important. This respect appears to have been developed through experience rather than any cues – this statement does not mean cue-based trust did not exist, just that there was no evidence of it in the responses during the interviews. Yet a theme of tempering trust was found in several examples spread across all sized firms; larger firms talk about working together and helping each other - bonding and trust - but they also have these expectations written into contracts. Such behaviour raises the issue identified by Moberg and Speh (2003) and Wood et al (2002) - to what degree do these firms actually trust each other and how will that affect relationships and the effective management of the supply chain? It also highlights a degree of vulnerability felt by firms, consistent with Moorman et al (1992).

Structural and social bondings, as identified by Wilson (1995), were found across all companies to some extent. Structural bonding was particularly prevalent in larger firms, possibly due to the need for more formal arrangements related to accountability issues. Wilson (1995) suggests that social bonds consist of personal and social activity, that organisations with such bonds generally had a higher commitment in their relationships. However, our research showed that this was not necessarily the case in the BCI. While social bonding appeared to be accepted as a practice it was not encouraged by all firms, particularly the larger firms - with at least one firm actively discouraging such bonding.

5. CONCLUSIONS

The research project conducted in the Australian BCI companies in residential sector has demonstrated that SCM is of importance to achieving and sustaining competitive advantage. SCM is understood as a temporary partnership with suppliers based on the project level. Since temporary partnership, as suggested by Dulaimi et al (2002), has potentials to increase interests in research development in the BCI organisations, it may lead to innovations that should improve the BCI competitiveness.

Relationships with suppliers were found to be more "personal" in the case of smaller firms, while with larger organisations they had more "business-like" approach. From the point of view of trust and bonding, a respect to suppliers developed through experience was indicated as important. Structural bonding was characteristics of larger firms, while social bonding was applied in smaller and medium firms.

Our research was limited to a small selection of firms providing several opportunities for further research. A more extensive survey needs to be conducted to verify initial findings in other remaining sectors of BCI; that is in non residential building and engineering construction sectors. The possible link between trust, bonding and innovation is also worth exploring given the need for BCI firms to be constantly innovating for competitive advantage.

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EVALUATION OF INVESTMENT PROJECTS USEING BLCC (BUILDING LIFE CYCLE COST) METHOD

Summary: *This paper analyzes possible fields of application of BLCC (Building Life Cycle Cost) software. Software was developed in USA and it is used for calculation of buildings life-cycle energy savings. This model produces multyleveled grading of building scenarios and comparative analyses of grading results. The comparative analyses takes into account initial building investments and energy cost savings through certan period of biulding`s use. Each scenario (model of the fasade and energy use of building) is compared with the basic scenario (fasade model and enegy use of the initial design), and each scenario has a different increase or decrease of initial investments in comparation with the basic scenario. The overall behavior of different scenarios is mesured according to: (a) initial/capital investments, (b) energy costs (c) operating, maintenance, and repair (OM&R) costs and (d) capital replacement costs. The benefits of results are graded as: (a) measurable and immeasurable, (b) direct and indirect and (c) material and nonmaterial. Results of the analyses are used as quantitative inputs for investment decision making.*

Key words: investment project, life cost analyses, building energy comsumption.

VREDNOVANJE INVESTICIONIH PROJEKATA METODOM BLCC (BUILDING LIFE CYCLE COST)

Rezime:*Predmet ovog rada je analiza mogućnosti primene kompjuterskog programa BLCC (Building life cycle cost) koji je razvijen u SAD za vrednovanje korisnosti ušteda energije u životnom veku objekta. Model omogućava višekriterijumsko ocenjivanje varijantnih rešenja i njihovu komparaciju, odnosno rangiranje kao pomoć kod donošenja investicionih odluka. Osnovni metodološki postupak je komparativna analiza varijanti projekta na osnovu troškova invetsicije, sa jedne, i ušteda u veku projekta sa druge strane. Svaka varijanta (scenario) ima specifično povećanje ili smanjenje investicionih ulaganja u odnosu na osnovni model (projekat). Merenje se vrši na osnovu ukupnog učinka određenih rešenja za redukciju potrošnje energije prema troškovima: (a) investicije, (b) energije, (c) tekućeg održavanja i (d) periodičnih remonta koji su neophodni u veku projekta.Koristi koje se vrednuju kao: (a) merljive i nemerljive, (b) direktne i indirektne i (c) materijalne i nematerijalne. Rezultati analize služe kao merljivi kvantitativni podaci na osnovu kojih se donose pravilne investicione odluke.*

Ključne reči: investicioni projekti, life-cost analiza, energija objekata.

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1. INTRODUCTION

In the work are presented starting assumptions of the possibility of application and necessary modifications of the BLCC program which are the subject scientific project "Model of economic and ecological valutation of justification of opti,isation of energetical efficacy of buildings and use of alternative and restored energy sources in buildings", which is done at the faculty of Civil engeneering in Belgrade.[1] Computer program BLCC (Building life cycle cost) which is developed in USA for the evaluation of advantage of saving energy in whole life of building. With certain modifications, this program was used for analysis of some investment projects in Serbia during the evaluation of economical justification of different solutions of application of some measures of optimization of energetic efficiency on (a) building cover (b) building structure (c) lighting and (d) system of term technique. The aim of this research is adoption of unique model of varied evaluation of justification of energetic optimization from the position of consummation of primary energy and use of alternative and reconstructed energy sources (AiO) - in concordation with adequate directives of EU, ISO standards and with satisfaction of specific needs of approach of domestic economy for the establishment of the way for permanent development. The contribution of this scientific project is in overcoming of model for the project evaluation and bringing investment decisions with whom are favored energetical efficacy and ecological acceptance of the new solutions of AiO. The field of application of evaluation model for the economical efficacy :

- Reconstruction and modernization
- Development of infrastructure
- Hardwer and softwer solutions
- Substitution of energy sources
- Application of organization and technical measures for the rationalization for using of energy

2. METHODS AND CRITERIA

Basic methodological action is comparative analysis of project variations in the basis of investment expenses, from one side, and saving in the duration of project, from the other side. The duration of project will be defined as a result of specific technical and economical amortization. With the method of BLCC (Building life cycle cost) is

[1] Scientific project is financed by the Ministery of science and protection of natural environment. This model will be applied in real projects - investment studies which are done at the Faculty for Civil engineering and Architecture. The guarantee fund of Serbia, which supported this research, will renounce to the exploring team investemnt studies for the verification of the valutation or for the justification and acceptance from the aspect of energetic efficacy and rational using of AiO sources of energy.

confirmed the net actual value of project varieties - to be possible to find the solution which gives the best results during the life period (the period of 25 years). Each variety (scenario) has a specific increasing or reducing of investments regarding the basic model (project). The measuring is performed in the basis of total effect of determined solutions for the reduction of energy consumption according the expenses : investments, energies, current maintenance and periodical repair which are necessary during the life of the project[1].

In the final analysis the varieties are ranked according each of included criteria. The financial and market analysis and evaluation of the project presume the market prices and individual time preferences, while the analysis of the ecological aspects relates to the acceptance of the project regarding the pollutions. So are determined **Elimination criteria** for the project which are :

- Internal rate of rentability (ISR),
- Net actual value of the project (NSV),
- Time for refundation of invested capital (VPS),
- Ecological acceptance of the project

Expected results of the research are: (a) defining of relevant entrance data, (b) establishing of the criteria for the valuation and (c) quantitative validation of varieties of the solutions for the optimization of energetical efficacy and application of AiO energy resources. Data bases which are used for the validation of economical and ecological justification of investments in the field of energetic efficacy are:

- Prices and expenses of the material
- Prices for the work
- Prices of the energetics
- Discount rates
- Interest rates

This model should enable trustful validation of varieties and choice of the solution when deciding of the application of the measures which optimize the expenses and interests - according all the criteria. Final validation of the economic efficacy and social justification are used for bringing up of the decisions when the location should be found, for the kind and power of the new energy resources. The choice of criteria and methods of the validation of technical, technological and economic solutions enable the choice of the variety which mostly regards the principles of Kyoto protocol of the UN, or reducing of emission of CO_2 in the period from 2008. to 2012.

[1] It is thought about so called "economical duration of the project".

3. FINANTIAL ASPECTS OF THE VALUTATION

The main task of economic analysis is the validation of profit of intended project. The decision about the acceptance is based on knowledge that *the benefits of the project exceed its expenses*. Classic economic evaluation favorite NSV, ISR and VPS. Establishing of the criteria for the evaluation of the project ***includes three basic indexes of economic efficacy of the project, or variants::***

- IRR Internal rate of return
- NPV Net present value
- Simple Pay Back Period

First of all, depending on specific economic and finantial conditions, it is established the order and value of each quoted criteria for the evaluation of economic and finantial efficacy of the project. Sometimes it is the length of pay back period but most often it is net present value of the project as an index of present value of the future savings. The method of the evaluation of BLCC shows the results through three areas :

- valutation of finantial and market efficacy of the project, where is established the justification of the investment under the real market conditions, measured with the accumulation of the project,
- Valutation of social and economic efficacy of the project, which estimate the effects of the project to the social and economic development of the country,
- Valutation of the efficacy of the project under the conditions of changing of the key parametres, or valutation of the sensibility of the project under the aggravation of projected conditions.

The results and the expenses analysis and saving during the life period for a set of chosen scenario in the final analyses are ranked under each of included criteria. The adaptation of BLCC and development of the model for the evaluation of economic and ecological benefits of energetic efficacy of using of renovated and alternative (AiO) energy resources. Methods and criteria for the evaluation of the justification of investment in projects of optimization of energetic efficacy will result with the model of synthesis evaluation. This model must be compatible with the criteria and measures of social and economic justification of investment which are applied for the evaluation of acceptance of the credit projects in domestic and foreign financial institutions. Cost-benefit analyses enable the calculation of net profit or internal rate of interests for the investment in some improvements. The basic methodological procedure is comparative analysis of variants of the project in the basis of investment expenses from one and savings during the life period of the project from the other side. The measurement of the total effect of some certain solutions for the reduction of energy consumption according the expenses :

- investments
- energy
- current maintenance and
- periodical maintenance which is necessary in this period.

The subject of the analyses from the aspect of economic justification of the project is:

- Comparation of the model from the viewpoint of total amount of consumption and quality of energy from one and viewpoint of economic factors of application of the optimization measurments of the efficacy to the covering of the building, building structure and lightening and termo technic from the other side.
- The analysis of the life period of considered measures for the improvement of energetic efficacy of the covering, building structure and system of termo tecnnic in total.

Besides savings on the projects of the optimization of energy efficacy, are valued also the effects of reduction of the emission of harmful materials as a kind of project benefit. Also, in contrast to the savings, some other benefits from the group "outside the project", belong mostly to the society, and not only to the investor. The types of benefits of energetic efficacy as social and economic reasons which need not to be quailified are so called "public goods", "abundant effects" or externals. The criteria for the evaluation of social benefits are protected life surroundings and non reconstructed resources, and the influence to the tecnical progress, quality of population life, increasing of consumer excesses and similar. The values of external effects of the project as a benefit for the choice of the best solution are shown as :

- Protection and developemt of the life surroundings
- Long lasting development of energy sources
- Rationalisation of investment funds
- Influence to the technical progress

The benefits of the project present the realized savings. The benefits are valued as: (a) measurable and non measurable, (b) direct and indirect and (c) material and non material. The forecasting of project behavior in the circumstances of uncertainty and risks of changing of key inputs of the project. Each variant (scenario) should include the evaluation of the influence of the aggravating of the basic prices inputs - as a evaluation of the sensibility of the project to the expenses.

4. ECOLOGICAL ASPECTS OF THE EVALUATION

Beside savings in installed forces and total consumption of all kinds of final energy, besides the share of substitution of alternative energy resources, in the projects of optimization of energetic efficacy and application of renovated sources of energy are valuated the effects of reduced emission of harmful materials as well as a kind of profit of this project. The reduction of energy consumption means the reduction of the gas emission. The acceptance of the project from the ecological aspect are evaluated with measurable indexes of pollution of water, air and ground. In accordance with practice of countries which are in EU, which have legal sanctions for each exceeding of the emission of harmful gasses, domestic practice will find the solutions which have less annual

quantities of polluters. The aim of these analysis is receiving of data - information which contain the necessary measures for the reducing of consequences of climate changes as:
Reduction of gas emission which cause the effect of glass garden
Their implementation in future and application at the adaptation of existing construction fund
Cooperation between scientific and technical researches in the aim of development and promoting of the new system of observation
Observing and recording of economical and social consequences of the application of different strategies by state organs, all members of Protocol.
Cooperation and promotion on the national level of the program for development, education and expert's training for the work on the exploration field of expected climate changes.
Adopted criteria and methods of the value of techno-economic solutions enable the choice of the variant which mostly take into consideration the principles of the Kyoto protocol of UN, and principles of the reduction of CO_2 emission in the period from 2008. to 2012. Approximately one third of the CO_2 emission exists as a consequence of heating, warming and lightening of the objects. From that reason, in the program of BLCC is incorporated the calculation of the gas emission regarding the type and energy consummation. The analysis for the CO_2, SO_2 and Nox (nitrogen evaporating) are performed. With the calculation is included the whole life period of the object and the value is showed in kilograms on the annual level.

5. CONCLUSION

The model of BLLC includes the evaluations of all elements of the financial, economic and ecological analysis for the planning and following of the investment, and the planning of expenses and activities, management of the realization and exploitation of the object. When all the criteria are observed as equally worth, the decision will be brought by finding of these solutions satisfies the most both of these conditions. Each variant (scenario) has a specific increasing or reducing of investments regarding the basic project. In the final analysis the variants are ranked by each of included criteria. The program gives the possibilities of choice like :

- The variant which is most favorable from the aspect of the lowest expences of the life period
- The variant with the shortest time for return of invested means
- The variant with the smallest emissions of harmful materials.

One of the solutions which shows the best performances according the defined criteria is the one that has the lowest indexes either for the expenses of tightening or emission of the harmful materials in the life period. The evaluation includes also the using types from the group "out of the project" which belong to the society and not only to the investor, and the benefits of energetic efficacy as social and economic reasons which need not to be quantified. Those are so called "public benefits", "overflowed effects" or externals. Social benefits are: (a) rationalization of the use of energy resources, (b) rational use of

investment funds (c) determination of the optimal national implementation EI directive and standards of EU, (d) protection of life environment. Model of BLLC enables multiply evaluation of variant solutions and their comparasion, and ranking as a kind of help when bringing decisions. The aim of these analysis is to make possible the evaluation of justification of energetic optimalization from the view of the consumption of the primary energy in accordance with EU-ISO standards. The result of analysis serves as measurable quantitative data in the basis of which are brought regular investment decisions.
Simple and reliable methods of the evaluation of economic efficacy make easier very much the choice between more alternatives of investments. The ranking of projects for the financement according to the competitive system is based on the most important factors of the economic efficacy and ecological acceptance of the project and for intended investments.

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COMPARATIVE ANALYSIS OF BEHAVIOUR OF TRANSVERSELY LOADED PILES TREATED BY CLASSICAL METHODS AND BY FINITE ELEMENT METHOD

Summary: *In the paper are presented results of the fulfilled analysis of behaviour of transversely loaded piles, whose purpose was as accurate as possible predicting of behaviour of the pile and the surrounding soil, as well as determining of governing influences needed for design. The pile and the surrounding soil are firstly treated by classical analytical methods and linearly deformable Winkler's space, and then using numerical methods based on the concept of discretization of the problem with application of method of deformation and the finite element method.*

Key words: pile, soil, transverse loading, interaction, modeling, finite element method

UPOREDNA ANALIZA PONAŠANJA POPREČNO OPTEREĆINIH ŠIPOVA TRETIRANIH KLASIČNIM METODAMA I METODOM KONAČNIH ELEMENATA

Rezime: *U radu su prikazani rezultati sprovedene analize ponašanja poprečno opterećenih šipova čiji je cilj što tačnije prognoziranje ponašanja šipa i okolnog tla, kao i nalaženje merodavnih uticaja potrebnih za dimenzionisanje. Temelj se najpre tretira klasičnim analitičkim metodama i linearno deformabilnim Winkler-ovim prostorom, a zatim primenom numeričkih metoda zasnovanih na konceptu diskretizacije problema metodom deformacije i metode konačnih elemenata.*

Ključne reči: šip, tlo, poprečno opterećenje, interakcija, modeliranje, metoda konačnih elemenata

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1. INTRODUCTION

During the construction foundation making, piles, as foundation structure elements, are very often loaded with considerable transverse (horizontal) loads besides the longitudinal (vertical) loads. Piles are most often supports for bridge columns, power transmission line columns, chimneys, towers, and they are very convenient for building od retaining structures in the form of continual walls, where the influence of the transverse load is dominant.

During the interaction of the pile and the surrounding soil at transverse loading, there occur displacements and member forces in the pile whose distribution along the pile length, and location of emerging of maximal values depend on soil characteristics, length, stiffness, and supporting conditions of the upper and lower pile end.

In order to determine as accurate as possible the behaviour of the pile at load, it is necessary that calculation models comprehend the mentioned parameters, modeled in such a way to reflect the site situation as truly as possible. In that purpose, in our engineering practice are used different methods that are most often based on application of different soil models.

2. CALCULATION METHODS

Calculation methods for transversely loaded piles are based on application of one of the following soil models:

- linearly deformable Winkler's model;
- linearly elastic, homogenous subspace;
- non-linearly elastic and non-homogenous subspace.

Most of the methods found in practice treat the soil by the simplest, and the easiest for application, Winkler's model. Here the soil is represented by system of independent linearly elastic springs, where deformations occur only in those springs where the load occurs. The soil is most often described by one parameter – soil reaction coefficient, which is in most cases taken as constant, or linearly increasing vs. depth. Using this model, two groups of methods for calculation of transversely loaded piles are developed: first, which starts from the differential equation of the elastic line of the pile, and it solves the problem in analytical form, and the second, which is based on the concept of discretization of the pile and the surrounding soil, and which found its wide application in considerable measure in numerous non-commercial and commercial software.

Methods based on the assumption that soil is a linearly elastic, homogenous continuum describe the soil using two parameters: modulus of elasticity E_s (which also can be constant or variable vs. depth) and Poisson's coefficient ν_s. Analytical procedure of problem solving based on this model, substantially more complex for application, has not found wider use in application just because of the bulkiness of the solution. This model of soil may be met more often in procedures of discretization and solving the problem using the finite difference method and the finite element method.

Much more complex non-homogenous and non-linearly elastic constitutive models of soil have not found more considerable use due to the large number of unknown material constants and difficulties in their determining. Also, it is very complicated to treat those models by mentioned numerical methods, so there has not been even developed in more considerable measure software that describe the interaction between the pile and the surrounding soil in that way.

Though the development of computer technology made conditions for analysis and calculation of the problem in an essentially better way, with application of more perfect soil models and adopted much more accurate theoretical postulates, and possibility of more complete comprehension of geometry, supporting, mechanical characteristics of material, load, and other external influences, one may say that still there has not been reached the calculation procedure that would have given the results which would completely correspond to the measured results.

3. APPLIED CALCULATION METHODS

These problems were treated by several authors, whose papers may be found in foregn as well as in domestic literature. This paper too represents further extension of research of this problem in purpose of choosing the optimal calculation method of transversely loaded pile. In that purpose, in this paper is treated example of the pile described in [5], by different calculation procedures. First, the initial parameters method was applied, and then several software packages.

The initial parameters method treats the problem analytically and it gives the solution in closed form suitable for use in practice. Soil is represented as linearly deformable environment, which is characterized by the horizontal soil reaction coefficient $C_{H.}$ This coefficient, expresses the dependence between the horizontal displacements of the axis points of the vertical deformable pile (y) and reactive pressures of the soil p(z) in the form of :

$$C_H = \frac{p(z)}{y(z)} = f(z)$$

(1)

where $f(z)$ is function of distribution of soil reaction coefficient vs. depth.

In solving the problem of transversely loaded piles, it is most often started from constant value $C_H = f(z) = C_h = \text{const}$ or linearly increasing value $C_H = f(z) = C_h \dfrac{z}{h}$, though there exist other, more complex distributions of soil reaction coefficient vs. depth.

Depth in which the governing value of soil reaction coefficient C_h is calculated according to [2] and [8] is $h = 2\ (d_š + 1)$, so it is $C_h = mh$, where $d_š$ is pile diameter (m), and m is soil characteristics (kN/m^4) that defines the variety of C_H vs. depth.

In the interaction of the pile with the soil, the part of the soil around the pile is engaged, so the calculation width (diameter) is introduced in calculation:

$$b_{rač} = k_\phi (d_š + 1) \tag{3}$$

where: k_ϕ - shape coefficient (0.9 for circular and 1.0 for square and rectangular shape)

Starting from the basic differential equation of the elastic line of the pile, which for constant reaction coefficient in horizontal direction, $C_H = C_h = \text{const}$, stands:

$$EI\frac{d^4y}{dz^4} + yC_h b_{rač} = 0 \tag{4}$$

while in case of linearly increasing soil reaction coefficient, it has the form:

$$EI\frac{d^4y}{dz^4} + yC_h b_{rač} = 0 \tag{5}$$

where EI is bending stiffness of the pile. Solutions of the equations (4) and (5) according the corresponding boundary conditions are known in literature and given in analytical form.

For $C_H = C_h = \text{const}$ according to [2] and [8] is:

$$y(z) = \frac{2\lambda}{C_h\ b_{rač}} e^{-\lambda z} \left[H_0 \cos\lambda\ z + M_0 \lambda\ (\cos\lambda\ z - \sin\lambda z\ \right]$$

(6)

where: $\lambda_z = \sqrt{\dfrac{C_h\ b_{rač}}{4EI}}$

For linearly increasing soil reaction coefficient according to [1] and [8] is:

$$y(z) = y_0 A_1 + \frac{\varphi_0}{\alpha} B_1 + \frac{M_0}{\alpha^2 EI} C_1 + \frac{H_0}{\alpha^3 EI} D_1 \tag{7}$$

where: $\alpha = \sqrt[5]{\dfrac{mb_{rač}}{EI}}$

A_1, B_1, C_1, D_1 are influential functions depending on $\bar{z} = \alpha \cdot z$, and H_0, M_0, y_0, φ_0 are initial parameters. Initial parameters H_0 and M_0 are for z=0: M_0=M and H_0=H, while y_0 and φ_0 must be determined from the supporting conditions of lower pile end, where: $z = 1 \quad y_l = 0 \quad \varphi_l = 0$ or $Q_l = 0,\ M_l = \varphi_h C_v I$

By gradual differentiation of expression (6) and (7), one gets expressions for rotational angles of the section $\varphi\ (z)$, bending moments $M\ (z)$ and shear forces $Q\ (z)$.

The second group of calculation methods of transversely loaded piles, which also starts from Winkler's hypothesis, is based on the concept of pile discretization and on replacing the surrounding soil by system of elastic supports (springs, fictive members), on which base were made several programme packages. One of them is the STRESS programme, where from the condition of equal deformation of the ground (Δl_0), and contraction of the fictive members (Δl), one determines cross section areas of the fictive members:

$$A_{f\,i} = \frac{AC_{h\,i}\,l}{E} \tag{9}$$

where: A is belonging area of one pile segment;

$C_{h\,i}$ is soil reaction coefficient for " i "-th fictive member;

l_f is length of fictive member;

E is *modulus* of elasticity for concrete.

In the built frame structure with known dimensions and given load, by calculation of static influences, one gets forces in replacing members, and then the reactive pressures of the soil, bending moments, shear forces, displacements, and rotational angles of the section across the pile axis.

The problem was solved also using the finite element method (FEM) and modern software for engineering analysis of structures - software package ANSYS, where soil was treated as linearly elastic, homogenous continuum.

4. EXAMPLE OF CALCULATION AND ANALYSIS OF OBTAINED RESULTS

In purpose of comparison of described calculation methods, an example taken from [5] was analyzed. This case treats a pile in a shape of steel pipe 3.0m long, and with diameter of D=0.273m, which is loaded by transverse force with maximal intensity of 120 kN. The pile and the surrounding soil were firstly treated by the initial parameters method, and then using software packages STRESS and ANSYS. Obtained results of the calculation are compared with measured data in situ and with results obtained by authors in [5].

In the location of the mentioned pile, the soil consisted of medium compact sand in depth of 6.0m below which was very firm clay. Soil was examined by static penetrometer with capacity of 100kN, and obtained results are given in Fig *1*. During the pile loading the value of horizontal displacement of the pile head was measured, and based on that a diagram “horizontal force-horizontal displacement” was constructed, and shown in Fig. *5*. The calculation procedure which was proposed by authors in [5] represents application of the finite difference method, starting from the assumption that the soil is approximated by Winkler’s soil model. The pile was divided into 100 segments, and the soil was described by the soil reaction coefficient in horizontal direction, whose values were obtained from the expression: $k_h = \frac{E_s}{D}$, where Es is modulus of elasticity of soil, and D is pile diameter.

Value of the modulus of elasticity of soil Es was obtained by multiplying of the penetration resistance of the static penetrometer cone Rp by the non-dimensional coefficient α. The coefficient α has a very broad range - from 1.5 to 10, and values depend on local empirical experiences. According to [3], Bogdanović, based on parallel research and measuring on structures built in Belgrade, recommends α in range of 2.5 to 3 for soil penetration resistance Rp from 0.5 to 1.0 MPa, and for Rp >4.0 MPa recommends even lower values, even to α =1.5. In Russia is most often applied α=3.0, and in G. Britain α =1.5-2.0. These values relate to normally consolidated sands, while for the preconsolidated sands one may adopt α =6.0-10. In the paper [5] authors for α take value 5 and based on it obtain for soil reaction coefficient on the level of terrain surface value of 18 000 kN/m^3, and 55 000 kN/m^3 in the level of the pile top. For these values, using the finite differences method, they obtain horizontal displacement of approximately 31.5 mm at pile head, with applied horizontal force of 120 kN.

For problem solving using the initial parameter method, for calculation width of the pile was $b_{rač}$=1.16 m, moment of inertia of the pile $8.63*10^{-5}$ m^4, and for soil characteristic wad adopted, according to [7], m=6000 kN/m^4. For applied force of 120kN there was obtained horizontal displacement of the pile head of 38.8 mm.

Using the STRESS software, two models were treated: with E_s constant along depth, and with dividing of soil up to the pile top into three layers of 1.0 m depth. In this case, in the first model was taken E_s=9500 kN/m^2, (i.e. k_s=34800kN/m^3) for homogenous soil, and for the second model, i.e., multilayer soil, it was taken: for the first layer - E_{s1}=6500 kN/m^2 (k_s=23800kN/m^3), second layer - E_{s2}=9500 kN/m^2 (k_s=34800kN/m^3), and the third - E_{s3}=12500kN/m^2 (k_s=45 800kN/m^3). For the first model and applied horizontal force of 120kN, obtained horizontal displacement of the pile head was 17.9 mm, and for the second 23.1 mm.

Character of the problems in the given example and method of their solving using FEM and software ANSYS is given in *Table 1*.

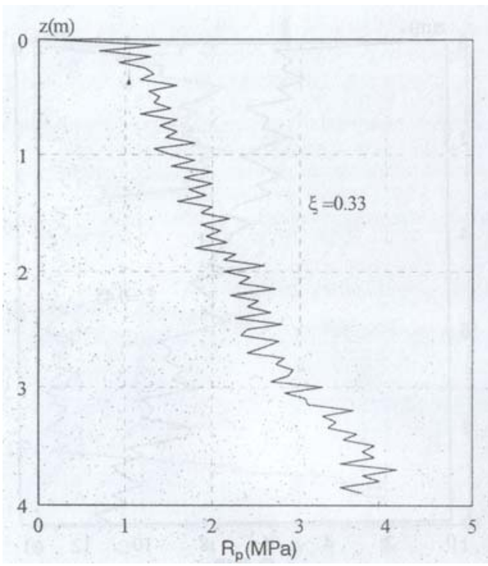

Figure 1. Static penetration test

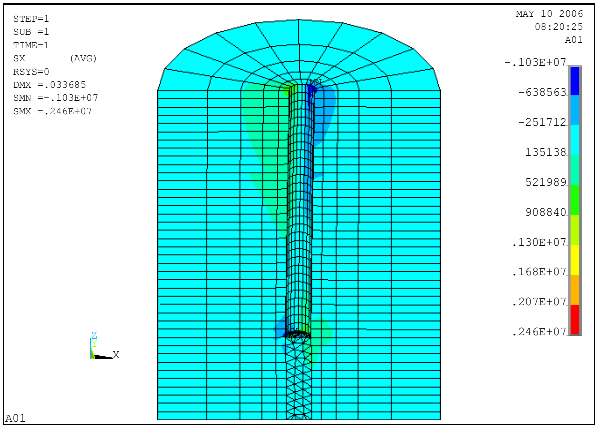

Figure 2. Model A01 – stresses σ_x in soil, linear analysis

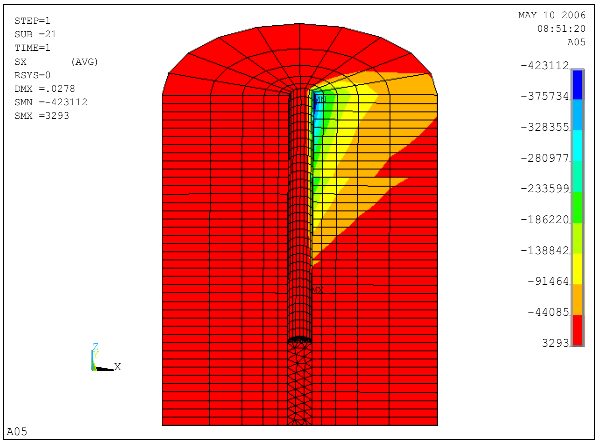

Figure 3. Model A05 - stresses σ_x in soil, contact analysis

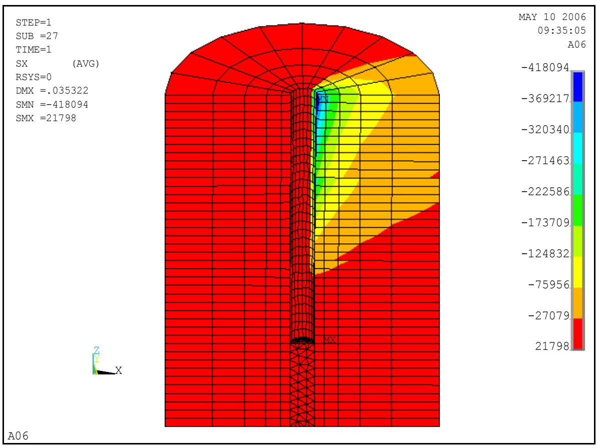

Figure 4. Model A06 - stresses σ_x in soil, contact analysis with excluding of tensioned soil elements

No	PROBLEM	SOLVING METHOD	COMMENT
1	Modeling of soil mass	Modeling applying 3D FE (parabolic hexahedra), cylinder shaped.	Modeling reality: high; soil mass dimensions towards the perimeter could be larger, but do not have significant influence on problem solution
2	Modeling of the cylindrical structure of the pile	Modeling applying 2D FE (parabolic shells).	Modeling reality: high.
3	Boundary conditions for soil	Base of soil mass: all displacements constrained; lateral sides of soil: constrained displacements orthogonally to the soil plane. Symmetry applied.	Modeling reality: high.
4	Boundary conditions for pile	Pile free in the soil mass, i.e. connected only through contact FE. Symmetry applied.	Modeling reality: high.
5	Variable modulus of elasticity of soil vs. depth	Soil modeling in 4 layers by 1.0 m with gradually variable. E_{s1}=6500kN/m2, Es_2=9500kN/m^2 E_{s3}=12500 kN/m^2, E_{s4}=16500 kN/m^2	Modeling reality: satisfying. Uniformness of stress distribution in soil may be increased by finer discretization of soil layers, or by special layer-like FE.
6	Contact of two non-homogenous materials (pile-soil)	Contact analysis – modeling of pile-soil connection by contact FE (type”surface-surface”.	Modeling reality: high.
7	Disability of soil to accept tension stresses	Analysis was done in 2 steps: 1. primary: all soil FE equal in stress transfer; 2. secondary: soil FE that showed in primary analysis stress SX>0 are excluded, by reduction of their modulus of elasticity to ≈0.	Modeling reality: satisfying. Total elimination of tension in soil (if needed) may be achieved by iterative repeating of described procedure.
8	Friction pile-soil	Not included in analysis.	Possible, with precise data of friction coefficient.

Table 1. FEM analysis – problems and solution metodology

Numerous analyses were performed – from the simplest, linear, to the complex, contact non-linear, with excluding of soil elements exposed to tension. Only small number of models and results appeared to be really acceptable, and they are summarized in the *Table 2*, where notes about behaviour of the chosen model are given. The last analyzed model (label: **A06.db**) was the most realistic, considering the obtained value for pile head displacement **(33.7 mm, deviation from experimental value: 6.98%),**

BR	PARAMETER	MODEL A01.db	MODEL A05.db	MODEL A06.db
1	ANALYSIS DOMAIN	LINEAR	CONTACT, (friction coefficient: 0)	CONTACT, (friction coefficient: 0)
3	MODEL DESCRIPTION	See Table 1.	See Table 1. Elements of pile and soil connected by contact elements "surface-surface"	See text. Elements of pile and soil connected by contact elements"surface-surface". For elements with SX≥0 introduced: E=10Pa≈0
4	NODES	13032	14114	14114
5	ELEMENTS	3692	4592	4592
6	C. SYSTEM	X=horizontal, force direction; Y=orthogonal to the symmetry plane; Z=vertical.		
7	BOUNDARY CONDITIONS	Symmetry; soil base: ux=uy=uz=0; lateral sides of soil: ux=uz=0.	Symmetry; soil base: ux=uy=uz=0; lateral sides of soil: ux=uz=0. pile: bottom contour: uz=0.	Symmetry; soil base: ux=uy=uz=0; lateral sides of soil: ux=uz=0. pile: bottom contour: uz=0.
8	PILE HEAD LOAD	FX=2*30=60 kN	FX=2*30=60 kN	FX=2*30=60 kN
9	**SOIL: UX (mm)**	**-9.4/33.2 (top/pile head)**	**-0.9/27 (top/pile head)**	**-7.8/33.7 (top/pile head)**
20	COMMENT	Stress and strain picture expected, except unreal tension stresses in soil. Noticed reactive stress at pile top, opposite to force.	Soil tension region negligible. Stress and strain picture very good, contact pressures correspond very well to SX stresses.	Soil tension region negligible. Stress and strain picture very good, contact pressures correspond very well to SX stresses.
21	CONCLUSION	Model formally acceptable, but insufficiently real due to the tension stresses in soil. Displacement values of the soil in the pile top very close to the experimental.	Model acceptable, and by conception has the best quality. Eventually introduce the friction coefficient. Displacement values of the soil in the pile top very close to the experimental, but not satisfying.	Model acceptable, and by conception has the best quality. Eventually introduce the friction coefficient. Displacement values of the soil in the pile top very close to the experimental, **(deviation: 6.98%), - satisfying. Governing model for this problem.**
22	MODEL VALIDITY	THEORETICAL, PRACTICAL	THEORETICAL, PRACTICAL	THEORETICAL, PRACTICAL

Table 2. Review of theoretically and practically valid models and results of analysis

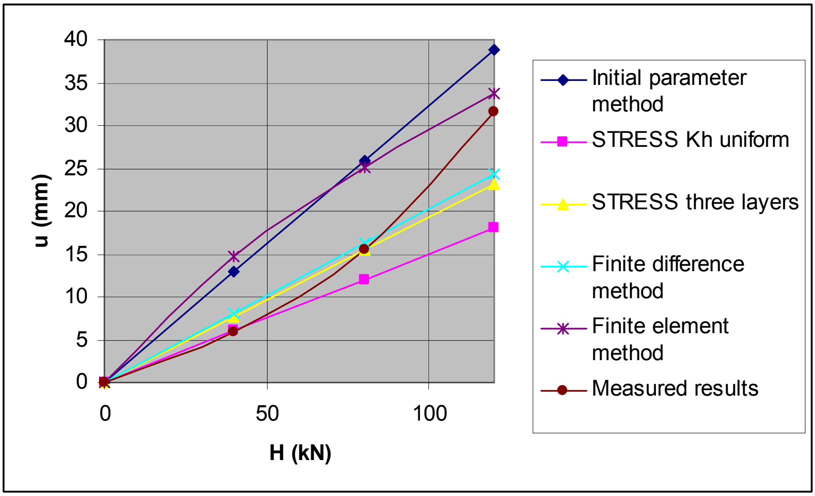

Figure 5. Diagram "horizontal force-horizontal displacement" for applied calculation models

5. CONCLUSION

Quality of the FEM analyses, as well as the rality of the model "pile-soil" is not exhausted by this research. Cited contact analysis could be performed with even greater accuracy with fulfilment of the following conditions: introduction of the pile-soil friction coefficient, variation of the modulus of elasticity of soil as function of load, variation of the modulus of elasticity of soil as function of depth, with greater number of layers. We consider that application of this methodology may have great advantages in regard to the remained treating this problem, with necessary fulfilment of the mentioned conditions. Experiences in this field are also very important due to the sensitivity of the FEM models, whose behaviour and results may vary in (unacceptably) wide range.

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TENTH NATIONAL AND
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RESEARCH OF POSSIBILITY OF APPLICATION OF RECYCLED CONCRETE AS AGGREGATE FOR NEW CONCRETE - PART I

Summary: *Lack of natural aggregate in urban areas and increasing distance between deposits of high-quality natural aggregate and building sites, forced building contactors to analyze possibility of change of natural aggregate with recycled materials (masonry, slag, concrete, etc.). On the other hand, huge amount of old concrete exists in urban areas and it's removal and deposition is a big ecological problem. For that purpose wide experimental investigation was conducted with aim to determine characteristic properties of concrete with recycled concrete aggregate. Obtained results were base for evaluation of possibility of using of these concrete in contemporary civil engineering. Part of research results of other authors, as well as part of own comparative investigation of properties of fresh and hardened concrete with natural and recycled aggregate are briefly presented in this paper. It was concluded that concrete mixtures with recycled aggregate are very similar to concrete mixes with natural aggregate if rules for design and production of this new concrete type are taking into account.*

Key words: recycled concrete *aggregate, fresh concrete, consistency, air content.*

ISTRAŽIVANJE MOGUĆNOSTI PRIMENE RECIKLIRANOG BETONA KAO AGREGATA ZA IZRADU NOVOG BETONA I DEO

Rezime: *Nedostatak prirodnog agregata u urbanim sredinama i sve veće rastojanje između nalazišta kvalitetnog prirodnog agregata i gradilišta prisilili su graditelje na razmatranje mogućnosti zamene prirodnog agregata recikliranim materijalima (građevinska keramika, zgura, beton itd.). Sa druge strane, u urbanim sredinama se često javlja velika količina starog betona čije uklanjanje i deponovanje predstavlja ekološki problem. Zbog toga je sprovedeno obimno eksperimentalno istraživanje sa ciljem da se definišu karakteristična svojstva betona spravljenih sa agregatom od recikliranog betona, radi ocene mogućnosti korišćenja ovih betona u savremenom građevinarstvu. U radu je ukratko prikazan deo rezultata istraživanja drugih autora, kao i deo sopstvenih komparativnih ispitivanja svojstava svežeg betona sa prirodnim i sa recikliranim agregatom. Zaključeno je da se uz poštovanje pravila za projektovanje i spravljanje betona sa recikliranim agregatom mogu dobiti betonske mešavine koje se bitnije na razlikuju od betonskih mešavina sa prirodnim agregatom.*

Ključne reči: agregat od recikliranog betona, sveži beton, konzistencija, uvučeni vazduh.

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1. INTRODUCTORY

Demolishing of old and deteriorated buildings and their substitution with new buildings is frequent phenomenon in urban areas and in the scope of traffic infrastructure. Main reasons for demolishing of existing buildings is change of their purpose, ageing of structures, rearrangement of a city parts, expanding of traffic directions and increasing of traffic load, natural disasters (earthquake, fire, flood) etc. For example, in countries of EEC, forty years ago 50 million tons of concrete was demolished per year and the prediction was that in the beginning of this millennium that quantity will increase three times [1]. In USA the construction waste produced from building demolition alone is estimated to be 123 million tons per year [2]. The most common method of managing this material has been through disposal in landfills. In that way huge deposits of construction waste are created and consequently agriculture land is decreasing and that present real ecological problem because construction waste is potential polluter of human environment. For that reason in developed countries, restricted laws in a form of prohibitions or special taxes for creating of waste areas are bring into practice.

From the other hand, production and application of concrete is rapidly increasing which lead to increasing of consumption of natural aggregate as a largest concrete component. This can be proved by example that two billion tons of aggregate are produced each year in the United States . Production is expected to increase to more than 2.5 billion tons per year by the year 2020 [2]. This situation led to question about the availability of natural aggregates and where we will find new aggregate sources. Many European countries have placed a tax on the use of virgin aggregates.

The solution of these problems state agencies and the aggregate industry find in recycling concrete debris as an alternative aggregate.

Last twenty years possibilities of use of recycled materials (bricks and concrete) for production of new building materials are intensively investigated. As a result of wide research last years two main directions of use of recycled concrete are adopted:

- as a incoherent crushed material for different types of embankments and sub-bases and
- as a fractioned aggregate for production of a new concrete.

In our country there are no available data about investigation of new concrete with recycled concrete aggregate (RAC) nor recommendations for it's application.

That was the reason to carry out own experimental research to explore possibilities of use of recycled concrete as a aggregate for production of new concrete.

2. RECOMMENDATIONS FOR PRODUCTION AND BASIC PROPERTIES OF CONCRETE WITH RECYCLED AGGREGATE

In this chapter recommendations for production and the most important properties of fresh and hardened concrete with recycled aggregate are briefly presented. Presented data are results of comparative testing of properties of ordinary concrete and concrete with recycled aggregate of other researchers.

Recycled aggregate compared to natural aggregate has following properties:

- increased water absorption,
- decreased bulk density,
- decreased specific gravity,
- increased abrasion,
- increased crushability,
- increased quantity of dust particles,
- increased quantity of organic impurities (if concrete is mixed with earth during building demolition) and
- possible content of chemically harmful substances (depends on exploitation conditions in building from which by demolition and crushing recycled aggregate is obtained).

It is recommended to use aggregate from recycled concrete only as a course aggregate if quality concrete have to be produced. In USA it is allowed partial substitution of natural fine aggregate with recycled fine aggregate up to 20%.

Technology of production of concrete with recycled aggregate is different from the production procedure for ordinary concrete. One possibility to provide design consistency of fresh concrete is to previously saturate recycled aggregate to the condition "water saturated surface dry".

Available testing results of concrete with recycled aggregate vary in wide limits, sometimes are even opposite, but generally conclusions about properties of concrete with recycled coarse aggregate compared to ordinary concrete with natural aggregate, are [1]:

- Increased drying shrinkage (up to 40%),
- Increased creep (up to 50%),
- Water absorption depends on differences between water-cement ratio of new and old concrete which was used for recycling (there are no differences if new concrete has bigger water-cement ratio from recycled concrete),
- Decreased compressive strength (5-30%),
- Decreased splitting tensile strength (0-10%),
- Decreased flexural strength (0-10%),
- Decreased modulus of elasticity (10-30%),
- Same or increased frost resistance.

3. OWN EXPERIMENTAL INVESTIGATION

With this investigation was planning to compare basic properties of referent concrete (concrete with natural aggregate) and concrete with recycled aggregate.

Experimental part of research program included three types of concrete mixtures. Tested concrete mixture composition was determined in accordance to the following conditions:

- same cement quantity,
- same consistency after 30 min,
- same max. grain size (32mm),

- same granulometric curve of aggregate fractions,
- same kind and quantity of fine aggregate,
- variable kind and quantity of coarse aggregate.

Kind and quantity of coarse aggregate were varied in the following way:

- the first concrete mix has 100% natural river coarse aggregate (R0)
- the second concrete mix has 50% natural river coarse aggregate and 50% recycled coarse aggregate (R50)
- the third concrete mix has 100% recycled coarse aggregate (R100)

Adopted beginning conditions for determination of mixture compositions make possible analyze of tested concrete properties in relation to used quantity of coarse recycled aggregate (0%, 50% i 100%).

Following properties were tested on fresh concrete:

- consistency (slump test) immediately after mixing and 30 min after mixing,
- bulk density and
- air entrained quantity.

On hardened concrete samples following properties were tested:

- bulk density,
- water absorption (age 28 days),
- water impermeability (age 28 days),
- abrasion resistance (age 28 days),
- compressive strength (age 2, 7 and 28 days),
- splitting tensile strength (age 28 days),
- flexural strength (age 28 days),
- modulus of elasticity (age 28 days),
- shrinkage (age 3, 4, 7, 14, 21 and 28 days),
- bond between reinforcement and concrete (age 28 days).

For the testing of listed properties of hardened concrete 99 samples were formed.

Component materials for concrete mixtures were:

- Portland - composite cement CEM II/A-M(S-L) 42.5R, (Lafarge -BFC)
- Fine aggregate (river aggregate, separation Luka Leget, fraction 0/4mm)
- Two kinds of coarse aggregate:
  - river aggregate, separation Luka Leget, fractions 4/8, 8/16 and 16/31.5mm,
  - aggregate form recycled concrete (fractions 4/8, 8/16 and 16/31.5mm)
- potable water.

Aggregate from recycled concrete was produced by crushing of „old “ concrete with class C30/37 and C40/50. As a raw material for crushing were used concrete cubes for compressive strength testing (Fig. 1) and one precast reinforced concrete column, which had inappropriate dimensions (Fig. 2). First was carried out primary crushing with pneumatic hammer (Fig. 2) and than secondary crushing in rotating crusher (Fig. 3 and 4). Obtained material after primary crushing is shown in Fig. 5, and after secondary crushing in Fig. 6. After completion of crushing process concrete particles were separated in standard fractions of course aggregate (4-8mm, 8-16mm and 16-31.5mm). Appearance of aggregate produced from recycled concrete, after sieving is illustrated in Fig. 7.

Fig. 1 –Concrete samples (cubes)

Fig. 2 – Precast RC column

Fig. 3 - Rotation crusher (detail)

Fig. 4 – Rotation crusher

Fig. 5 – Material after primary crushing

Fig. 6 – Material after secondary crushing

Fig. 7 – Fractions of recycled concrete aggregate

Component materials were tested before mixing of concrete. Regarding test results it was concluded that tested cement and river aggregate satisfy prescribed requirements of quality. The results of testing of recycled concrete aggregate are shown in Table 1:

Tab. 1 - Results of testing of recycled concrete aggregate

Tested property	Measured value	Fraction			quality requirement
		4/8	8/16	16/32	
Crushing resistance (in cylinder)	mass loss (%)	18.3	26.7	30.7	< 30
Freezing resistance test	mass loss (%)	2	1.4	1.0	< 12
Chemical testing (mortar part of recycled aggregate)	chloride content	0	0	0	< 0.1
	sulfate content	in traces	in traces	in traces	< 1
	pH	9.85	9.85	9.85	-
Content of weak grains	(%)	0	3.7	7.1	< 3 (4)
Crushing resistance (machine "Los Angeles")	mass loss (%)	29.6	33.7	34.0	< 30
Water absorption after 30 minutes	(%)	4.59	2.87	2.44	-
Fines content	(%)	0.45	0.23	0.36	< 1

3.1. Concrete mixtures composition

Compositions of concrete mixtures were chosen on the basis of following conditions:

- cement content 350kg/m³,
- water content according to required consistency (slump test, $\Delta h=10\pm2cm$ after 30 minutes from mixing of concrete),
- aggregate content from the amount of absolute volume of component materials in 1m³ of concrete, ($\gamma_{s,a}$=2670 kg/m³, $\gamma_{s,ar}$ =2500 kg/m³ and $\gamma_{s,c}$ =3060 kg/m³)
- Granulometric content of aggregate mixture according to the curve of Fuler,
- air content Δ p=1%.

Designed compositions of all tested concrete mixtures are shown in Table 2 and quantity of each aggregate fraction in Table 3:

Tab. 2 - Design quantities of component materials

Concrete mixture	Cement content (kg/m³)	Water content (kg/m³)	Aggregate content (kg/m³)	Additional water content* (kg/m³)	Bulk density (kg/m³)
R0	350	180	1857	0	2387
R50	350	180	1816	19	2365
R100	350	180	1776	37	2343

* - For preparing of concrete mixtures dry recycled aggregate was used and to achieve required consistency additional water quantity was calculated (quantity of water which recycled aggregate absorb in first 30 minutes after mixing of concrete). For determination of additional water quantity results of water absorption of recycled aggregate after 30 minutes were used.

Tab. 3 - Design quantities of aggregate fractions

Concrete mixture	Content of natural river aggregate (kg/m³)				Content of recycled aggregate (kg/m³)		
	0/4	4/8	8/16	16/32	4/8	8/16	16/32
R0	612	298	390	556	0	0	0
R50	600	145	191	272	118	136	354
R100	586	0	0	0	231	266	693

3.2. Results of testing of fresh concrete

To obtain real data about influence of recycled aggregate on properties of fresh and hardened concrete, mixing time and procedure of compacting were equal for all tested concrete types. Mixing time was 90 seconds and compacting was conducted on a vibrating table for a time of 30 seconds.

Results of testing of consistency, air content and bulk density are presented in Table 4. In the same table calculated real quantities of component materials, are shown also.

Tab. 4 - Testing results of fresh concrete

Concrete mixture	$m_{c,stv}$ (kg/m³)	$m_{v,stv}$ (kg/m³)	$m_{a,stv}$ (kg/m³)	m_v/m_c	m_a/m_c	Δh_1 (cm)	Δh_2 (cm)	Δp (%)	$\gamma_{b,stv}$ (kg/m³)
R0	352	181	1866	0.514	5.306	16	10	1.5	2399
R50	352	200	1826	0.5683	5.188	14.5	8.5	1.4	2378
R100	348	216	1765	0.62	5.074	11	9	1.3	2329

Δh_1 - measured slump immediately after mixing

Δh_2 - measured slump after 30 minutes

Δp - measured air content

Slump after mixing

Slump after 30 minutes

Fig. 8 – Consistency (slump test)

4. ANALYSE OF THE TESTING RESULTS

Differences in water content, which is necessary to achieve the same consistency after 30 minutes, are shown in table 5.

Tab. 5 - *Differences in water content between concrete mixtures R0, R50 and R100*

$m_{v,R0}$ (kg/m³)	$m_{v,R50}$ (kg/m³)	$m_{v,R100}$ (kg/m³)	$m_{v,R50}-m_{v,R0}$ (kg/m³)	$(m_{v,R50}-m_{v,R0})/m_{v,R0}$ (%)	$m_{v,R100}-m_{v,R0}$ (kg/m³)	$(m_{v,R100}-m_{v,R0})/m_{v,R0}$ (%)
181	200	216	19	10.55	35	19.33

It was concluded that for preparing of concrete mixture R50 is necessary 19 kg/m³ or 10.55% of water more than for preparing of concrete mixture R0. The reason is water absorption of recycled aggregate.

For preparing of concrete mixture R100 it is necessary 35 kg/m³ or 19.33 % water more than for preparing of referent concrete mixture R0.

By comparing of air content (Δp) in concrete mixtures R0, R50 and R100 (Tab. 4) it was concluded that differences are insignificant.

Values of measured bulk densities of tested concrete types are presented in Fig. 9.

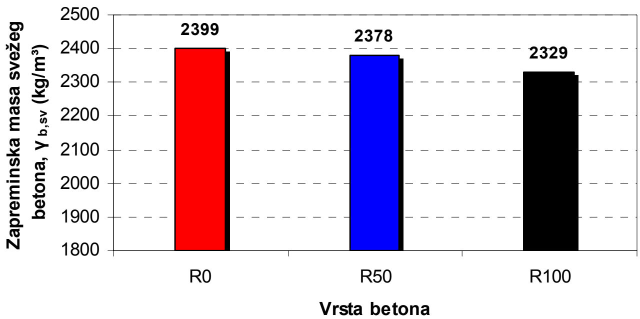

Fig. 9 - Bulk densities of fresh concrete

Differences between bulk densities of concrete mixtures R0, R50 and R100 are shown in table 6.

Tab. 6 - Differences between bulk densities of concrete mixtures R0, R50 and R100

$\gamma_{sv,R0}$ (kg/m³)	$\gamma_{sv,R50}$ (kg/m³)	$\gamma_{sv,R100}$ (kg/m³)	$\gamma_{sv,R50}-\gamma_{sv,R0}$ (kg/m³)	$(\gamma_{sv,R50}-\gamma_{sv,R0})/\gamma_{sv,R0}$ (%)	$\gamma_{sv,R100}-\gamma_{sv,R0}$ (kg/m³)	$(\gamma_{sv,R100}-\gamma_{sv,R0})/\gamma_{sv,R0}$ (%)
2399	2378	2329	21	0.88	70	2.92

By analyze of bulk density values, shown in table 6, it is concluded that bulk density of concrete with natural aggregate is max. 3% bigger than bulk density of concrete with recycled aggregate.

5. CONCLUSION

Increasing consumption of aggregate and restrictions in exploitation of natural raw materials for production of aggregate, led to application of alternative materials. These materials are often industrial secondary products or building waste and their disposal is serious ecological problem. One of such materials is concrete waste which is formed by demolition of buildings. Last twenty years in some countries of Europe, North America and in Japan, use of aggregate from recycled concrete is stimulated by state institutions. In these countries national standards for this kind of aggregate are adopted and as a result of intensive investigation, recommendations for preparing and application of new concrete with aggregate from recycled concrete, are published. Possibilities of application of aggregate from recycled concrete are not explored in our country, and that's why authors of this paper try to start application of recycled aggregate with own experimental investigation.

On the basis of comparative analyze of testing results of properties of fresh concrete with natural coarse aggregate, combination of coarse aggregate (natural and recycled) and with recycled coarse aggregate, it was concluded:

- Kind of course aggregate has not influence on air content;
- With increasing of quantity of recycled aggregate bulk density of concrete is decreased;
- The way of preparing of recycled aggregate for concrete mixtures influents on concrete consistency. In the case of use of "water saturated - surface dry" recycled aggregate consistency of concrete with natural and with recycled aggregate will be almost the same. If dry recycled aggregate is used and additional water quantity, the same consistency could be achieved after a definite time.
- Recycled aggregate has influence on mobility of concrete mixture, because of decreasing of free water content in cement paste due to water absorption. By adding of certain water quantity to concrete mixture, the same consistency as for concrete with natural aggregate could be provided. Additional water quantity depends on time when same consistency have to be achieved and is determined with water quantity which recycled aggregate absorb for the same period of time.

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TENTH NATIONAL AND FOURTH INTERNATIONAL SCIENTIFIC MEETING

iNDiS 2006

NOVI SAD, NOVEMBER 22-24,2006.

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RESEARCH OF POSSIBILITY OF APPLICATION OF RECYCLED CONCRETE AS AGGREGATE FOR NEW CONCRETE – PART II

Summary: *Lack of natural aggregate in urban areas and increasing distance between deposits of high-quality natural aggregate and building sites, forced building contactors to analyze possibility of change of natural aggregate with recycled materials (masonry, slag, concrete, etc.). On the other hand, huge amount of old concrete exists in urban areas and it's removal and deposition is a big ecological problem. For that purpose wide experimental investigation was conducted with aim to determine characteristic properties of concrete with recycled concrete aggregate. Part of own research results related to properties of hardened concrete with natural and recycled aggregate with certain quality, are briefly presented in this paper. Obtained testing results showed that recycled aggregate concretes have satisfactory performances that are not differ from properties of ordinary concrete significantly. This paper, altogether with previous paper "Research of possibility of application of recycled concrete as aggregate for new concrete – part I", is unique totality .*

Key words: recycled concrete *aggregate, shrinkage, compressive strength, tensile strength.*

ISTRAŽIVANJE MOGUĆNOSTI PRIMENE RECIKLIRANOG BETONA KAO AGREGATA ZA IZRADU NOVOG BETONA II DEO

Rezime: *Nedostatak prirodnog agregata u urbanim sredinama i sve veće rastojanje između nalazišta kvalitetnog prirodnog agregata i gradilišta prisilili su graditelje na razmatranje mogućnosti zamene prirodnog agregata recikliranim materijalima (građevinska keramika, zgura, beton itd.). Sa druge strane, u urbanim sredinama se često javlja velika količina starog betona čije uklanjanje i deponovanje predstavlja ekološki problem. Zbog toga je sprovedeno obimno eksperimentalno istraživanje sa ciljem da se definišu karakteristična svojstva betona spravljenih sa agregatom od recikliranog betona, radi ocene mogućnosti korišćenja ovih betona u savremenom građevinarstvu. U radu je ukratko prikazan deo sopstvenih komparativnih ispitivanja svojstava očvrslog betona sa prirodnim i sa recikliranim agregatom. Rezultati ispitivanja su pokazali da betoni spravljeni sa recikliranim agregatom određenog kvaliteta imaju zadovoljavajuće performanse koje se bitnije ne razlikuju od performansi betona sa prirodnim agregatom. Ovaj rad zajedno sa radom "Istraživanje mogućnosti primene recikliranog betona kao agregata za izradu novog betona- I deo" predstavlja jedinstvenu celinu.*

Ključne reči: agregat od recikliranog betona, skupljanje, čvrstoća pri pritisku, čvrstoća na zatezanje.

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1. INTRODUCTION

Testing results, comparative analyze and conclusions from own experimental investigation of properties of hardened concretes with natural aggregate (R0), with combined aggregate (R50) and with recycled aggregate (R100), are presented in this paper. Concrete mixtures compositions, results of testing of fresh concrete, their analyze and final remarks are presented in the paper „Research of possibility of application of recycled concrete as aggregate for new concrete – part I“ written by same authors.

According to the program of experimental investigation, following properties of hardened concrete, are tested:

- $\gamma_{b,o\check{c}}$ - Bulk density,
- u_v - Water absorption (age 28 days),
- h - Waterproofness according to DIN 1048, (age 28 days),
- Wear resistance (age 28 days),
- f_p - Compressive strength (age 2, 7 and 28 days),
- f_{zc}- Splitting strength (age 28 days),
- f_s - Flexural strength (age 28 days),
- E - Modulus of elasticity (age 28 days),
- ε_s - Drying shrinkage (age 3, 4, 7, 14, 21 and 28 days),
- f_{at} - Bond between reinforcement and concrete (ribbed and mild reinforcement),

In the comparative analyze of testing results, results of properties of concrete with natural aggregate (R0) were adopted as referent.

With tested concrete types (R0, R50 and R100) reinforced concrete beams were prepared for flexural testing. Beams were executed with length of 3.0m and with rectangular cross section 15/25cm. In lower zone the beams were reinforced with ribbed reinforcement 3R∅12, in upper zone with 2R∅10 and as a stirrups ∅6/20 were used. At age of 28 days beams were subjected to load testing (bending with concentrated force in the middle of the span). Load was increased till collapse of the beams. Results of this testing help authors to study behavior of concrete with recycled aggregate in structural elements, but they are not presented in this paper due to limited space.

2. RESULTS OF TESTING OF HARDENED CONCRETE

Results of testing of compressive strength of concretes R0, R50 and R100 at age of 2, 7 and 28 days, are presented in Table 1. For testing 15cm cubes were used.

Tab. 1 – *Compressive strength of concrete R0, R50 i R100, at age of 2, 7 and 28 days*

Vrsta betona	$\gamma_{b,o\check{c}v}$ (kg/m³)	$f_{p,2}$ (MPa)	$f_{p,7}$ (MPa)	f_{p28} (MPa)	σ (MPa)
R0	2424	27.55	35.23	43.44	1.57691
R50	2379	25.74	37.14	45.22	1.2089
R100	2332	25.48	37.05	45.66	3.50163

Measured values of drying shrinkage of concretes R0, R50 and R100, for characteristic age of concrete, are shown in Table 2. Presented data are mean values from 3 measuring results. This testing was performed on prismatic samples 10x10x40cm and for measuring of dilatation changes extensometer with base of 25cm was used (Fig. 1).

Tab. 2 – *Results of testing of drying shrinkage of concrete R0, R50 and R100*

Concrete type	$\varepsilon_{s,4}$ **(mm/m)**	$\varepsilon_{s,7}$ **(mm/m)**	$\varepsilon_{s,14}$ **(mm/m)**	$\varepsilon_{s,21}$ **(mm/m)**	$\varepsilon_{s,28}$ **(mm/m)**
R0	0.0173	0.124	0.2027	0.2773	0.3387
R50	0.0360	0.086	0.1760	0.2540	0.3060
R100	0.0907	0.204	0.2507	0.3347	0.4067

Fig. 1 – Measuring of drying shrinkage

Results of testing of other properties of hardened concrete at age of 28 days are presented at Table 3.

Tab. 3 – *Results of testing of properties of hardened concrete at age of 28 days*

Concrete type	**R0**	**R50**	**R100**
Water absorption, (%)	5.61	6.87	8.05
Waterproofnees , (mm)	26.27	17.67	34.67
Splitting strength, (MPa)	2.66	3.20	2.78
Flexural strength, (MPa)	5.4	5.7	4.2
Wear resistance, (cm^3/ 50 cm)	13.40	15.58	17.18
Bond between mild reinforcement and concrete, MPa	6.48	5.87	6.76
Bond between ribbed reinforcement and concrete, MPa	8.22	7.50	7.75

Waterproofness of concrete R0, R50 and R100 was tested on cubic samples 20cm, (Fig. 2). Splitting strength of concrete was tested on cubes 15cm (Fig. 3) and flexural strength on prismatic samples 10x10x40cm.

Fig. 2 – Testing of waterproofness of concrete R50

Fig. 3 – Testing of splitting strength of concrete (concrete R50)

For testing of bond between reinforcement and concrete R0, R50 and R100, cylindrical samples with diameter 10cm and height 15cm, with embeded ribbed and mild reinforcement (R∅12mm and ∅12mm). Length of embeded part of reinforcement was 15cm. For testing of bond, axial tension procedure and tearing device were used (Fig. 4).

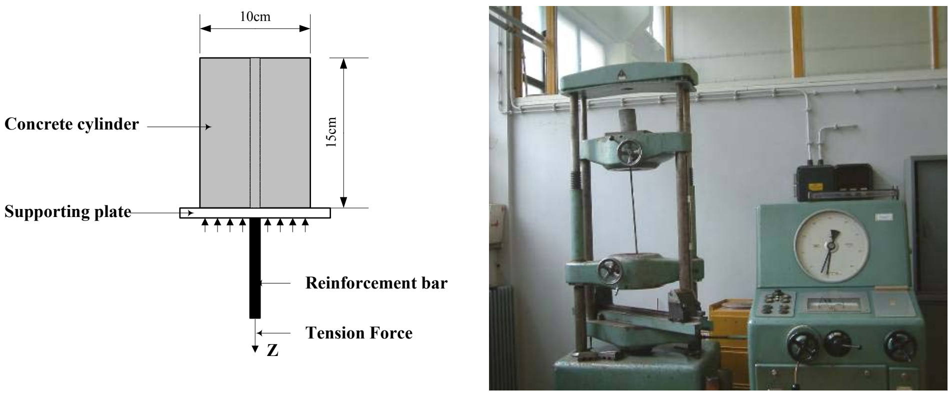

Fig. 4 – Testing of bond between concrete and reinforcement

3. ANALYZE OF TESTING RESULTS

To describe change of **concrete compressive strength** in time, fraction function (1) was adopted:

$$fp(t) = \frac{a \times t}{t + b} \qquad (1)$$

Calculated parameters of this functional relation ("a" and "b") for concrete R0, R50 and R100, altogether with correlation coefficient ("r"), are presented in Table 4. Values of correlation coefficients point to the fact that chosen fraction function presents realistic the gain of compressive strength through time for all tested concrete types.

Tab. 4 – Parameters of functional relations between compressive strength and age of concrete

Concrete type	a	b	r
R0	44.242	1.320	0.976
R50	47.556	1.761	0.997
R100	48.116	1.856	0.996

Testing results for concrete compressive strength at age of 2, 7 and 28 days (Table 1) and established functional relations fp(t) for concrete R0, R50 and R100 are illustrated in Fig. 5.

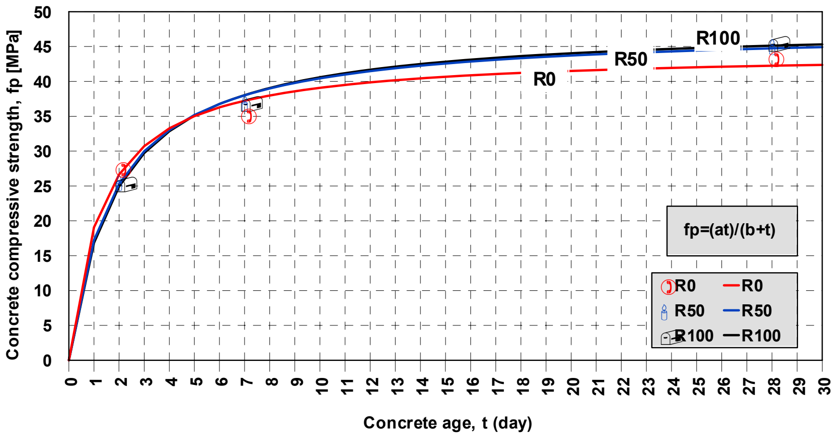

Fig. 5 – Concrete compressive strength gain through time

By analysis of the concrete compressive strength values, it was concluded:

– all three concretes have bigger 28-day compressive strength than 40MPa.

– concrete "R50" and "R100" have bigger 28-day compressive strengths than concrete "R0".

– concrete "R50" and "R100" have approximately the same growth of compressive strength.

– differences between compressive strengths of concrete R0, R50 and R100 are negligible for the same concrete age.

To find out are differences between obtained compressive strengths of concrete R0, R50 and R100 at age of 28 days, accidental or not, difference between their mean values was statistically tested. For that purpose pairs of corresponding 28-day strength were formed (R0-R50, R0-R100 and R50-R100). Tested value is defined with expression:

$$t_0 = \frac{x_{sr,1} - x_{sr,2}}{s \cdot \sqrt{\frac{1}{n_1} + \frac{1}{n_2}}} \tag{2}$$

$$s = \sqrt{\frac{(n_1 - 1) \times \sigma_1^2 + (n_2 - 1) \times \sigma_2^2}{n_1 + n_2 - 2}} \tag{3}$$

$$\text{criteria:} \quad t_0 < t_\alpha \tag{4}$$

Used signs have following meanings:

t_0	quantile of Student distribution for No. of freedom degree $\nu=n_1+n_2-2$	t_α	critical value of Student distribution for No. of freedom degree $\nu=n_1+n_2-2$
$x_{sr,1}$	average (I set)	σ_1	standard deviation (I set)
$x_{sr,2}$	average (II set)	σ_1	standard deviation (II set)
n_1	No. of testing results (I set)	n_2	No. of testing results (II set)

Results of this statistical test are shown in Table 5.

Tab. 5 – *Testing of difference significance for compressive strengths*

Testing pairs	n_1	n_2	**s**	t_0	t_α, **for** α**=0.05**
(R0 and R50)	6	6	1.406523	2.189924	2.2281
(R0 and R100)	6	6	2.7163	1.417718	
(R50 and R100)	6	6	2.61943	0.29425	

On the basis of results presented in Table 5 and criterion (4), it was stated that differences between tested compressive strengths of concrete R0, R50 and R100 are accidental (all results belong to the same set of results). This conclusion led to the fact that coarse aggregate kind didn't influence on concrete compressive strength value.

Change of **shrinkage deformation** through time, for all concrete types, was described with a function in the form of polynomial function:

$$\varepsilon_s = a + b \times t + c \times t^2 \tag{5}$$

Calculated parameters of this polynomial function, for tested concrete types, are presented in Table 6.

Tab. 6 – *Parameters of relation between shrinkage deformation and concrete age*

Concrete type	**a**	**b**	**C**	**r**
R0	-0.06239	0.02464	-3.767×10^{-4}	0.98929
R50	-0.0467	0.01982	-2.595×10^{-4}	0.99696

R100	-0.0306	0.0281	-4.598×10^{-4}	0.93702

Measured values of drying shrinkage and established functions (ε_s-t) are shown in Fig. 5.

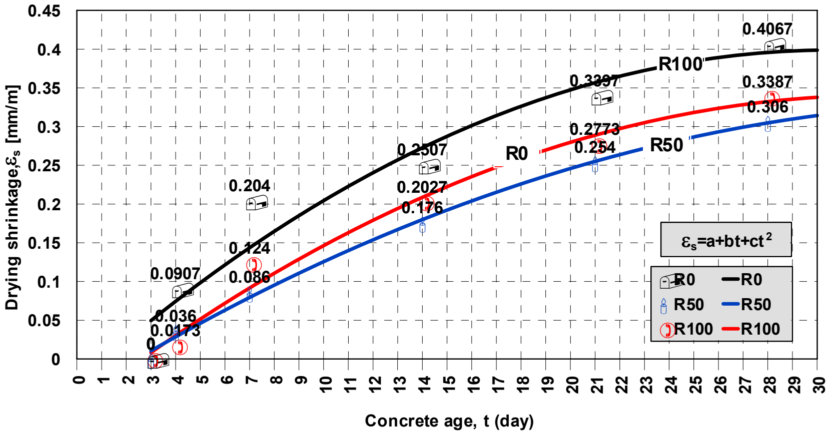

Fig. 6 – Drying shrinkage of concrete R0, R50 and R100

By analysis of 28-day drying shrinkage values for concrete R0, R50 and R100, it was concluded :

- The lowest shrinkage value is registered for concrete R50 (0.31mm/m), and the highest for concrete R100 (0.41mm/m).
- Drying shrinkage of concrete R100 is 20% higher than shrinkage of concrete R0.
- Differences between 28-day shrinkage of concrete R0 and R50 is less than 10%.
- Adopted quadratic functions fit well to time dependent change of drying shrinkage.

Values of **water absorption** of tested concrete types, are presented in Fig. 7.

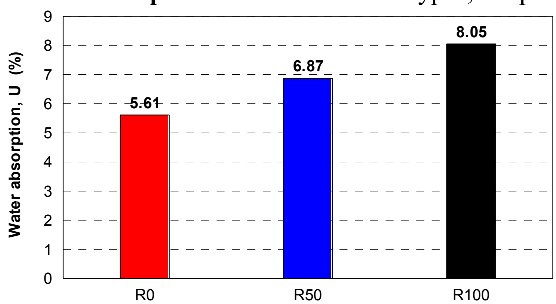

Fig. 7 - Values of water absorption of concrete R0, R50 and R100

By analysis of water absorption values (shown in Fig. 7), it was concluded:

- The lowest water absorption was registered in concrete R0 and the highest in concrete R100.
- Concrete R50 has 22% higher absorption, while concrete R100 has 44% higher absorption than referent concrete R0.

After testing of concrete **waterproofing**, penetration depths were measured for concrete R0, R50 and R100. Registered values are shown in Fig. 8.

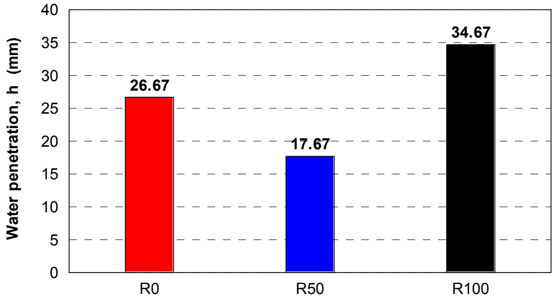

Fig. 8 - Values of water penetration depth

By analysis of water penetration depths, it was concluded:

– The lowest water penetration depth was registered for concrete R50 and the highest for concrete R100.

– There is practically no differences in waterproofness between tested concretes because they all satisfy prescribed condition of waterproofness according to standard DIN 1048.

Results of testing of 28-day **splitting strength** graphically are presented in Fig. 9.

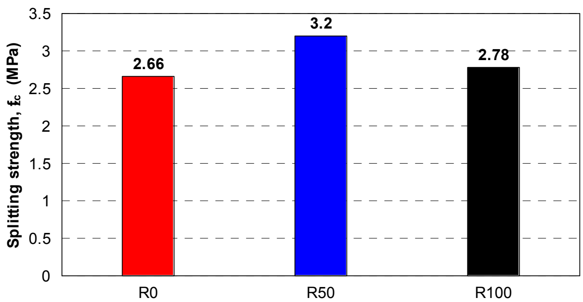

Fig. 9 - Values of 28-day splitting strength of concrete R0, R50 and R100

By analysis of presented values of splitting strengths, it was concluded:

– Concrete R50 has the highest value of splitting strength (3.2MPa), while concretes R0 and R100 have approximately equal splitting strengths at age of 28 day (cca 2.7MPa).

Values of **flexural strength** of tested concretes are shown in Fig. 10.

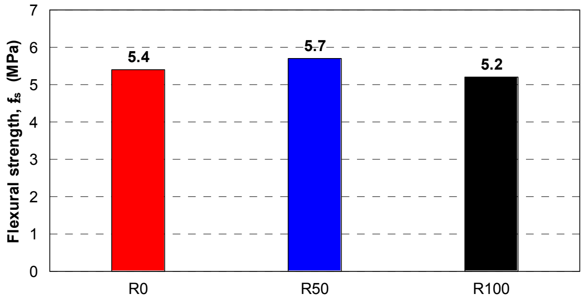

Fig. 10 - Values of 28-day flexural strength of concrete R0, R50 and R100

After comparison of obtained flexural strengths, it was concluded:

- The highest flexural strength has concrete R50 (5.7MPa). This value is only 5% higher than for concrete R0.
- Concretes R0 and R100 have approximately equal flexural strengths at age of 28 day (cca 5.3MPa).

Testing results of **wear resistance** of concrete R0, R50 and R100 are shown in Fig. 11.

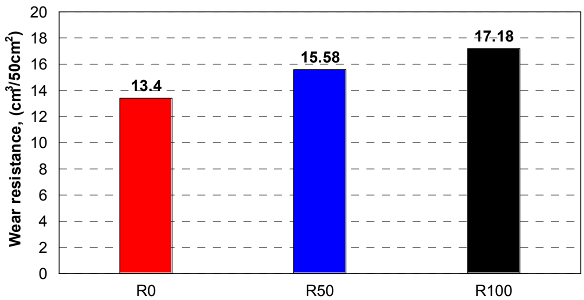

Fig. 11 – Testing results of wear resistance for concrete R0, R50 and R100

By comparison of results of testing of wear resistance, it was concluded that the highest material loss, due to wear, has concrete R100 and the lowest concrete R0.

Results of testing of **bond** between mild and ribbed reinforcement and concrete R0, R50 and R100, are presented in Fig. 12. By analysis of obtained values, it was concluded:

- Difference between lowest and highest bond value for both reinforcement types is about 10%.
- Bond between tested concretes and ribbed reinforcement is higher at least 15% than bond between tested concretes and mild reinforcement.

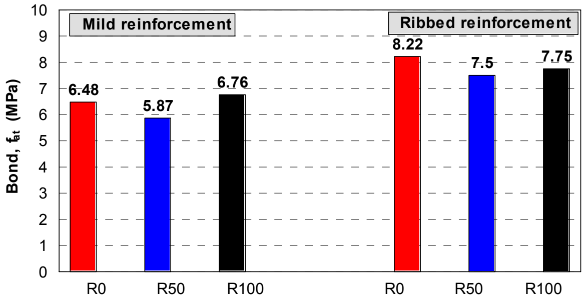

Fig. 12 – Bond between reinforcement and concrete R0, R50 and R100

4. CONCLUSION

On the basis of presented testing results and comparative analysis of properties of hardened concrete with natural coarse aggregate, concrete with coarse recycled concrete aggregate and with combined natural and recycled coarse aggregate, it was concluded:

- Concrete compressive strength depends mainly from the quality of recycled aggregate. If for production of concrete is used quality aggregate which is obtained by crushing of higher strength class concretes, in that case recycled aggregate will not influence on decrease of compressive strength, disregarding quantity of replaced natural coarse aggregate with recycled.
- Quantity of recycled aggregate has influence on water absorption. With increasing of recycled aggregate participation, quantity of absorbed water is proportionally increased.
- Concretes with recycled aggregate could achieve property of waterproofness. On concrete waterproofness, porosity of cement matrix in new concrete and porosity of cement matrix of recycled concrete, have influence. If recycled aggregate is produces from concrete with low porosity, waterproofness of new concrete depends on aggregate granulometric composition and achieved structure of new cement matrix.
- Kind of used aggregate (natural or recycled) has not significant influence on splitting and flexural concrete strength.
- Wear resistance of concrete depends on quantity of recycled aggregate. With increasing of recycled aggregate content, concrete wear resistance decreases due to the increased quantity of hardened cement paste, which wears easier than grains of natural aggregate.
- Bond between recycled aggregate concrete and reinforcement is not significantly influenced by recycled concrete aggregate because bond is realized through new cement paste.

Performances of recycled aggregate concrete could be adopted as satisfactory ones, which mainly depends on compressive strength of recycled concrete, but also on considering all specifications related to mixture composition design and production of this concrete type.

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TENTH NATIONAL AND
FOURTH INTERNATIONAL
SCIENTIFIC MEETING
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NOVI SAD, NOVEMBER 22-24,2006.

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NONLINEAR STATIC ANALYSIS OF FIVE STORY FRAME ON ELASTIC GROUND

Summary: *This paper analyzes a five story frame on an elastic ground by applying nonlinear static analysis. The variation of the structure foundation beam stiffness and the variation of the embedment index were examined. Sequence and positioning of plastic hinge formations were analyzed, as well as the total base shear change. The results were compared to that of a braced five story structure of the same characteristics.*

Key words: *Nonlinear static analysis, plastic hinge, elastic ground, foundation girder*

NELINEARNA STATIČKA ANALIZA PETOSPRATNE OKVIRNE KONSTRUKCIJE NA ELASTIČNOJ PODLOZI

Rezime: *U radu je analizirana petospratna okvirna konstrukcija na elastičnoj podlozi primenom nelinearne statičke analize. Kod konstrukcije varirana je krutost temeljnog nosača, a kod podloge variran je koeficijent posteljice. Analiziran je redosled i mesto formiranja plastičnih zglobova kao i promena ukupne seizmičke sile. Dobijeni rezultati su poređeni sa uklještenom petospratnom konstrukcijom istih karakteristika.*

Ključne reči: *Nelinearna statička analiza, plastični zglob, elastična podloga, temeljni nosač.*

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1. INTRODUCTION

In numerical analysis of structures under seismic action different methods can be applied. For regular and symmetrical structures the regulations allow for a calculation of the maximum response values by means of spectral analysis methods. This kind of approach yields satisfactory results in minor seismic action which do not lead to more significant structure damage. In case of stronger ground motions that do result in significant damage of structure elements, actual structure behavior can be simulated more accurately through nonlinear analysis. For tracking the change in structure response in time, the direct numerical integration method is used.

This paper exhibits the analysis of five story structure through nonlinear static pushover analysis.

Common analysis of structures under seismic action includes an absolutely stiff ground and a disregard for the interaction of the soil and the structure. This paper performs a comparative analysis of a five story structure on a non-deformable as well as on an elastic ground represented by the Winkler model.

2. NONLINEAR STATIC PUSHOVER ANALYSIS

When an earthquake takes place, depending on its strength, it causes nonlinear behavior in specific structure elements, which can be analyzed through nonlinear static pushover analysis. The influence of geometrical nonlinearity is introduced in the calculation through the P-Δ effect, and the material nonlinearity is introduced in the formation of plastic hinges.

The structure model load stems from the real vertical load and the horizontal forces in the story levels. The distribution of these forces is linear (triangular), which approximately corresponds to the first specific type of structure oscillation. With a constant vertical load, the horizontal forces are gradually increased and the structure response is monitored. Lateral loads are increased up to the structure collapse point. Basic calculation process control is set after reaching the limit of the maximum displacement of the structure's specific point (horizontal displacement of the frame top), with the total structure unloading method applied, i.e. having every drop in the moment-curve stop further loading and start gradually unloading the plastic hinge and redistributing the loads taken out.

3. STRUCTURE DESCRIPTION

The analyzed five story frame consists of sill props, columns and the foundation beam firstly braced in an absolutely stiff ground (frame A), and in other cases in an elastic ground (frames B, C and D). The evenly distributed load affects the frame sill props and the foundation beam, and the horizontal forces affect the story ceilings. The static system of the structure is shown in Figure 1. The structure beams cross sections are b/d=0.5/0.8m, and the columns are b/d=0.5/0.7m. Four different models of varying dimensions of the foundation girder cross section were considered:

- *frame A*: no foundation girder,

- *frame B*: foundation girder cross section dimensions b/d=2.0/1.0 m and bending moment of M_{tg}=5230. kNm
- *frame C*: foundation girder cross section dimensions b/d=0.8/1.4 m and bending moment of M_{tg}=4400. kNm,
- *frame D*: foundation girder cross section dimensions b/d=0.5/1.0 m and bending moment of M_{tg}=1300. kNm,

The elastic ground the foundation structure leans on (frames B, C and D) was replaced by the Winkler model with different embedment indexes k, varying in the range of 1000 to 100000 kN/m^3. All of the structure elements are reinforced concrete ones, concrete type MB 35 for beams and columns, and MB 25 for the foundation beam. The maximum moment the columns and beams can absorb, with presumed reinforcement dilatation of ε_a=10‰ and concrete dilatation of ε_b=3.5‰, is for beams M_g=1116. kNm, and for columns M_s=855 kNm.

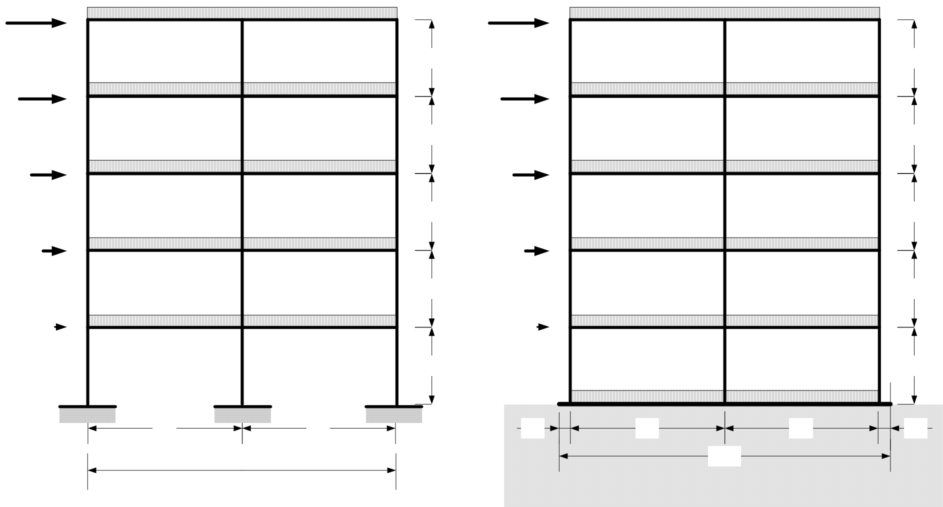

Figure 1. Five story frame

4. NUMERICAL ANALYSIS

Calculations were done using nonlinear static pushover analysis in the SAP 2000 software. Basic parameters used in the analysis reflect the material nonlinearity by applying plastic hinges (Figure 2). The idealized plastic hinge force - strain (moment-curve) interdependency is shown in five specific points. Point B represents yielding, whereas point C denotes the ultimate capacity. Point D represents the residual strength (20% of the capacity limit), while point E indicates the total collapse of the plastic hinge. The B-C section introduces the effects of material strengthening (0-10%) in the calculation. The values for the specific points in forming the idealized plastic hinge force - strain interdependency were set based on the recommendations given in the FEMA-273 Guidelines.

Plastic hinges were formed to correspond with the bearing capability of every specific structure element section. Since all the structure elements were symmetrically reinforced on both sides throughout their lengths for each of the sections in all element types, the same plastic hinge was applied.

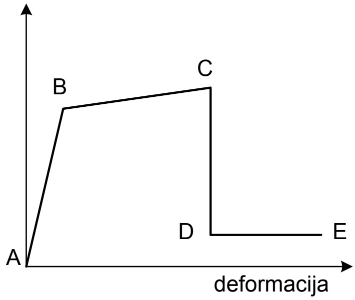

Figure 2. Idealized plastic hinge force -strain (moment-curve) interdependency

Nonlinear static analysis of frame A yielded the maximum base shear of S=1684kN, and the building top displacement of 0.16m, of which the "pushover curve" is shown in Figure 3.

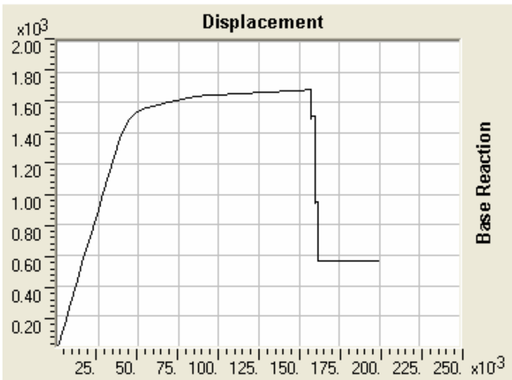

Figure 3. "Pushover curve" for frame A

Figure 4 shows the positions of the first plastic hinge, as well as the positions of plastic hinges at the total structure collapse time point. Plastic hinge formation starts when the maximum bending moment with braced middle column is reached, followed by the formation of hinges at first story beam endings. Further increasing the horizontal strain progresses plastic hinge formation towards higher stories. When bearing capacity is reached in the first hinge section (i.e. the section collapse), due to the redistribution of the static influences, the bearing capacity of other plastic hinges is rapidly reached, causing total structure collapse. The figure shows that at the moment the structure collapses the maximum bending moments of the top two stories are not reached.

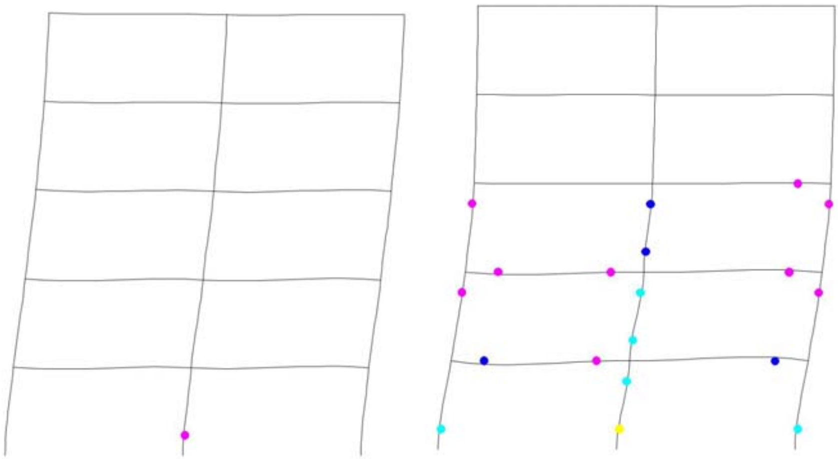

Figure 4. Plastic hinges positioning for frame A

Frame B is characterized by a stiff foundation beam in an elastic ground. Calculations were made for various values of the elastic ground response index, resulting in different base shear values. Lower embedment index values (~1000 kN/m^3) result in lower base shears values which, compared to the base shear of frame A, were ~10% lower. Higher embedment index values (≥100000 kN/m^3) resulted in practically the same values of base shear as in frame A. Caused by extremely stiff foundation beam, the influence of the deformable ground is non-existent. Thus, frame B may be considered to behave the same way the braced frame A did, and because of that the positions and distribution of the plastic hinges is the same as in frame A (Figure 4).

Frame C is characterized by a somewhat more flexible foundation beam than in frame B. Base shear for lower values of the embedment index (~1000 kN/m^3) differ by ~20% from the base shear of frame A, and with higher stiffness of the ground (≥100000 kN/m^3) the base shear is the same as in frame A. The positions and distribution of plastic hinges formation regardless of the ground stiffness is almost the same as in frame A (Figure 4).

Frame D is characterized by a very flexible foundation girder, thus the influence of ground stiffness is greater. Base shears for embedment index values of 1000 kN/m^3 to 25000 kN/m^3 respectively differ ~45% to ~15%, from the base shear of frame A. With ground stiffness (≥25000 kN/m^3) the base shears tend to approach the base shear of frame A.

Figure 5 shows the total base shear change in frames B, C and D for various embedment index values. The diagram shows that with higher ground stiffness values, the ground elasticity influence is insignificant, resulting in base shear of frame A being the merit.

Owing to the more flexible foundation girder the positions and distribution of plastic hinges formation in frame D is significantly different relative to frame A. For lower vales of the embedment index (1000 kN/m^3 - 25000 kN/m^3) the first plastic hinge appears at the joining of the foundation beam and mid column. Plastic hinges continue to form in sections around the spans of the foundation beam, followed by those at the ends of the first story columns and beams endings. The collapse at the first plastic hinge progressively induces collapses in the succeeding plastic hinges formed on the foundation beam, and plastic hinges formation in second story columns and beams. Therefore it can be assumed that the total structure collapsed along the foundation beam. The specific positioning of the plastic hinges in this case is shown in Figure 6.

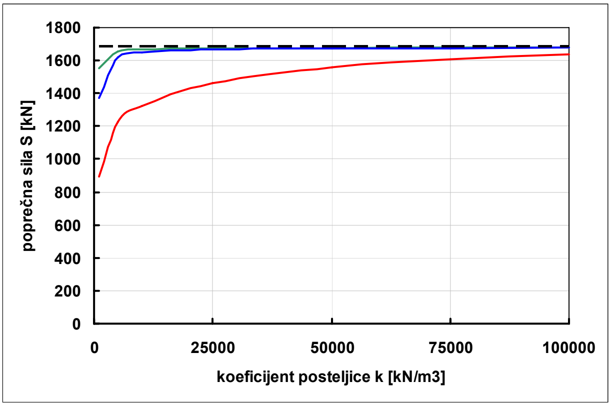

Figure 5. Base shear change relating to ground stiffness diagram

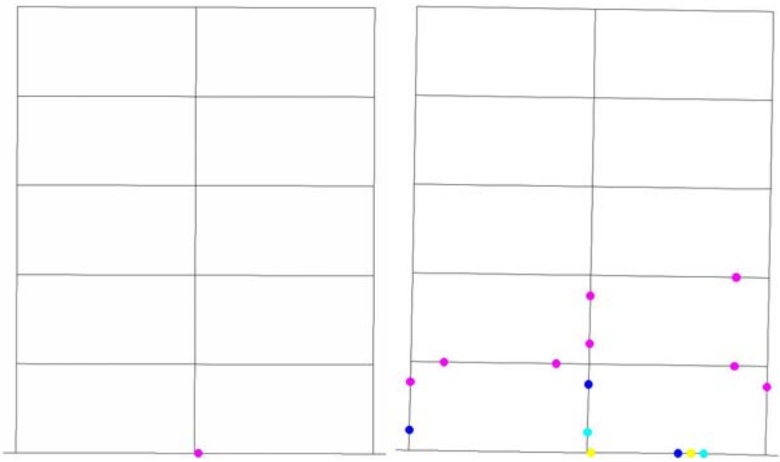

Figure 6. Plastic hinges positioning for frame D with the embedment index of 1000 kN/m^3 to 25000 kN/m^3

Specific positioning of plastic hinges in frame D for higher values of ground stiffness (k≥50000 kN/m^3) is shown in Figure 7. Just as well as in the case of less stiff grounds, the position of the first plastic hinge is the joining of the foundation structure and the mid column. Further increases of the horizontal stress causes the continuing formation of plastic hinges at first story columns and beams endings, as well as the mid span of the foundation structure. The bearing capacity is reached in the section of braced mid column, and due to the redistribution of the static influences, plastic hinges are rapidly formed at higher stories columns and beams endings. The figure shows that at the collapse time point the top two stories do not reach the maximum bending moments.

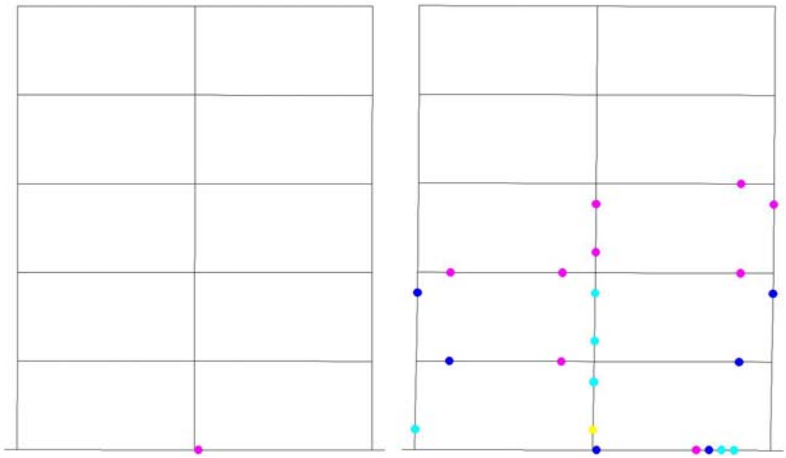

Figure 7. Plastic hinges positioning for frame D with the embedment index of ≥50000 kN/m³

5. CLOSING REMARKS

These analyses of a five story frame on an elastic ground under seismic action show that the influence of ground interaction is significant only in cases of softer grounds and more flexible foundation beams. These cases result in total structure collapse due to the collapse of the foundation column (frame D). In cases of stiff foundation structures (frames B and C), regardless of ground elasticity, structure collapse is caused by lower stories elements collapsing without the influence of the foundation girder. This justifies conducting the analysis on the braced structure in absolutely stiff ground (frame A)

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TENTH NATIONAL AND
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SCIENTIFIC MEETING

iNDiS 2006

NOVI SAD, NOVEMBER 22-24,2006.

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PRACTICAL USE OF SCC HIGH STRENGTH

Summary: *There are many cases of applications of SCC in structures in the world. Leak of new materials in SCG markets, especially new generation of admixtures (polycarboxilate) has occurred late of practical applications of SCC. Some experimental results are presented but there were not practical applications. In last two years we can find new generations of admixtures and start new period in concrete use. First practical use of SCC was on bridges on the river Bosnia in Doboj and Modrica. In this structures there were a plenty of reinforced and we were not possibilities to use conventional concrete. Recommendations for concrete were high fluidity and high strength C50/60. Also, we tested frost resistance, creep and shrinkage.*

Key words: *SCC, high strength.*

PRAKTIČNA PRIMENA SAMOZBIJAJUĆEG BETONA VISOKE ČVRSTOĆE

Rezime: *Primena samozbijajućih betona u konstrukcijama u svetu je veoma raširena. Nedostatak novih materijala na Srpskom tržištu, posebno aditiva nove generacije (polikarboksilata), uslovio je kasnu praktičnu primenu samozbijajućih betona kod nas.Prikazani su eksperimentalni rezultati dok praktične primene nije bilo. U zadnje dve-tri godine moguće je naći aditive nove generacije tako da to predstavlja početak nove praktične primene betona kod nas. Prva praktična primena samozbijajućih betona bila je na mostovimana reci Bosni u Doboju i Modriči. U ovim konstrukcijama bilo je mnoštvo armature tako da nije bila moguća primena konvencionalnih betona. Zahtevi koje je morao beton da ispuni odnosili su se na veliku fluidnost kao i visoku marku pritisne čvrstoće C50/60. Takođe, mereno je skupljanje, tečenje i otpornost betona na dejstvo mraza.*

Ključne reči: *samozbijajući beton, visoka čvrstoća.*

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1. INTRODUCTION

In the last few years self compacted concrete began to take its own place in the region of former Yugoslavia.

Thanks to the donation of Japan′s government two concrete bridges over the Bosna river in Doboj and Modriča (Republic of Srpska) are under construction (expected end of construction is October 2006.).

Both bridges are designed by Nippon Koei Co Ltd in Consortium with Central Consultant Inc., Tokyo, Japan, main contractor is Obayashi Co. Japan, subcontractors are "Integral Inženjering" Laktaši and "ŽGP" Sarajevo.

Institute for testing materials (IMS) from Belgrade, Serbia, took great part in construction of mentioned bridges, including preliminary investigations of materials, preliminary trials of concrete mixes for all concrete classes, production control and final evidence of concrete properties, both in fresh and hardened state.

2. DESCRIPTION OF BRIDGES

Bridges over the Bosna river in Doboj and Modriča are classically designed and constructed bridges. Bridge in Doboj is 200 m long (five spans of 40.00 m), perpendicular on river main flow; bridge in Modriča is 240 m long (six spans of 40.00 m), at the angle of 60^o between bridge and main river flow.

Bridge substructures are composed of massive reinforced footings, abutments, piers and pier heads, made of concrete class C 25/30. Superstructures are composed of four main girders (prestressed single beams) in each 40.00 m span, supported on abutments and pier heads by means of neoprene bearings. Each single beam is 2.50 m high, classical double T shape in cross section, volume of concrete of 52.0 m^3, approximately mass 130 tons. All single beams in one span are conected above bearings and in quarters of span with prestressed concrete cross beams.

Despite three heavy floods in summer of 2005. all substructure works were succesfully finished in the autumn of 2005. Main girders production started on 01.09.2005. on both sites, finished 28.04.2006. in Doboj and 21.05.2006. in Modriča, interrupted during winter season 2005/2006.

First intention of the designer was to produce main girders in section 10 m long in precast concrete plant 18 km from the Doboj site, transport to the site, connect sections with epoxy resins in final position and then apply prestressing force. After carefully examining all relevant factors this idea was abandoned and main girders were produced in situ over heavy scaffolding.

After structural design following classes of concrete were required:

Footings, piers and pier heads:	C 25/30
Concrete piles:	C 30/37
Main girders:	C 50/60
Cross beams and bridge deck slab:	C 30/37

Compressive strengths on preliminary trials shall be 20 % greater than required strengths.

All concretes shall have entrained air in fresh state 4.5 ± 1.5% for durabillity reasons (this requirement was later set up to 3.5 ± 1.5% for main girders concrete because of strength requirements).

This work represents all activities, problems, solutions and final results of main girders production and quality control.

3. CHOICE OF CONCRETE INGREDIENTS

First investigations of materials for construction of bridges in Doboj and Modriča started in late autumn of 2004. After carefull examination of all available materials and some pilot trials on concrete, following materials were chosen for concrete preliminary trials:

Cement: CEM I 52.5N from factory Našicement, Croatia,
Aggregates: 0-4 mm natural from Bosna river (for all concretes)
4-8 and 8-16 mm natural from Bosna river (for concrete classes up to C 30/37)
4-8 and 8-16 mm crushed from Vrbas river for concrete class C 50/60.
Admixtures: Glenium 51 (hyperplasticiser, produced by MBT Italia),
Meta Air (air entraining agent, produced by MBT Italia).
Water: from city supply

All concrete ingredients are under permanent control of certified Testing house. Basic tests on all materials for making preliminary trials have been made too.

Fractions 0-4 mm (both natural and crushed) from all suppliers in wide region around Doboj and Modriča show lack of fine particles so cement content used in preliminary trials was some higher than usually, just to replace lacking particles. This question was more a matter of economics than technics.

4. PRELIMINARY TRIALS

Preliminary trials for all classes of concrete have been carried out during winter 2004/2005 in laboratory for concrete of Institute IG in Laktaši, Republic of Srpska, in presence of representatives of Nippon Koei Co. and Obayashi Co. On the basis of test results on preliminary trials and work trials on chosen and adopted concrete plants, mix proportions were adopted.

Being air entraining was prescribed for durabillity reasons, having in mind high required strengths for main girders and that entrained air decreases compressive strengths, comparative concrete mix of class C 50/60 without entrained air and all necessary tests on hardened concrete were made. Compressive strength increases from 75.3 MPa to 92.1 MPa, unit weight of concrete increases from 2377 to 2437 kg/m3. Further tests had shown that loss of compressive strength after 250 cycles of freezing and thawing is less than 2 % (after standard JUS M.M1.016 maximum permissible strength loss is 25 %) and concrete satisfies requirements of standard JUS U.M1.055 regarding resistance against freezing and deicing salts. Proposal for alternative concrete mix without entrained air (for main girders only) with additional tests were submitted to Contractor but not accepted.

5. CONCRETE PLANTS

Choice of concrete plants for construction of bridges was very important because of required properties for both fresh and hardened concrete and total amount of concrete. Carefully examining available concrete plants in region of Doboj and Modriča, following concrete plants have been chosen:

Doboj bridge:

"Širbegović" concrete plant, 18 km from Doboj (abandoned after few weeks for problems in transport and retention of fresh concrete properties),

"ŽGP" Sarajevo, plant Doboj (25 m3/h, 1.5 km await from the site) as main plant,

"Autokomerc" Doboj (15 m3/h, 100 m await from the site) as spare plant, sometimes used simultaneously with main plant.

Modriča bridge:

"ŽGP" Sarajevo, plant Modriča (25 m3/h, situated on the site) as main plant,

"ŽGP" Sarajevo, plant Doboj (25 m3/h, 47 km await from the site) as spare plant,

Those setups proved to be satisfactory, both for capacity and fresh concrete uniformity. None of concrete plants have automatic devices for batching of admixtures, even so, thanks to the very good control fresh concrete was uniform regarding required properties.

6. PREPARATION AND START OF CONCRETING

Concreting of both bridges started in the spring of 2005. During the first months of concreting some problems concerning placeabillity and workabillity of fresh concrete have been noticed. This regards mainly on relative fast loss of workabillity (slump loss, fresh concrete became sticky, supplementary adition of admixtures on the site for improving fresh concrete properties was not permitted). Being first blocks of concrete (footings, pier lifts) were relative massive, there was no great problems in placing concrete (by concrete pumps and buckets) and in final appearance and quality . Although cement and admixtures (hyperplasticizer, polycarboxylate aesther based, applied within recommended limits) used, were of high quality, it seems that something in their combination is wrong. Mutual opinion of Consultant, Engineer and Contractors were not to waste time to establish what is wrong in applied combination of materials than to find out better combination of materials which will suit all requirements. Trial concreting of one 10 meters long section of main girder had shown that first adopted mix proportion for concrete class C 50/60 was not suitable for main girders production:

lot of vibrations (both with external and internal vibrators) had to be applied.

lower flange and partially girder web (25 cm thick), being congested with reinforcing bars and tendon ducts, cannot be satisfactory filled with concrete, great honeycombs occured.

In advance presumming such problems, Concrete Engineers from IMS and IG Institutes proposed to Consultant following changes in fresh concrete composition which will not affect negativelly on required concrete properties:

water to cement ratio retain as low as 0.35,

retain the same cement quantity (on the demand of Consultant, although cement content could be decreased), increase workabillity of fresh concrete (increase slump up to 25 ± 3 cm) by adequatelly adjusting amount of plasticizer,

replace existing aggregate fraction 0-4 mm with already found, much better regarding grain size distribution, fraction from Sava river (produced by MGM Tolisa, Orašje),

find out better combination cement/ hyperplasticizer.

Several preliminary trials were carried out in period May – July 2005, followed by another trial concreting of main girder section in the beginning of August 2005 with mix proportion adopted by Consultant. Replacing hyperplasticizer was Dynamon SX (modified acryl polymer based, produced by MAPEI, Italia). Compressive strengths, both of preliminary trials and work trials were of the same level as in preliminary trials with precedent combination of materials (about 75 MPa after 28 days). Fresh concrete used for work trial was of flowing consistency, Abrams cone slump 25-27 cm, Abrams cone spread 60-65 cm. Despite relative high ambiance temperatures (about 30^{o}C), concrete retained the same consistency for more than one hour. During work trial concreting settlement of the steel mould bottom (made of wood) occured and about 4 m^3 of concrete leaked through the created gap to the distance of about 6 meters to the left and right side. During leaking of concrete neither segregation nor lamination were observed. After repairing of mould concreting was successfully finished, after removing of mould concrete surface was quite uniform.

Comparative compressive strength tests were carried out on cubes cast with slight vibration on vibrating table and without any vibrations (concrete was simply poured into moulds and levelled with steel ruler). No significant difference in unit weight (less than 10kg/m^3) and compressive strength (about 1 - 2 MPa) were observed.

Although concrete used for making main girders on both bridges could be named and used as "selfcompacting concrete" for plain and light reinforced structures, some light vibrations (much less than usually) were applied, on the demand of Consultant, because of great amount of reinforcing bars (about 200 kg/m3) and presence of total six prestressing tendon ducts 8.5 cm in diameter so for the greatest part of main girder there was only 2.5-3 cm space left for concrete to pass into the lower part of girders. Slight vibrations were applied just to be sure that each reinforcing bar and each tendon duct were totally wrapped and covered with concrete. So, this concrete could be named as "almost self compacting concrete" and expirience and knowledge from those structures are good start point for future use of self compacting concrete in various types of structures.

7. QUALITY CONTROL OF CONCRETE WORKS

Very important part of Doboj and Modriča bridges construction was good organized and strictly performed quality control at all stages of construction, under permanent supervision of Consultant and Designer representatives. Site laboratories for concrete for basic tests of fresh and hardened concrete were established at both sites prior to start of concrete works.

Quality control consisted of:

Control of all materials delivered on the site (visual inspection, sampling and testing in laboratories of IMS and IG Institutes, providing of certificates issued by certified Testing Houses), Control of materials on concrete plants prior to concreting (amounts of each material, visual inspection, moisture content in aggregate),

Permanent control during batching and mixing of concrete by expirienced technician of IMS Institute (amount of ingredients, consistency, entrained air),

Control of fresh concrete from each truck mixer delivered to the site (consistency, entrained air, temperature, chloride content),

Making of concrete specimens (15 cm cubes and 15/30 cm cylinders for compressive strength test), curing specimens to the moment of testing,

Compressive strength test at 7, 28 and 91 days.

All control tests of concrete ingredients performed at Laboratory for concrete of Institute IG in Laktaši.

Compressive strength tests performed at site laboratory in Doboj (15 cm cubes), laboratory for concrete IG laktaši (15 cm cubes) and laboratory for concrete IMK Banja Luka (15/30 cm cylinders).

Thanks to the very good control in concrete production, fresh concrete have had very uniform properties in the moment of delivery on the site:

Variation in slump were low (24 up to 27 cm),

Entrained air varied mainly from 2.5 up to 3.5%.

Hardened concrete satisfies all requirements regarding compressive strength and appearance. It is worth to mention that cement used for concrete works on both bridges has significant strength gain from 28 up to 91 days (18 %), this can be useful for structural analysis on future projects.

Shrinkage and creep of concrete has been determined at IMS Institute in Belgrade, according to standards JUS U.M1.027 and JUS U.M1.029 using specimens cast on the site of Doboj bridge. Mean valuse of deformations after 91 days were:

shrinkage: 0.27 mm/m.
creep: 0.85 mm/m.

TRIAL MARK	PP-08 PRELIMI-NARY	PP-11 PRELIMI-NARY	08D PRELIMI-NARY	WORK TRIAL MODRIČA	WORK TRIAL DOBOJ
Date of execution	24.12.04.	27.12.04.	27.07.05.	09.08.05.	10.08.05.
Cement CEM I 52.5N kg	578	575	567	567	567
Aggregate total kg	1622	1682	1615	1615	1615
Fraction 0 – 4 mm kg	730[1]	757[4]	727	727	727
Fraction 4 – 8 mm kg	324	336	323	323	323
Fraction 8 – 16 mm kg	568	589	565	565	565
Hyperplasticizer kg	2.53[2]	3.17[5]	4.56	4.56	4.56
Air entrainer kg	0.29	0.00	0.06	0.06	0.06
Water kg	191	190	198	198	198
Slump cm	22.0	23.0	25.0	27.0	25.5
Entrained air %	3.5	0.7	2.7	3.0	2.8
Temperature ^{o}C	17	16	31	26	26
Unit weight kg/m^3	2393	2450	2385	2385	2385
7 days strength MPa	57.9	75.4	71.7	64.2	65.6
28 days strength MPa	75.3	92.1		77.4	76.6

Table 1 Summary on preliminary and work trials

[1] from Bosna river (trials PP-08 and PP-11); from Sava river (trial 08D and work trial)
[2] Glenium 51 (trials PP-08 and PP-11); Dynamon SX (trial 08D and work trials)

DOBOJ BRIDGE					
	15 cm cubes			15/30 cm cylinders	f_{91}/f_{28}
age days	7	28	91	28	
number of results	30	30	20	20	
minimum strength MPa	58.4	64.6	77.5	57.4	
mean strength f_{cm} MPa	66.5	73.3	85.4	65.2	1.18
maximum strength MPa	74.1	84.6	94.7	77.5	
standard deviation MPa	4.6	5.4	4.4	4.6	
characteristic strength MPa	59.8	65.3	78.9	58.4	
MODRIČA BRIDGE					
age days	7	28	91	28	
number of results	38	38	3	21	
minimum strength MPa	57.3	64.3	80.1	58.3	
mean strength f_{cm} MPa	66.3	71.4	80.3	68.3	1.18
maximum strength MPa	77.5	78.7	80.6	77.7	
standard deviation MPa	5.9	3.7		5.8	
characteristic strength MPa[1]	57.6	66.0		59.7	

Table 2 Compressive strengths – statistical parameters

[1] Characteristic strength: $f_{ck} = f_{cm} - 1.48\ \sigma$ (after DIN EN 206-1: 2000)

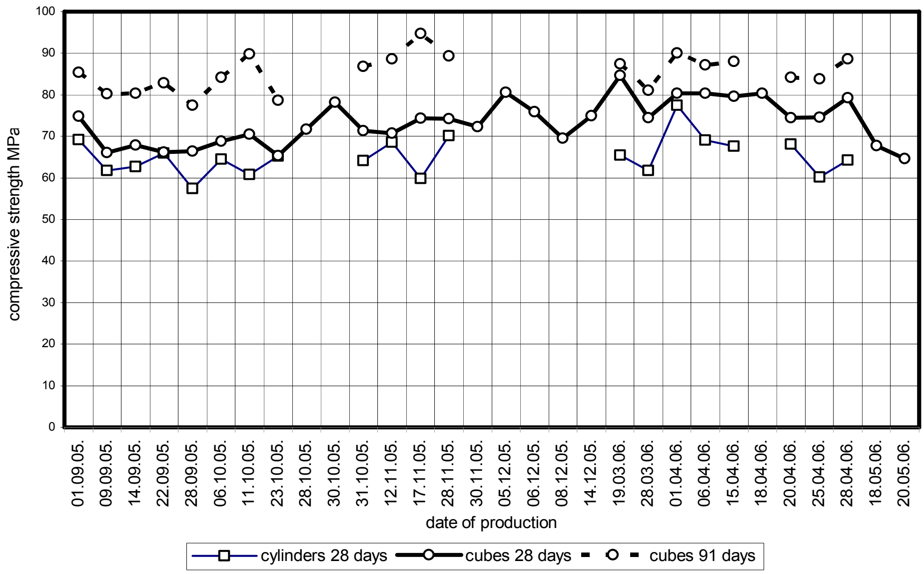

Figure 1. Doboj bridge – strength test chartConcrete class C 50/60

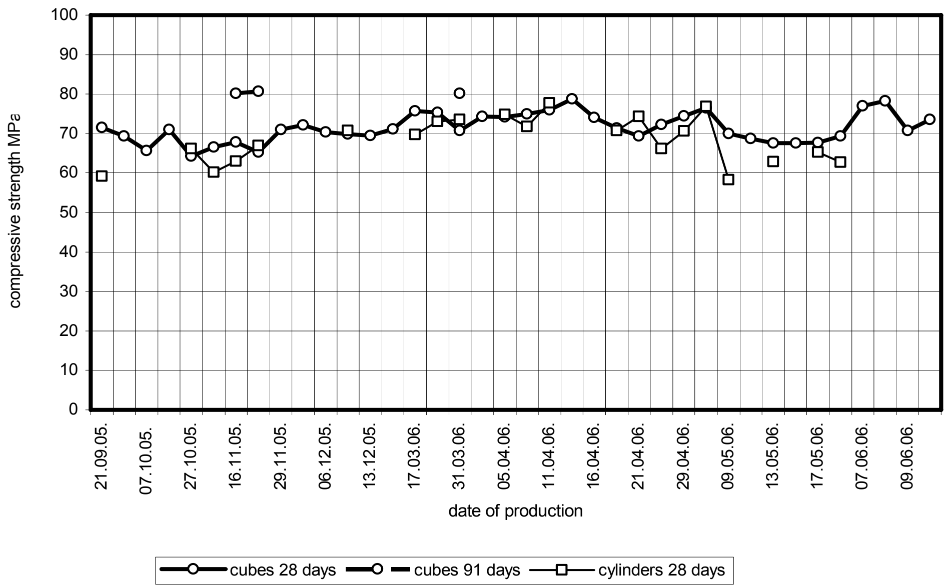

Figure 2. Modriča bridge – strength test chart concrete class C 50/60

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iNDiS 2006

NOVI SAD, NOVEMBER 22-24,2006.

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RETROFITTING OF A HISTORIC MASONRY BUILDING

Summary: The behavior of a historic unreinforced masonry building is investigated for seismic motions. The building suffered extensive damage and partial collapse during the Aegion earthquake of June 15th, 1995. The objective of the investigation is to propose suitable measures to restore and strengthen the structure while retaining its architecture. In addition to the analysis imposed by the seismic code time-history analysis is performed using an elaborated finite element (FE) model and the excitations of the Aegion 1995 earthquake recorded at a station near the structure. The material properties in the FE model were obtained by laboratory testing. The accuracy of the analytical results is, at least, qualitatively validated in terms of the actual damage of the structure after the M_w=6.4 Aegion seismic event. The diaphragm behavior resulted to be determinant in the seismic behavior of the building. Alternative strengthening schemes have been developed and compared in terms of their effectiveness. Enhancing diaphragm action is proven to be the most effective means to retrofit the structure.

Key words: masonry building, near-source seismic motion, seismic analysis and retrofit

POJAČAVANJE ISTORIJSKI ZNAČAJNIH ZIDANIH OBJEKATA

Rezime: Ponašanje istorijski vrednih zidanih objekata pri seizmičkim pomeranjima je istraživano. Objekat je pretrpeo izuzetno velika oštećenja i delimičan lom tokom "Aegion" zemljotresa 15 juna 1995. Cilj istraživanja je da se predlože odgovarajuće mere za restauraciju i pojačavanje objekta uz očuvanje njegove arhitekture. Pored analize prema seizmičkim propisima, urađena je i analiza vremenskog odgovora korišćenjem detaljnog modela sa konačnim elementima (FE) i podaci o pobudi "Aegion" zemljotresa iz 1995 godine koji su zabeleženi u mernoj stanici u blizini objekta. Svojstva materijala u FE modelu su dobijena laboratorijskim ispitivanjem. Tačnost analitičkih rezultata je, u najmanju ruku, kvalitativno potvrđena stvarnim oštećenjima konstrukcije nakon M_w=6.4 "Aegion" zemljotresa. Dobijeni dijagram ponašanja je odlučujući za seizmičko ponašanje objekta. Alternativni planovi pojačavanja su bili razvijeni i upoređeni prema njihovoj efektivnosti. Dijagram povećanih dejstava je dokazano najefikasniji za pojačavanje konstrukcija.

Ključne reči: zidane zgrade, bliski zemljotres, seizmička analiza i pojačavanje

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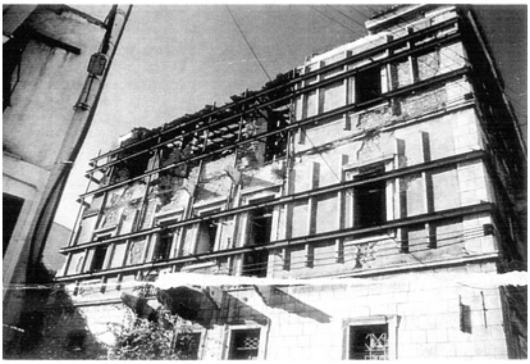
1. INTRODUCTION

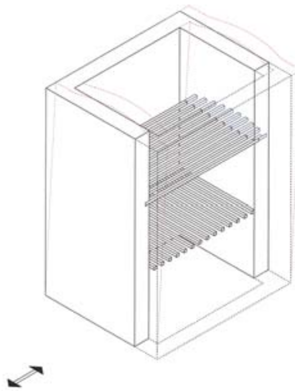
The city of Aegion has suffered several strong earthquakes through the years. The most recent powerful earthquake which caused human losses occurred on June 15, 1995. It was a M_w=6.4 shallow depth earthquake with an epicentral distance of about 18 kilometers southwest of Aegion that caused the death of 26 people, massive, and in many cases, irreparable, damages to both modern and older masonry buildings [6]. In 1997 a weaker earthquake caused additional damage on the already deteriorated buildings.



Photograph 1. Horizontal ties at corners Photograph 2. South view of the building.

The three story Panagiotopoulos' mansion is located at the city center of Aegion, [Photo 1]. It has a basement of similar dimensions with the other floors. Its plan is rectangular with dimensions 18.68m by 14.65m and height of floors of about 4.5m [7]. The external walls of the building have been prestressed with either metal or wooden tendons at the ground floor level. Metal blades have been used to anchor the tendons, as shown in Photograph 1.

a) *Representative section of external masonry wall*



b) *Arrangement of wooden floors*

Figure 1. Structural details

The external walls are made of sack masonry with thickness that is reduced with height. The walls are three-layered with exterior sides constructed with large boulder stones connected with mortar and smaller stones with thicker mortar placed between them. In the whole building, there are quite a few closets incorporated in the masonry walls that reach the height of interior openings. The openings of the external walls are formed at the upper part with shallow arches made of bricks. About 18% of the external walls are openings in the northern, southern and eastern view. In the western view about 10% are openings. In the interior of the building there are two walls made of stone masonry.

The external walls are made of three-layered limestone masonry of reduced thickness ranging from 95cm at the ground floor to 86cm at the second floor (Fig. 1a). The light internal partition walls on the first and second floor of the building are made of bricks with a thickness of 17cm to 18cm supported on either powerful masonry walls or wooden floor beams.

The first and second story floors are made of timber place as shown in Fig. 1b. The floors of the ground floor are either timber or stone masonry domes located at the central part of the building where the wall openings of the basement are relatively small. The roof is covered by byzantine style tiles supported by a timber truss [3].

2. DAMAGE EVALUATION

The building has suffered extensive damages caused by the 1995 and 1997 earthquakes. An important role to the damages has played, on the one hand, the insufficient maintenance prior to the earthquakes, and on the other hand the lack of taking all the necessary protection measures after the 1995 earthquake.

Photograph 2 shows an extensive collapse of a large part of the external wall on the second floor. The southwestern corner of the building on the second floor has also collapsed. A qualitative interpretation of the representative damages is shown in Fig. 2:

A. Crack of parabolic form over a window opening caused by inadequate bending strength. After the collapse of the southern external wall, the rest of the stone masonry carries even higher compressive loads.

B. “Shear”cracks forming an “X” caused by the alternating direction of seismic action. The inclination and form of the cracks depends on the vertical compressive loads carried by the walls, the position and size of the openings [2,14].

C. Cracks round the openings developed by the seismic forces parallel to the wall.

D. Crack located at the level of the arch at the ground floor. The crack is caused by either horizontal thrust acting on the masonry wall resulting in bending stresses and the co-existence of the intense shear stresses in this area.

E. Vertical cracks in the panels between windows; caused by intense compression.

F. Diagonal crack at a wall with reduced thickness to form the chimney.

G. Bending cracks at the top level of windows caused probably by the absence of timber reinforcing beams in the wall frame.

H. Vertical crack at the transverse wall connections and detachment.

3. SEISMIC ANALYSIS AND DYNAMIC CHARACTERISTICS

The building walls were modeled and analyzed using finite elements [8]. Linear elastic behavior was assumed using three and four node thick shell elements, a process that is usually adopted for the analysis of similar structural systems. Beam elements were used to model the timber parts, such as the floors. Time history analysis was carried out for the seismic records shown in Fig. 3 [1].

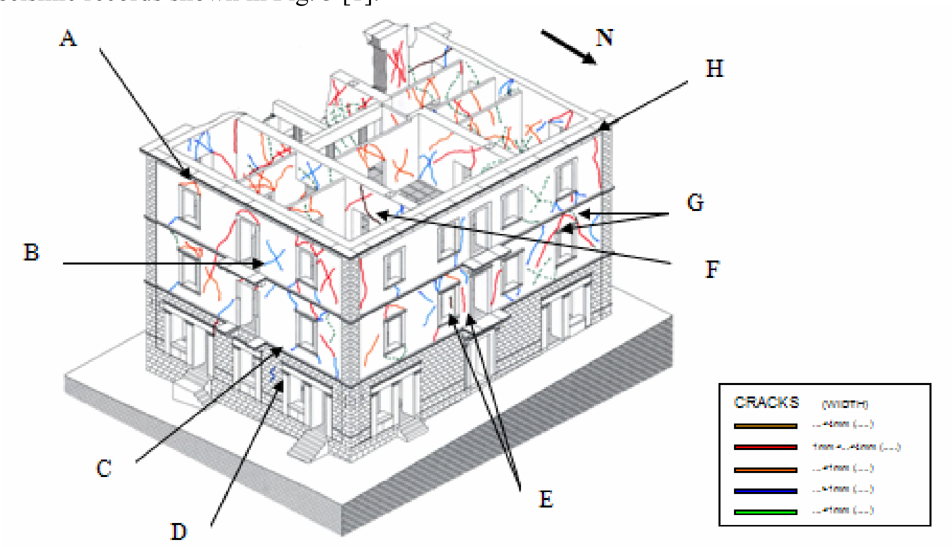
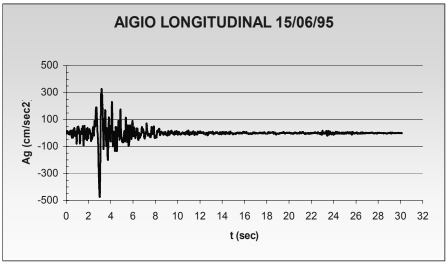
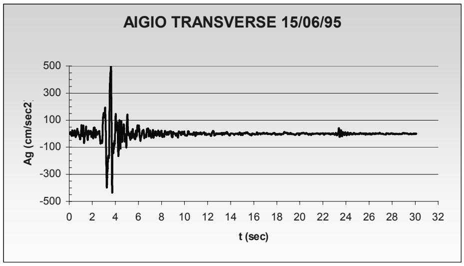


Figure 2. Damage distribution from visual inspection.

It must be emphasized that because of the orientation of the building along the direction of the coastline (which is also the direction of the earthquake fault), the longitudinal record applied along the X direction of the building, whereas the transverse record was applied along the Y direction. This selection was based on the fact that the X direction of the building is parallel to the coastline. The records correspond to horizontal ground acceleration recorded a few kilometers from the mansion.



a) Longitudinal component



b) Transverse component

Figure 3. Acceleration time history of the Aegion 1995 earthquake

The near-source ground motions are characterized by pulses with a maximum acceleration of 0.54g, a long period of about 0.5sec along the transverse direction to the fault and velocity of about 52cm/sec [4,10,11]. The first most significant mode along the X direction was found to be the 8^{th} mode of the building with T = 0.17sec which is with the mode with the largest participation factor (-39.65) and modal mass (62.82%). The 9^{th} mode is the first most significant mode along the Y direction with a participation factor (31.02) and modal mass (38.46%).

4. ACTUAL PERFORMANCE AND ANALYSIS RESULTS

Masonry structures, such as the Panagiotopoulos building in Aegion have a small out of plane bending stiffness. The Aegion earthquake caused mostly bending out of level response and as result tension prevailed over the construction with detrimental effects.

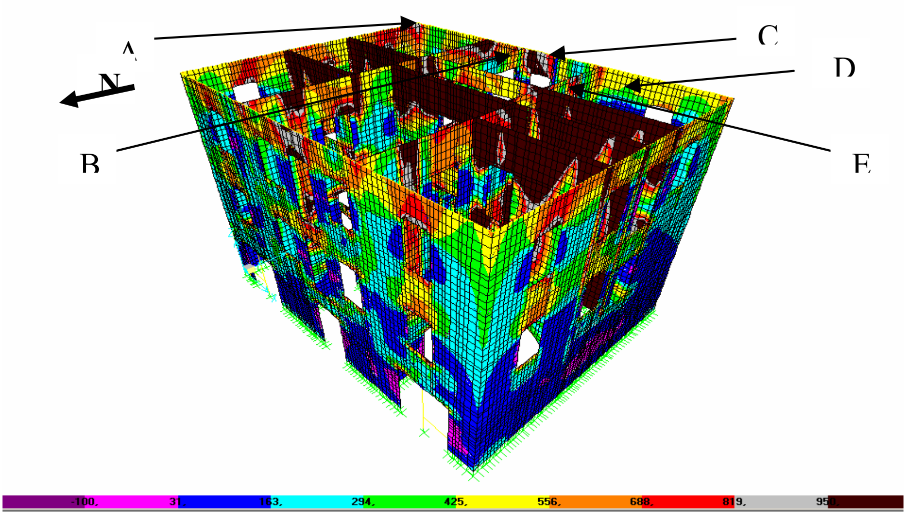


Figure 4. NW view of finite element model. Red color depicts the areas with equal stresses varying from 698KPa to 826 KPa, gray scale varies from 826 KPa to 977 KPa and brown denotes stresses greater than 977 KPa, for the G + 0,3Q + EY + 0,3EX combination [7].

5. INTERVENTION AND RETROFIT SCENARIOS

Several methods of intervention were examined in order to select the most effective intervention type for the building. Taking into account the results of dynamic analysis the following measures were proposed [5, 7, 13]:

1. Provide diaphragm behavior at the top of the ground and the first floor by strengthening the connections of the timber floor to the masonry walls combined with the construction of a concrete kerb at the top of the second floor.
2. Strengthen diaphragm function at all levels.
3. Construct a concrete kerb to create an overall "box" connection at the top of the masonry walls at the second floor. Also, construct wooden kerbs at the top of the ground and first floor connected with the masonry walls with steel bars and dowels.

The ratio of the final to the initial principal stress σ_{11} for the seismic combination was evaluated at critical positions at the south view of the building, as a means to compare the efficiency of the interventions proposed (Fig. 5). The median values of this ratio for the internal side of the building are presented in Table 1, with the locations A_i, B_i, C_i (i=1,2,3) indicated in Fig. 5. The 'initial' condition of the building, that is without any structural alteration, is listed as (i/n)=0, while the alternative schemes of intervention are denoted with the numbers (i/n)=1 to 3. The most effective method appears to be the second one which decreases the stress ratio drastically. Creating diaphragm behavior only at the top of the ground and the first floor appears to be as effective as the second one, only for the locations A_i and C_i. The least effective intervention method is the third one.

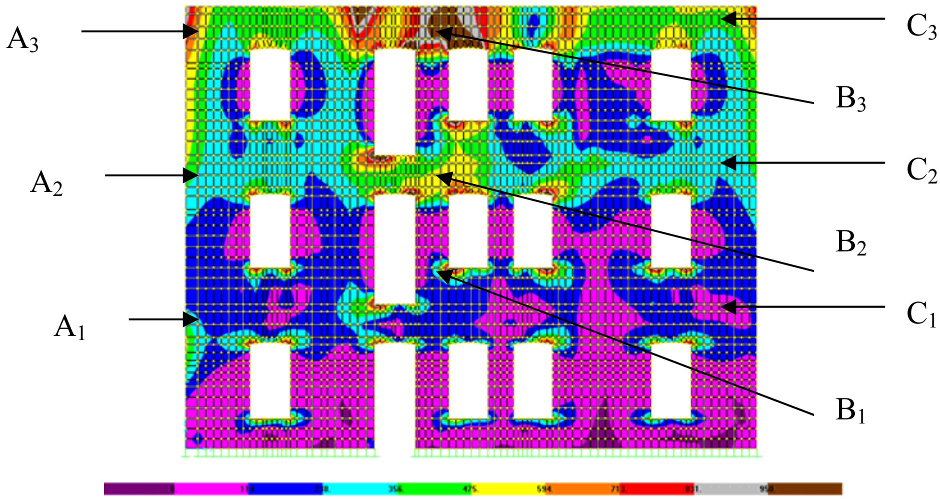


Figure 5. Stress valuation on south view.

	Ground floor			A' Floor			B' Floor		
i/n	A_1	B_1	C_1	A_2	B_2	C_2	A_3	B_3	C_3
0	1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00
1	0,08	0,85	0,19	0,07	0,64	0,04	0,25	0,43	0,29
2	0,05	0,18	0,14	0,02	0,05	0,04	0,08	0,02	0,05
3	1,00	0,82	0,86	0,89	0,78	1,00	0,87	0,68	0,92

Table 1: Values of final to initial principal stress ratio for the different types of interventions, for the internal side of the building (Fig. 5).

Analysis and architectural restrictions lead to the following types of retrofitting [5,7,9,12,15]: i) Replacement of the timber elements, necessary measure to increase the in-plane shear stiffness at the floor levels; ii) Increasing of the structural wall thickness in several vulnerable positions such as the walls near the chimney breast; iii) Strengthening of insufficient wall sections based on the analytical results; iv) Reconstruction of the collapsed part of the structure and wall retrofitting at the staircase; v) Demolition and reconstruction of inclined and deflected walls; vi) Reconstruction of corner walls and retrofitting of the transverse wall connections; vii) Repair and retrofit of the staircase; viii) Construction of a concrete kerb at the top level of the second floor; ix) Restoration of the timber roof.

Some representative examples of the proposed interventions are shown in the following figures. In Fig. 6 the implementation of diaphragm behavior of a wooden floor is depicted and the connection details of walls intersection are presented. A pair of timber cross-beams (no.1) is placed at the top of the bearing walls. Metal blades with beam forms (no. 2) are used to link the wooden beams together and with the masonry body. Metal blades are also used to connect lengthwise (no. 3) and at intersection positions (no. 4) the beams. The timber beams are extended using metal sections (no. 5) at the external side of the walls in order to provide compressive resistance through anchoring. The timber beams supporting the floor are anchored into the cross-beams at both sides using metal beam bearing plates (no. 6) bolted inside the perimetric timber beams that are placed at the level beneath them. Several transverse timber beams (no. 7) are used in both directions of the floor, while diagonal metal beams (no. 8) are placed between the timber beams underneath the plywood (no. 9) and the wooden planking (no. 10) in order to increase the in plane stiffness of the floor.

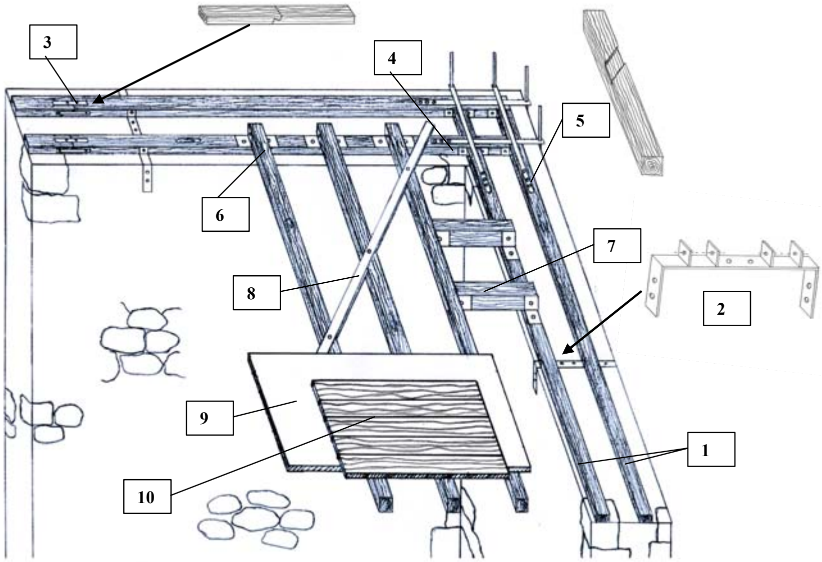
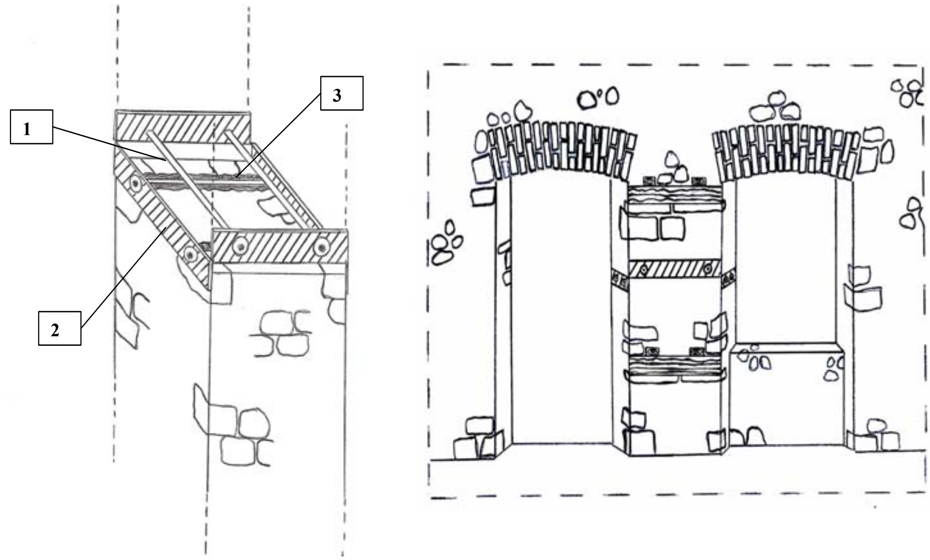


Figure 6. Enhance of diaphragm behavior of wooden floor.

Some of the most vulnerable portions of the structure appeared to be the panels between openings which, in many cases, have insufficient sections and undergo large vertical compressive forces. This is the main reason of the observed extensive vertical cracking and deflection. Figure 7a presents a typical method of retrofitting for this type of damage. Pairs of stainless metal tendons (no. 1) are placed at both directions and at different levels of the damaged panel in order to properly connect all the sides of the wall. Anchoring plates for the tendons (no. 2) are placed at the surface of the wall, while regional failure caused by compressive stresses at the anchoring position should be avoided. Special grout (no. 3) should be used to protect the tendons from corrosion.



a) Strengthening arrangement of wall panel using metal tendons *b) Inside view of the retrofitted panel*

Figure 7.Typical retrofitting techniques.

This retrofit arrangement is suggested to be limited at the middle height of the panel, as shown in Fig. 7b, in order to maintain the aesthetics of the structure at the stone lintel level. Retrofitting of arch-shaped sections, such as the staircase part of the structure, which was the one that suffered the highest tensile stresses during the Aegion 1995 earthquake, could be performed by using stainless tendons anchored at both sides of the arch. The anchorage plates should also be protected against corrosion. Special care should be taken against the stress decrease along the tendons due to creep and relaxation with systematic inspection. A detailed discussion and presentation of an extensive repair and strengthening scheme is presented in [7].

6. CONCLUSIONS

The seismic response of a three story historic unreinforced masonry building is studied for a pulse type near-source seismic excitation recorded during the Aegion 1995 earthquake. Three different intervention scenarios are examined in order to select the most efficient retrofit scheme. Diaphragm action at all levels has proven to be the most effective scheme to dramatically decrease tensile stress in the masonry walls. Several retrofitting alternatives are presented and their effectiveness compared while considering both structural integrity and the aesthetics of the historic building.

7. ACKNOWLEDGEMENTS

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A NEW APPROACH ON THE OPTIMAL THICKNESS CALCULUS FOR BUILDINGS THERMAL INSULATION

Summary:*In this work is proposed a new method for the determination of the minimal necessary thickness for the thermal insulation layer, for which the U – value is under the normative. This method is based on working with ψ and χ transmittance determined by numerical modelling or extracted from a data base.*
The method is based on a certain variation law of thermal flow magnitude depending on the thermal insulation thickness.

Key words: *thermal insulation, optimal thickness, thermal bridges, ψ and χ transmittance*

NOVI PRISTUP PRORAČUNU OPTIMALNE DEBLJINE TERMOIZOLACIJE ZGRADA

Rezime:*U radu je predložen novi metod za određivanje minimalno potrebne debljine termizolacionog sloja, za koji je U-vrednost manja od propisane.Ovaj metod je baziran na radu sa ψ i χ prenosom određenog numeričkim modeliranjem ili uzimanjem iz baze podataka.*
Metod se bazira na određenim varijacijama zakona magnitude termičkog protokoa u zavisnosti od debljine termoizolacije.

Ključne reči:*termoizolacija, optimalna debljina, termički mostovi, ψ i χ prenos*

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1. INTRODUCTION

In the actual context of energy economy requirements and durable development, a special attention is paid to the study of thermal behaviour regarding new buildings, but also regarding old ones which must be rehabilitated.

Usually, if there are no special requirements regarding the building envelope, the thermal insulation is being realised by the use, one way or another, of an insulation layer on the exterior surface of the building.

In terms of calculation it is imposed a thermal insulation thickness, based on experience, and it verifies if this is sufficient to ensure the correct thermal parameters. If, for economical reasons, a minimal thickness is wanted, there is no other way but try-out calculus, i.e. repeating numerical modelling for the determination of ψ coefficients in diverse variants, with a great amount of work.

In this paper is proposed a new method for the determination of the minimal necessary thickness for the insulation layer, for which the U–value is under the normative. This method is based on working with ψ and χ transmittance determined by numerical modelling or extracted from a data base.

The method relies on a certain variation law of thermal flow magnitude depending on the thermal insulation thickness.

2. THE PROBLEM BACKGROUND

In case of the Romanian prescriptions the necessary documentation that needs to be made for starting the process of thermophysical modernization/retrofitting of an existing building consists of two main parts :

- **a.** Thermal expertise – consist in determining the thermophysical characteristics and actual function of an existing building, in order to characterise from thermal and energetic point of view of the building.
- **b.** Energetic Audit – represents the activity of energetic retrofitting technical solutions identification, and also the optimisation of the technical solutions through their economical efficiency analysis.

In consequences, the expertise and energetic audit phases must fulfil, among others, the determination of all buildings elements thermal insulation behaviour. This implies the thermal characteristics estimation, firstly of the corrected specific thermal resistance, for the initial state of the building and also for the case of modernised solution.

The fact that the corrected thermal resistance values must be computed twice, lead to the idea that, in case if are known the corrected resistance in the existing situation (without supplementary thermal insulation) and in the modernised situation (with an arbitrary insulation thickness, but thick enough), and knowing the variation law of the corrected resistance with the thickness of the supplementary thermal insulation, it can be determined the minimum sufficient thickness for witch the corrected resistance to be superior to the resistance required in the norms, but as closer to it’s value. This kind of solution to the problem has the advantage that the appreciation of the optimal thermal insulation supplementary thickness can be rigorously accomplished, without supplementary iterative calculus, by using the two sets of results (the values of the corrected resistance determined in the expertise and audit fields), which must be computed anyway.

In this context a large number of situations has been modelled and grouped in two categories:

a. tests to illustrate the manner in which the corrected thermal resistance varies according to the thickness of the supplementary insulation layer, only in zone of the thermal bridges
b. tests to show the corrected resistance variation of a whole element, and stabilising of some mathematical expressions that must reflect this variation.

3. PROMINENCE OF THE TYPES OF VARIATION OF THE CORRECTED THERMAL BRIDGES RESISTANCE

For showing the way in which the corrected thermal resistance in the bridge zone varies with the thermal insulation thickness layer it's been started from two case studies, one about a masonry exterior wall and the second on a large prefabricated concrete panel.

3.1. The masonry wall thermal bridges

It's been considered the case of an masonry exterior wall of normal brick , with 25 cm of thickness, having columns of 25 x 25 cm and belts of 25 x 25 cm. At a current level, the corresponding wall panel of a corner room contains 5 types of linear thermal bridges (see fig.1).

In the hypothesis of a thermal insulation layer placed at the exterior surface of the wall, there have been made some modelling of the plane thermal field, computed with a special program (based on the finite element method), for a variation of the thermal insulation thicknesses between 0...12 cm , with a rate of 2 cm.

For every thermal bridge from Fig. 1 there have been made 1+6=7 patterns, corresponding to the situation with no insulation and with insulation having the thickness 2, 4, 6, 8, 10 and 12 cm, and in the end the corrected specific thermal insulation have bad determined in each situation, only for the zones of each bridge. For example, the unidirectional thermal resistance R and corrected R' are graphically represented in Fig. 2 for the corner of the wall, and in Fig. 3 for the perimeter of the window.

The tests made for the masonry wall of 25 cm thickness, have shown that for all the types of thermal bridges excepting the perimeter of the windows, the corrected thermal resistance varies linearly with the thickness of the supplementary thermal insulation layer. In case of the thermal bridges situated on the surrounding window area the variation is not linear, but can be approximated with a straight line or, more accurate, with a parabola line.

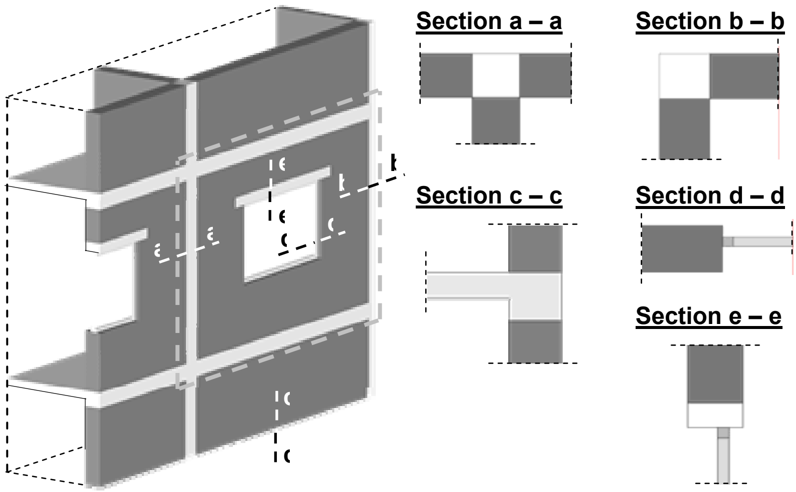

Fig. 1. Masonry wall elevation and thermal bridges details

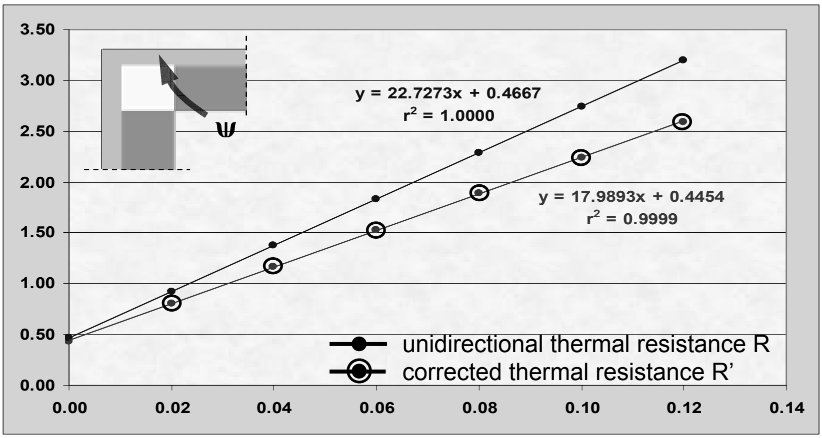

Fig. 2. *Walls corner – the variation of the thermal resistance R and R' with insulation thickness*

3.2. Thermal bridges of a prefabricated concrete panel

The prefabricated wall panel (Fig. 4), with the thickness of 27 cm, is composed of a concrete resistance layer (12.5 cm), a polystyrene thermal insulation layer (8 cm) and a concrete protection layer (6.5 cm). The characteristic section of the panel a–a, b–b, c–c and d–d represent four types of linear thermal bridges.

As in the first case of study, there have been made modelling with different supplementary thermal insulation layer disposed at the exterior surface of the wall, with their thickness growing from 0 to 12 cm, with a rate of 2 cm. The tests on the prefabricated big pannel have shown that for all types of bridges, excepting the area around the windows, the corrected thermal resistance varies linearly with the supplementary layer insulation thickness. In case of the window perimeter thermal bridges the variation is not linearly, but can be assimilated with a linear one or, more precise, with a parabolic variation.

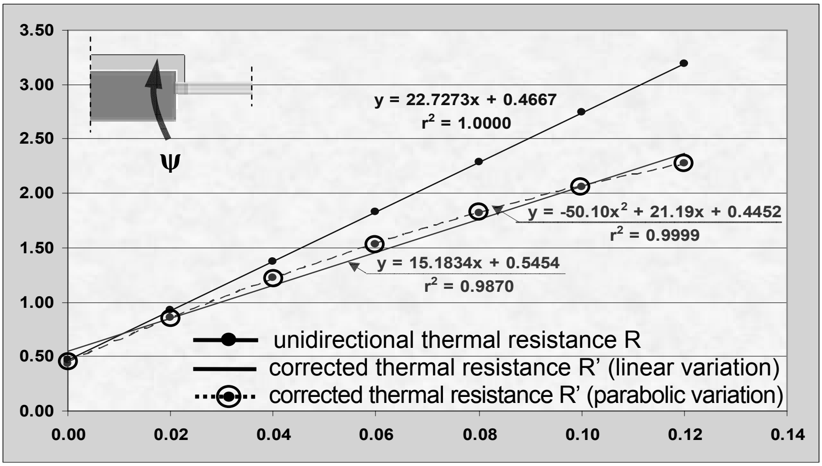

Fig. 3. *Window perimeter – the variation of the thermal resistance R and R’ with insulation thickness*

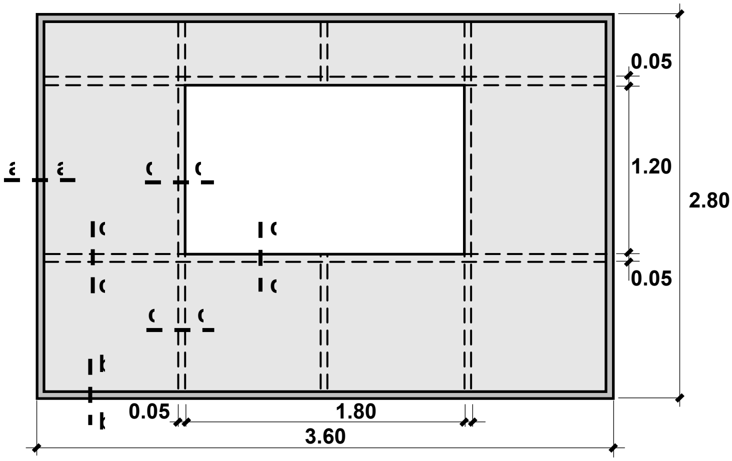

Fig. 4. *Large prefabricated wall panel*

4. THE CORRECTED THERMAL RESISTANCE VARIATION LOW SETTLEMENT

As it has been shown at the previous point, for all types of thermal bridges studied, from the masonry walls or from the prefabricated panel, in the bridge zones, the local variation of the corrected thermal resistance is linear, or almost a linear one, with the variation of the insulation thickness layer. In the next stages the global corrected resistance have been computed corresponding to an entire region of the building. For the first case (point 3.1 – masonry wall) the wall form a main facade of a corner placing room on a current floor has been chosen (Fig. 1 – the zone marked with a broken line), and for the second case (point 3.2) the zone corresponding to a large prefabricated panel (Fig. 4).

The corrected specific thermal resistance it's determined, according to the methodology recommended by our effectual regulations, with the relations:

$$R' = \frac{1}{U'}; \qquad U' = \frac{1}{R} + \frac{\sum \psi.\lambda}{A} + \frac{\sum \chi}{A} \qquad (1)$$

The linear coefficients of thermal transfer ψ have been taken from point 3.1. The unidirectional thermal resistance variation R and corrected R' with the thermal insulation thickness is shown in Fig. 5.

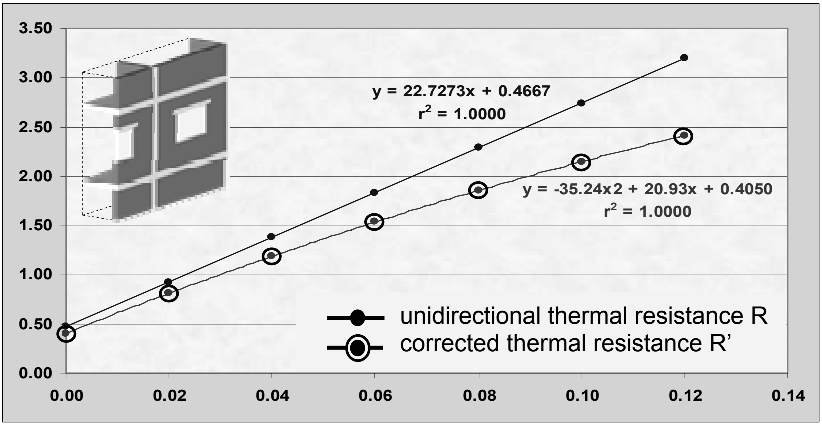

Fig. 5. *The variation of the thermal resistance R and R' for the masonry wall*

In the next stages it has been tried to deduce some mathematical expressions, as continuous linearly functions, that must express the corrected resistance variation with the thickness of the new thermal insulation, so that knowing the extreme values of the corrected resistance for the insulation thickness of 0 cm (no insulation) and for 12 cm, to be able to compute the corrected resistance for any intermediate thickness. The value of 12 cm has been chosen because it has been considered that in the case of the external walls represents a maximum practical value, for the thermal insulation of polystyrene, mineral wool, or other similar products.

A large number of possibilities regarding the types of variation have been tested: linear, second degree of conical curve, logarithmical, trigonometrically. The results have been compared with the values obtained by numerical modelling of each thickness variant (accepted as reference values R'_{ref}). In the end, the following possibilities have been retained.

a. Linear variation

The linear approximation of the corrected resistance variation values is simple: the variation curve is represented by the line portion that passed through the extremities A and B of the corrected resistance graphic R' (see Fig. 6); the equation of this line is given by the relation:

$$\frac{y(x)-y_1}{y_2-y_1}=\frac{x-x_1}{x_2-x_1} \quad\Rightarrow\quad \frac{R'_{lin}(d_{iz})-y_1}{y_2-y_1}=\frac{d_{iz}-x_1}{x_2-x_1} \quad\Rightarrow$$

$$\Rightarrow \quad \boxed{R'_{lin}(d_{iz})=\frac{R'_{ref}(0.12)-R'_{ref}(0)}{0.12}d_{iz}+R'_{ref}(0)} \qquad (2)$$

where: x_1, y_1 – A point coordinates ($x_1 = 0.00$; $y_1 = R'_{ref}(0)$;

x_2, y_2 – B point coordinates ($x_2 = 0.12$; $y_2 = R'_{ref}(0.12)$;

d_{iz} – the thickness of supplementary thermal insulation);

R'_{lin} – the corrected thermal resistance by linear approximation;

$R'_{ref}(0)$, $R'_{ref}(0.12)$ – the reference corrected thermal resistance for insulting layers of d_{iz} = 0 cm, respective d_{iz} = 12 cm.

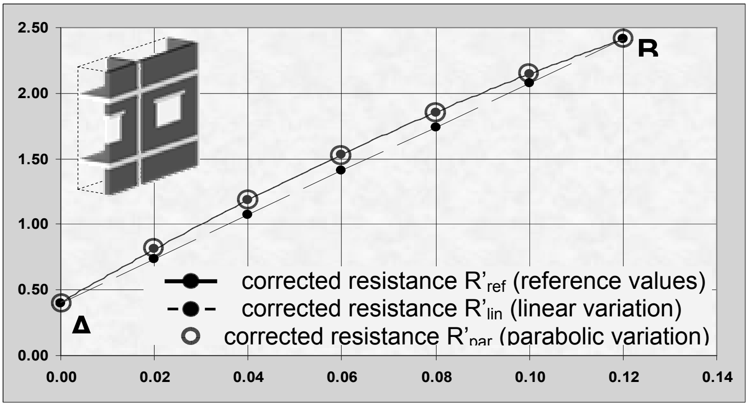

Fig. 6. *The approximation of the corrected thermal resistance*

The differences between the reference values of the corrected resistance and those approximated by accepting a linear variation are placed under 10%, with a mean of 5.27%. This difference is accepted for the practical calculus point of view. It must be mentioned that the values of approximated corrected resistance result lower than the reference values, in other words the calculus will be covering (sustainable).

b. Parabolic variation

The following law of variation have been considered :

$$y(x) = ax2 + bx + c \quad => \quad \boxed{R'_{par}(d_{iz})=a.d_{iz}^2+b.d_{iz}+c} \qquad (3)$$

where: $R'_{par}(d_{iz})$ – the corrected thermal resistance approximated by a parabolic variation, function of the thickness d_{iz} of the supplementary thermal insulation layer.

To determine the coefficients a, b, and c are necessary three conditions. The first two result from the fact that the parabola must pass through the two points from the extremities of the reference corrected resistance variation graphic (points A and B from Fig. 6).

For the third condition it has been imposed that the geometrical tangent to the parabola in the point A to have a certain slope „m”, approximately appreciated to be between the slope AB line (Fig. 6) and the slope in the point A of the unidirectional thermal resistance linear graphic (Fig. 6 superior graphic).

The differences between the reference values of the corrected resistance and the one’s approximated by parabolic variation, in the case of the masonry wall, are of a maximum value of 1.39%, with a mean value of 0.36 %.

The two types of variation, linear and parabolic, proposed for the specific corrected thermal resistance have been tested for many other enclosing elements constructive solutions.

Generally, the deviations registered don’t override 10%, in the case of linear approximation and 2% and in case of parabolic approximation, proving the methods applicability in practical calculus.

5. METHODOLOGY ALGORITHM

The methodology proposed has the following main stages:

a. The prominence of all the types of thermal bridges of the element or of the analysed building and specifying the characteristics of each bridge.

b. The determination of the linear ψ and punctual χ coefficients of thermal transfer, in two situations: the initial state of the building, without supplementary thermal insulation (d_{iz} = 0), and a theoretical state for which it is adopted an arbitrary thickness of the thermal insulation. The value of this thickness depends on the nature of the insulating material used and the element type that is computed (wall, floor etc), and it is appreciated on experience basis. In usual conditions, for materials of high thermal efficiency, like polystyrene, mineral wool etc, usually used in the case of modernising projects, it can be chosen an arbitrary thickness of 10...12 cm for the exterior walls, and 12...20 cm for the floor over last level or above the unheated basement.

c. On the base of the linear and punctual coefficients of thermal transfer, are computed two sets of corrected specific thermal resistance values, corresponding to the previous two presented situations. Those values has a function of pivot value, with them being able to determine the corrected thermal resistance for any value of the thermal insulating layer thickness between the minimum value (0 cm), and the one chosen arbitrary.

d. With the help of the pivot-values and applying the variation law of the corrected thermal resistance with the thermal insulation thickness, described at pct. 4, it is determined on a tabular base (EXCEL programme) the intermediate corrected thermal resistance and, at the end, the optimum dimensions for which the effective corrected resistance is superior, but very close to the value of the minimum resistance in the normative.

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NOVI SAD, NOVEMBER 22-24,2006.

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TRANSVERSE VIBRATION TESTING TECHNIQUES FOR ASSESSING PERFORMANCE OF WOOD FLOOR SYSTEMS

Summary: *The primary means of inspecting buildings and other structures is to evaluate each structure member individually. This is a time-consuming and expensive process, particularly if sheathing or other covering materials mast be removed to access the structural members. The objective of this study was to investigate the possibility of using transverse vibration testing techniques for assessing the performance of wood floor system by measuring the fundamental natural frequency. For the purpose of practical inspection, an analytical model based on flexural beam theory was proposed to represent the relationship between natural frequency and floor stiffness.*

Key words: *vibration, natural frequency, stiffness, wooden floor*.

PROCENA KARAKTERISTIKA DRVENIH TAVANICA UPOTREBOM POPREČNIH VIBRACIJA

Rezime: *Ispitivanje postojećeg stanja zgrada i ostalih konstrukcija podrazumeva procenu svakog elementa konstrukcije pojedinačno. Ovo je dugotrajan i veoma skup postupak, posebno ako se podna oplata i obloga moraju skidati radi adekvatne procene. U ovom radu se razmatraju mogućnosti upotrebe poprečnih vibracija za procenu globalnih karakteristika drvene tavanice, merenjem osnovne frekvencije oscilovanja. Za praktičnu upotrebu predložen je analitički model, zasnovan na teoriji savijanja grede, kojim se uspostavlja veza između frekvencije i krutosti tavanice.*

Ključne reči: *vibracija, frekvencija, krutost, drvena tavanica.*

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1. INTRODUCTION

The deterioration of wood structures often occurs internally before any external signs of damage appear. The load-bearing capacity of the affected member is greatly reduced before surface deterioration is visible. Determination of an appropriate load rating for an existing structure and rational decisions about rehabilitation, repair or replacement can be made only after an accurate assessment of the existing condition. Knowledge of the condition of a structure can reduce repair and replacement costs by minimizing labor and materials and extending service life.

In general, structural condition assessment requires the monitoring of some indicating that are sensitive to the damage or deterioration mechanism in question. Current inspection methods for wood structures are limited to evaluating each structural member individually, whish is labor-intensive and time-consuming process. For field assessment of wood structures more efficient strategy would be to evaluate structural systems or subsystems in terms of their overall performance and serviceability. From this perspective, examining the dynamic response of a structural system might provide an alternative way to gain insight into the ongoing performance of the system. Deterioration caused by an organism or physical damage to the structure reduces the strength and stiffness of the materials and thus could affect the dynamic behavior of the system. If, for example, one structural system or section of the system was found to respond to dynamic loads in a manner significantly different from that of other similar system or the surrounding sections of the system more extensive inspection of that system or section would be warranted.

Based on this conceptual strategy, explorers began to investigate the possibility of using a low frequency vibration approach for assessing the performance of wood structural systems by measuring the fundamental natural frequency (bending mode) and damping ratio of the entire system. This approach was investigated for two reasons. First, compared with other technique, the simplicity of this technique requires less experimental skill to perform field vibration testing. This fits the need of field inspectors who usually have little advanced training in structural dynamic testing. Second, the cost of testing a structure using the forced vibration method is very low compared with use of a modal testing method. Furthermore, because forced vibration is pure time domain method it eliminates the need for knowledge of modal analysis.

2. ANALYTICAL MODEL

The fundamental natural frequency was chosen as the indicator of global structure stiffness. For the purpose of practical inspection, an analytic model is needed for relating the fundamental natural frequency to the global stiffness properties of a floor system. Continuous system theory was chosen as the means for developing a theoretical vibration model based on the global physical properties of a system.

The wood floor systems in existing buildings are typically constructed of wood joists, decking and cross bridging (Fig. 1). We assume that the stiffness of the joists predominates over the transverse floor sheathing because the thickness of the decking boards is very small compared to the height of the joists. In addition, the cross bridging does not contribute to the bending stiffness of the floor because it mainly provides lateral bracing to the joints. Thus, a floor system behaves predominately like a beam with resisting moments in the transverse direction. The total mass of the deck and cross bridging is distributed into the assumed mass of the joists.

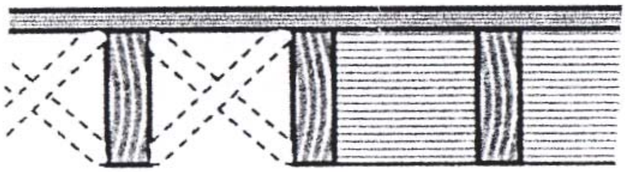

Figure 1- Typical wood floor construction

The partial differential equation governing the transverse vibration for a simple flexure beam is:

$$\frac{\partial^2 v}{\partial t^2} + \left(\frac{EI}{m}\right)\frac{\partial^4 v}{\partial x^4} = 0\,, \tag{1}$$

where
v is vertical displacement variable,
t time,
x variable along beam length,
E modulus of elasticity,
I moment of inertia of beam, and
m mass per unit length.

The solution of this partial differential equation is generally accomplished by means of the separation of variables and is largely dependent on the boundary condition at each end of the beam. General form for the natural frequency for any mode (i) can be derived as:

$$f_i = \frac{\lambda_i^2}{2\pi L^2}\left(\frac{EI}{m}\right)^{1/2}, \tag{2}$$

where
f_i is natural frequency (mode i),
λ_i a factor dependent on beam boundary conditions,
L beam span, and
EI stiffness of beam.

Consider the vibration of a beam supported at the ends. If vibration is restricted to the first mode, Eq. (2) can be rearranged to obtain an expression for stiffness (EI):

$$EI = \frac{f^2 W L^3}{kg}, \tag{3}$$

where f is the fundamental natural frequency (first bending mode), k is defined as a system parameter dependent on the boundary conditions of the beam (pin-pin support: k=2.46; fix-fix support: k=12.68), W is weight of the beam (uniformly distributed) and g is acceleration due to gravity.

3. EXPERIMENTAL PROCEDURE

The floor systems were subjected to both free and forced vibration. Free vibration was initiated by impact from a hammer. Forced vibration was imposed by a motor with an eccentric rotating mass attached to the floor decking. Motor speed could be manually changed to maximum of 1800 rpm. The rotating mass weighed 251 g with an eccentricity of 3 cm. The response to vibration was measured at the bottom of joists using a linear variable differential transducer (LVDT). The time-deflection signal was recorded by oscilloscope. A schematic diagram of the experimental setup is shown in Fig. 2. For free vibration the damped natural frequency was determined as inverse of the period measured from the time-deflection signal (Fig. 3). For forced vibration the damped resonant frequency was determined by increasing motor speed until the maximum deflection resonance was observed and then measuring frequency from the time-deflection signal.

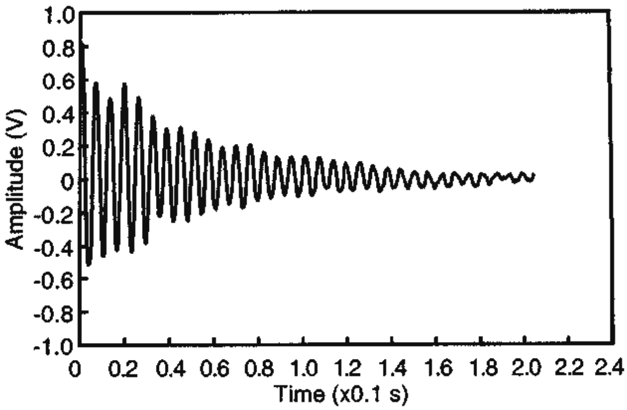

Figure 2- Experimental setup for forced vibration testing of wood floor system [3]

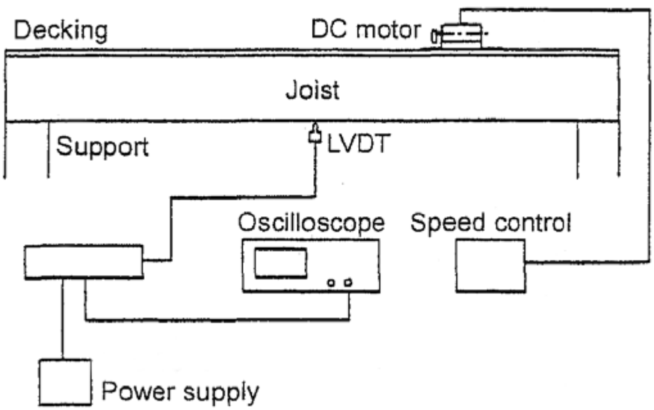

Figure 3- Example response of amplitude as a function of time for free vibration [6]

4. RESULTS OF INVESTIGATION

Recent cooperative research efforts of the USDA Forest Service, Forest Products Laboratory, Michigan Technological University and University of Minnesota-Duluth have resulted in significant progress in developing global dynamic testing techniques for nondestructively evaluating the structural integrity of wood structures systems. In these studies, a series of laboratory constructed wood floor systems and several in-place wood floor structures were examined and addressed practical problems on the use of vibration methods for floor inspection.

The first problem was related to the best way to obtain a good signal response when inspecting a floor with limited accessibility. Investigation showed that the location of the response measuring device and forcing function does not significantly affect frequency. Both free and forced vibration gave acceptable result. Free vibration has the advantages of being easy to apply. Its disadvantage is that the response is sometimes weak. Forced vibration enables a stronger response by use of a larger forcing function. It also appears to give more consistent results. However, an error could occur when other modes (typically torsion) were misidentified as the bending mode.

The second problem was whether vibration testing can be used to detect joist decay. The results have indicated a decrease in natural frequency, as simulated by progressively cutting the ends of three joists (each laboratory floor had five joists). Small changes in frequency was observed with the loss of one or two joist ends, but greater change was observed with the loss of three joist ends. This implies that the system effect of a floor with bridging and decking may make it difficult to detect decay in only one or two joists. The question is whether deterioration limited to one or two joist in a floor system has a significant effect on structural integrity, warranting repair of the floor.

The third problem was to inspect a floor with superimposed loads that are not easily removed (file cabinets, furniture, refrigerators). The additional mass of the loads should be included in frequency prediction calculations, but the location of the loads has a small effect on natural frequency. The effect of a superimposed load was larger for the floor system with cut end joists than for the undamaged floor.

A final problem was to determinate boundary conditions. The true boundary conditions in real floor structures cannot be absolutely known from visual inspection of the floor or floor plans. However, laboratory floor systems can provide an opportunity to investigate how the floor response under a forcing function is affected by different end conditions, from nearly free to the condition that approximates a real floor. The hardness of end supporting materials has little or no effect on the natural frequency of a floor. In contrast, the masonry pocket end supports, which simulate the end conditions of typical floor structures in existing buildings, yield a higher frequency than do pinned end supports.

The analytical model generated from the simple beam theory fits the physics of the floor structures investigated and has a potential to be used to correlate the natural frequency to *EI* product. However, for the model to be applied to floor inspection, it needs to be calibrated with field data from in-place floor systems.

5. CONCLUSION

The transverse vibration testing techniques has the potential to be used in field to quickly assess performance of wood floor systems. To improve the reliability of this method, more comprehensive mathematical models will be developed to quantify the sensitivity of floor response to various environmental factors. A new research project is necessary to further investigate key issues in vibration modes and boundary conditions and to refine field testing techniques and instrumentation systems. Also, future research is needed to determine whether similar results can be obtained for a wide range of floor spans and joist sizes.

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SEISMIC-RESISTANT STEEL STRUCTURES WITH REMOVABLE DISSIPATIVE ELEMENTS

Summary: *Eccentrically braced steel frames represent a suitable solution for multi-storey buildings located in seismic areas. A bolted connection between the dissipative element (link) and the beam is suggested to facilitate replacement of damaged dissipative elements (links) after a moderate to strong earthquake, which reduces repair costs. Influence of connection flexibility on the design and seismic performance of structures incorporating removable dissipative elements is investigated. Seismic response of structures is investigated using nonlinear time-history analysis under two different sets of seismic motions. Dual structural configurations are considered as a mean to reduce permanent drifts in order to facilitate replacement of dissipative elements.*

Key words *Eccentrically braced frames, removable links, seismic performance.*

SEISMIČKI OTPORNE ČELIČNE KONSTRUKCIJE SA POMERLJIVIM DISIPATIVNIM ELEMENTIMA

Rezime: *Ekscentrično postavljeni spregovi čeličnih okvira predstavljaju uspešno rešenje za višespratne zgrade locirane u seizmičkim područjima. Veze između disipativnih elemenata (link) i greda ostvarene zavrtnjevima omogućava uklanjanje - premeštanje oštećenih disipativnih elemenata konstrukcije posle umerenih i snažnih zemljotresa, čime se redukuju troškovi sanacije. Uticaj fleksibilnosti veza u projektovanju i seizmičke performanse konstrukcije, sa pomerljivim disipativnim elementima, su istraženi. Seizmički odgovor konstrukcije je određen nelinearnom metodom vremenske analize, sa dva različita seta seizmičkih zapisa. Dvojne konfiguracije konstrukcije su razmatrane da bi se redukovale vrednosti stalnih relativnih spratnih pomeranja da bi se zamenili elementi disipativnih zona.*

Ključne reči: *Ekscentrično ukrućeni okviri, pomerljive veze, seizmičke performanse*

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1. INTRODUCTION

Design of multi-storey structures in high-seismicity areas is usually based on dissipative structural response, which accepts significant structural damage under the design earthquake. It is believed however, that design criteria specified in modern seismic codes will prevent structural collapse, ensuring life safe. The earthquakes of Loma Prieta (1989), Northridge (1994) and Hyogoken-Nanbu (1995) showed that generally, modern structures behaved as expected. However, the unexpectedly high economic losses following these earthquakes urged for a limitation of damage to structures in future earthquakes, leading to the development of Performance-Based Design (PBD), Hamburger, 1996 [1]. Its objectives include minimizing structural and non-structural damage under low and moderate earthquake intensities, which is equivalent to reduction of the total cost (initial and repair).

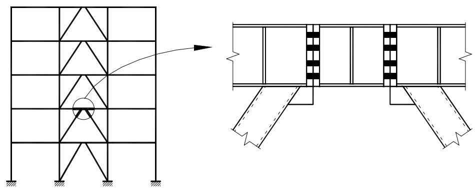

Figure 15. Bolted link concept.

On the other hand, capacity-based design, applied in most of the current seismic design codes allows design of structures that promote plastic deformation in predefined areas only, called dissipative zones. In the case of a bolted connection between dissipative zones and the rest of the structure it is possible to replace the dissipative elements damaged as a result of a moderate to strong earthquake, reducing the repair costs. Application of this philosophy to eccentrically braced frames, where link elements serve as dissipative zones, is presented in Figure 15. The connection of the link to the beams is realized by a flush end-plate and high-strength bolts. Bolted connection allows he link element to be fabricated from a lower-yield steel grade, assuring an elastic response of the elements outside removable link element. This system may be applied to both homogeneous structures (eccentrically braced frames alone) and dual ones (eccentrically braced spans combined with moment-resisting spans). Extended end-plate bolted connections for eccentrically braced frames with link-column connection configuration were previously suggested and investigated experimentally by Ghobarah and Ramadan, 1994 [4]. Their inelastic performance was found to be similar to fully-welded connections. Recently, Mansour et al. [6] investigated replaceable shear links composed of two bolted back-to-back channels.

2. DESIGN AND MODELLING OF STRUCTURES

The EBF-MRF dual structure, termed DUA in the following was designed according to Eurocode 3 (1993) and Eurocode 8 (1994). A 4.75 kN/m^2 dead load on the floors, 1.70 kN/m^2 for exterior cladding and 3.0 kN/m^2 live load were considered. Seismic design parameters were: 0.35g peak ground acceleration, stiff soil conditions (class A), a behaviour factor q=5.5, and interstorey drift limitation of 0.006 of the storey height. Capacity design according to Eurocode 8 governed dimensioning of the structure. Dimensions of structural elements were the same for both dual and homogeneous structural configurations. HEB260 S235 columns, IPE330 S355 beams in the outer bays, IPE240 S235 links and beams in the middle bay, and RHS 120x120x(7.1-12.5) were used. A short link whose behaviour is governed by shear only was considered.

A series of pushover and time-history analyses were carried out using the Drain-3dx computer program. The inelastic shear link element model proposed by Ricles and Popov [8] was used, slightly adapted to the trilinear envelope curve available in Drain-3dx. Beams, columns and braces were modelled with fibre hinge beam-column element, with plastic hinges located at the element ends.

3. SEISMIC RESPONSE

A standard EBF frame (EBF) with pinned beam to column joints in the outer bays was considered as an alternative to the dual frame (DUA). Dimensions of the structural shapes of the EBF structure are identical to those of the DUA structure. Fundamental period of vibration of the two structures were 0.58 s for DUA and 0.64 s for EBF.

3.1. Seismic action

Two sets of ground motions each containing seven records were used. Both sets of records are representative for Vrancea seismic source, but different soils conditions: stiff soil (T_C=0.5 s) and weak soil (T_C=1.4 s). The two target spectra (for stiff and weak soils) were scaled to match the same spectral acceleration in the 0.58-0.64 second period range as the EC8 spectrum used in design. As a result, peak ground accelerations of 0.28g and 0.23g resulted for the T_C=0.5 and T_C=1.4 sets of records, respectively, corresponding to 0.35g peak ground acceleration for the EC8 type A spectrum used in the design. This procedure assures approximately the same design earthquake forces as the one used in the initial design using the E8 design spectrum. While the T_C=0.5 set of records consisted in recorded accelerograms only, the T_C=1.4 set comprised both recorded and semi-artificial accelerograms.

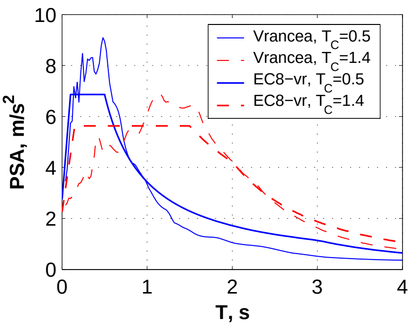

Figure 16. Target pseudo-acceleration spectra and average spectra of the two sets of accelerograms.

3.2. Performance levels

Three Performance levels were considered: serviceability (SLS), ultimate limit state (ULS), and collapse prevention (CP). Earthquake intensity at the serviceability limit state was characterised by a ground motion multiplier λ = 0.5 (corresponding to ν = 0.5 in Eurocode 8), while the one for the collapse prevention limit state was considered λ = 1.5 [2]. Intensity of earthquake action at the ultimate limit state was the design one (λ = 1.0).

Plastic deformations of links in eccentrically braced frames have an excellent ductility (Kasai and Popov [6]). Accepted plastic shear deformations range from 0.08 rad (AISC 2002 [1]) and 0.11 rad (FEMA 356 [3]). Ultimate link deformations of γ_u=0.1 rad were used in this research. Ultimate plastic rotation capacities of flexural members (beams, columns, connections) were considered equal to θ_u = 0.03 rad. Criterion for attainment of the ultimate limit state (ULS) was considered attainment of ultimate plastic deformations (γ_u, θ_u) in elements. Performance level corresponding to serviceability limit state was considered as the attainment of 0.006h interstorey drift, while the one for the collapse prevention – dynamic instability.

3.3. Pushover analysis

A pushover analysis was performed first (Figure 17), under two lateral load distributions (inverted triangular and uniform), while displacement demands were evaluated by the N2 method (Fajfar [2]). The EBF structure has a slightly lower stiffness than the dual one. However, both structures show similar base shears at the first plastic hinge, which results that design is governed by the properties of the eccentrically structure only. The difference between the two structures is more evident in the post-elastic range, the DUA structure being characterised by larger overstrength and hardening. Displacement demands obtained using the N2 method for an earthquake intensity corresponding to the ultimate limit state (λ=1.0) are larger for the EBF structure, especially in the case of the T_C=1.4 spectrum. Links were the elements that attained first the ultimate plastic deformations (ultimate limit state criterion).

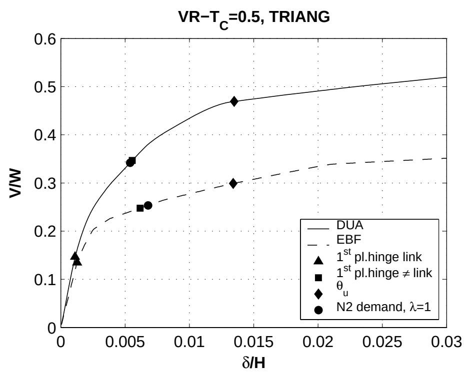

Figure 17. Normalised base shear force versus normalised top displacement for the inverted triangular load pattern.

3.4. Time-history analysis

Interstorey drifts at SLS are less than the performance level (0.006 rad), being between 0.0023 and 0.003 rad. Only links yielded at SLS. Elements outside link yielded later in the DUA structure.

ULS (λ=1) performance is accomplished by both structures. Maximum link shear distortions are presented in Table 3. Maximum interstorey drifts are below 0.006 rad limit with the exception of one case, indicating limitations of non-structural degradation even at the ULS. Ground motions from the T_C=1.4 set generated higher drift and element deformation demands. Effect of structural configuration was minor in the case of T_C=0.5 accelerogram set, but were important for T_C=1.4, dual configuration reducing both top displacements and inter-storey drifts. Distribution of drifts for λ=1 is presented in Figure 18. Though in both structural configurations displacement demands concentrate in lower storeys, dual structure is characterised by more uniform distribution over the eight of the building. The same trend is observed from local link deformation demands.

Accelerogram multiplier λ_u at the attainment of ULS criteria (generally governed by ultimate link deformations) was equal to 4.55 and 4.47 for the DUA and EBF structures under the T_C=0.5 set of ground motions, and 2.15 and 1.72 for the DUA and EBF structures under the T_C=1.4 set of ground motions. Safety level is high in the case of T_C=0.5, but decreases for the T_C=1.4 set of accelerograms. Dual structural configuration improves structural performance, but this effect is important only for the T_C=1.4 set of accelerograms.

structure	γ_{link}, rad		δ/H		IDR_{max}, rad		IDR_{per}, rad	
	T_C=0.5	T_C=1.4	T_C=0.5	T_C=1.4	T_C=0.5	T_C=1.4	T_C=0.5	T_C=1.4
DUA	0.020	0.024	0.0031	0.0036	0.0050	0.0059	0.0002	0.0011
EBF	0.018	0.034	0.0034	0.0051	0.0056	0.0091	0.0002	0.0032

Table 3. Link deformation (γ_{link}), top displacement δ/H, maximum (IDR_{max}) and permanent (IDR_{per}) inter-storey drift demands for λ=1.

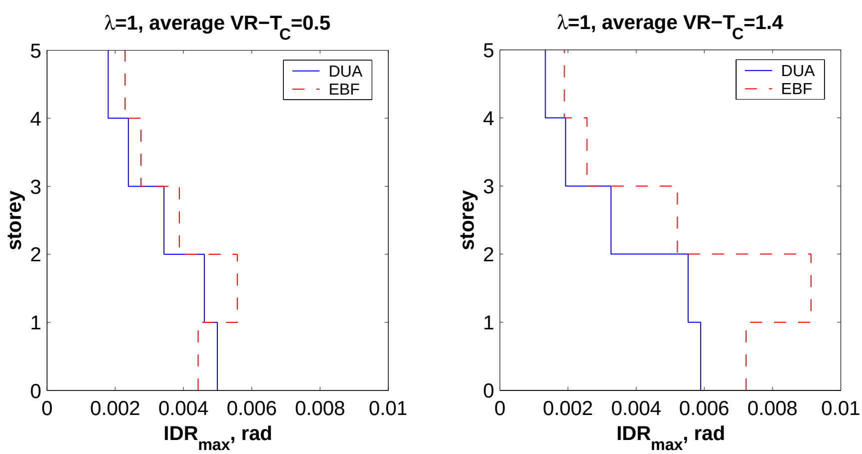

Figure 18. Distribution of inter-storey drift demands along the height, λ=1.

Dynamic instability was not detected for accelerogram multipliers less than λ=7 for the T_C=0.5 accelerogram set. In the case of T_C=1.4 accelerogram set there is much higher probability of dynamic instability. Dual structural configuration has an enhanced performance from this point of view.

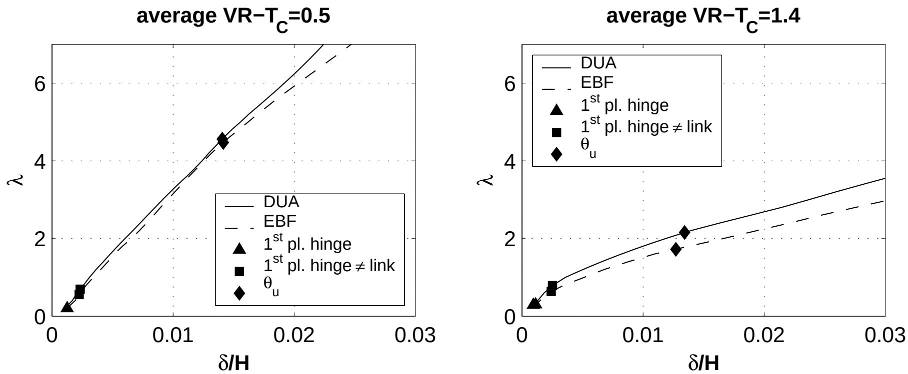

Figure 19. Incremental dynamic analysis curves for the T_C=0.5 and T_C=1.4 sets of accelerograms.

A comparison of top displacement demands in the form of incremental dynamic analysis (IDA) curves is presented in Figure 19. The dual configuration has two effects on the structural response. The first one is related to reduction of top displacements due to higher hardening of the global force-displacement curve. This effect is important for T_C=1.4 accelerogram set only. It is not important for the T_C=0.5 accelerogram set, because the structure response is governed by the "equal displacement" rule, when elastic and elastic-perfectly plastic behaviour yield the same seismic displacement demands. As systems with hardening behaviour should have intermediate behaviour between the two limits (elastic and elastic-perfectly plastic), hardening is not relevant in this case. The second effect is the reduction of interstorey drift demands by the dual configuration which is mainly related to a more uniform distribution of inter-storey displacements over the height of the structure. This effect is independent of the ground motion characteristics.

In addition to maximum transient drifts, the dual configuration is efficient in reducing permanent displacements of the structure, but only for displacement demands that do not cause yielding in structural elements outside links (beams, braces and columns). It is to be noted that degradation of the beam, columns and braces was minor for λ=1.0-1.5 in the case of the DUA frame.

4. SEMI-RIGID CONNECTIONS

Results presented in section 3 were based on the assumption of a continuous beam in the link zone. A flush end-plate connection can be used to connect the link to the rest of the beam, obtaining a removable dissipative element. Such a configuration was tested experimentally by Stratan and Dubina [9] and showed to be a feasible solution for short links. However, flush-end plate connections are generally semi-rigid and partial resistant. Therefore it was necessary to assess the influence of connection characteristics on the global response of the eccentrically braced frame. The same connection type that was used in the experimental program (Figure 20) was considered as part of the dual eccentrically braced frames. Its characteristics in terms of stiffness and strength were determined using Eurocode 3, using characteristics properties of connection elements. The initial stiffness amounted to $S_{j,ini}$=64023 kNm/rad, while the moment resistance was $M_{j,Rd}$=64.6 kNm. Even with respect to the short length of the link, this connection classifies as a semi-rigid connection, when compared to the $8EI_{beam}/e$ limit for braced frames (where e is the link length), which amounted to 163646 kNm/rad. The connection is partial resistant as well ($M_{pl,beam}$=86.2 kNm).

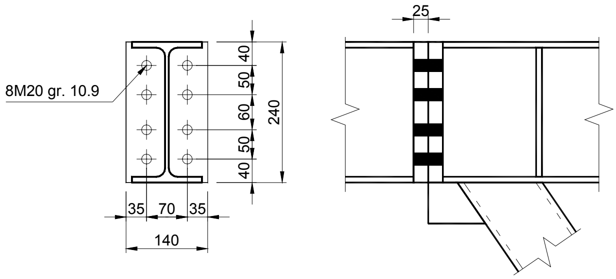

Figure 20. Geometrical characteristics of the link connection.

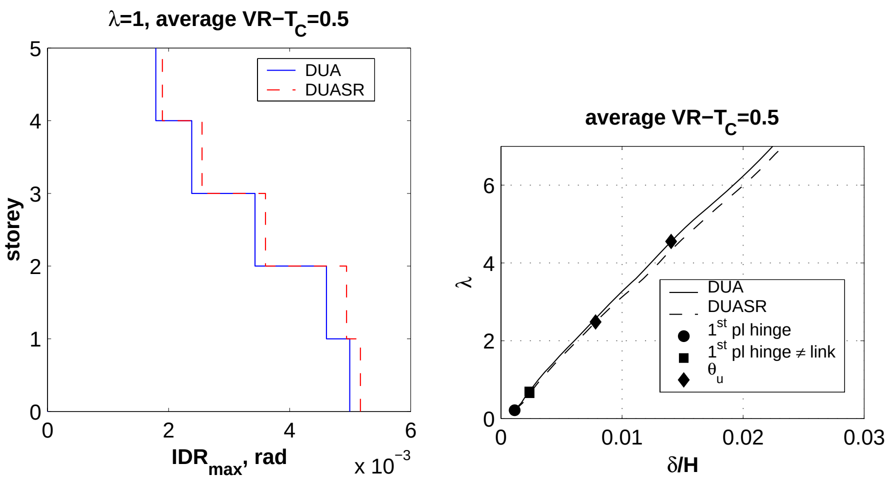

Figure 21. Distribution of inter-storey drift demands along the height, λ=1 and Incremental dynamic analysis curves for the DUA and DUASR structures.

A bilinear model of the connection was incorporated into the structural model, and the structure was analysed using pushover and time-history analyses. Considering that plastic deformations were expected in connection elements, a reduced "secant" stiffness of $S_{j,ini}/3$=21341 kNm/rad was used in the analyses. For the DUA structure, the fundamental period of vibration increased only slightly from 0.58 to 0.60 seconds due to semi-rigid connections considered in the DUASR frame. As it can be observed from Figure 21 overall structural performance was not much affected by the semi-rigid and partial resistant connection response. Interstorey and top displacement demands were slightly higher in the case of the semi-rigid connection response (DUASR structure). While in the case of rigid connections, the ultimate limit state criteria were predominantly due to attainment of ultimate plastic deformations in link elements, in the case of semi-rigid beam to link connections, ultimate plastic rotation of connections occurred was the governing criterion.

5. CONCLUSIONS

A removable link for eccentrically braced frames and dual (eccentrically braced frames combined with moment-resisting frames) is suggested. Previously reported experimental investigations proved feasibility of this solution. In the present paper seismic performance of homogeneous and dual structural configurations were investigated under two sets of ground motions. It was shown that ground motion characteristics affect in a greater extent structural response than the homogeneous/dual configuration. However, dual structural configurations have beneficial effects on seismic response. These effects include more uniform transient drift demands along the height of the structure and reduced permanent drifts. Beam to link connection flexibility and reduced moment capacity does not affect seriously global response of the structure. However, damage to the connection should be reduced, in order to allow for replacement of dissipative links.

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TENTH NATIONAL AND
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iNDiS 2006

NOVI SAD, NOVEMBER 22-24,2006.

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ANALYSIS OF ESSENTIAL THERMO-PHYSICAL PROPERTIES OF SANDWICH PANELS WITH METALLIC FACINGS

Summary: *The sandwich panels analyzed in the paper are structures made up of two profiled sheets between which a polyurethane rigid foam is injected. These panels are used in civil and industrial construction for cladding elements in both wall and roof systems. In the first part of the paper the thermal performances of sandwich panels using the ISOVER program are evaluated. Since the thermal inertia of these panels is small the thermal performances are evaluated in a non-steady regime. Therefore two dynamic thermal parameters, namely damping and phase delay should be calculated. The global thermal insulation coefficient in a multifunctional building is determined according to C107/1-'05 Romanian norms. The acoustic study of sandwich panels involves the sound transmission characterization evaluation in order to achieve an insulating efficient element. Several acoustic features are theoretically analyzed and computer simulation using WinFLAG program is performed to study the sound transmission loss.*

Key words: *sandwich panels, cladding elements, wall and roof systems, thermal and acoustic performances, global thermal insulation coefficient, sound transmission loss.*

ANALIZA OSNOVNIH TERMO-FIZIČKIH SVOJSTAVA SENDVIČ PANELA SA METALNOM OBLOGOM

Summary: *Sendvič paneli, koji su analizirani u ovom radu, su konstrukcije napravljene od dva profilisane ploče između kojih je injektirana kruta poliuretanska pena. Ovi paneli se koriste za gradnju kao obloga u sistemima zidova i krovova. U prvom delu rada ocenjene su termičke karakteristike sendvič panela korišćenjem ISOVER programa. Budući da je termička inercija ovih panela mala termička svojstva su ocenjena u nestabilnom režimu. Zbog toga su dva dinamička parametra, vla.nost i fazno kašnjenje, morali biti sračunati. Opšti termo-izolacioni koeficijent u multifunkcionalnim objektima je određen prema C107/1-'05 Rumunskim normama. Akustička istraživanje sendvič panela obuhvatilo je ocenu karakterizacije prenosa zvuka da bi se postigao izolaciono efikasan element. Teorijski je analizirano nekoliko akustičnih svojstava i sprovedena je kompjuterska simulacija primenom WinFLAG programa da bi se istražio gubitak zvuka pri prenosu.*

Key words: *sendvič paneli, elementi obloge, sistemi zidova i krovova, termička i akustička svojstva, opšti koeficijent termičke izolacija, gubitak zvuka pri prenosu.*

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1. INTRODUCTION

The sandwich panels for walls and roofs are layered products made of two lightly profiled sheet metal faces in which rigid polyurethane foam isolation is injected. There are some favourable characteristics that make these building extremely advantageous for roof and wall system cladding:

- their self weight is reduced, (table 1), so they can be easily transported and handled during the assembly; also the permanent loading is small compared to that given by other cladding elements;
- the thermal transfer coefficient of polyurethane foam very small compared to almost all other building materials provides the panel a high thermal resistance;
- the rigid polyurethane foam has 98% of its cells closed, which insures its impermeability to air and humidity;
- the behaviour in fire conditions is exceptionally good since thanks to its metallic facings a significant heat dissipation takes place and the polyurethane foam extinguishes itself in about 10 seconds (Class C2 for combustibility);
- the sandwich panels made of profiled metallic facings and polyurethane foam cores have enough in-plane stiffness to provide good lateral rigidity for cladding system.

The profiled sheet metal can be made of galvanizing steel metal sheets with a 0.4÷0.5mm thickness, from aluminium or stainless steel. The aluminium sheet metal is made of Al 99.5, flat or corrugated, of STUCCO (tension resistant) or of Al-Mg (semi-stiff) alloy material having a 0.6÷1.2mm thickness.

The sandwich panels of this type are utilized for the buildings that need closure elements with a reduced self-weight and with thermal isolation properties and of noise attenuation. The sound attenuation depends on the propagation environment, on the sound frequency and it is due to the absorption, refraction and reflection phenomena.

2. THE EVALUATION OF HEAT LOSS

The total heat loss from the dwelling buildings are expressed by the thermal isolation global coefficient (G), whose calculation method is determined by the C107/1-'05 normative, [4]. The thermal isolation global coefficient takes into account:

- the heat loss caused by thermal transfer corresponding to all the perimeter surfaces that determine the heated volume of the building;
- the heat loss due to normal conditions of internal air refreshment;
- the supplementary heat loss due to the exceeding infiltration of the external air, through the jointing line.

The technical standards also comprise the maximum rated values of the thermal isolation global coefficients (GN) admitted for dwelling buildings. The level of global thermal isolation is sufficient if the $G \leq GN$ condition is performed, [4].

Roof panels			Wall panels		
Core thickness [*mm*]	Self-weight for panels, [Kg/m^2]	Thermal transfer coefficient, [W/m^2K]	Core thickness [*mm*]	Self-weight for panels, [Kg/m^2]	Thermal transfer coefficient, [W/m^2K]
-	-	-	25	7.70	0.77
-	-	-	30	7.89	0.65
-	-	-	35	8.08	0.56
40	9.27	0.58	40	8.27	0.50
50	9.65	0.46	50	8.65	0.41
60	10.03	0.38	60	9.03	0.34
80	10.79	0.33	80	9.79	0.26
100	11.59	0.25	100	10.59	0.21
-	-	-	120	11.35	0.18

Table 1. The characteristic of the analysed sandwich panels

3. ANALYSIS OF THERMAL PERFORMANCES OF THE SANDWICH PANELS

This paper presents a case study performed on a multifunctional building (industrial hall on the ground floor and officers on the first floor), with the help of the ISOVER program. The walls and the roof are made of sandwich panels with profiled metallic facings and rigid polyurethane foam core, (figure 1 and 2).

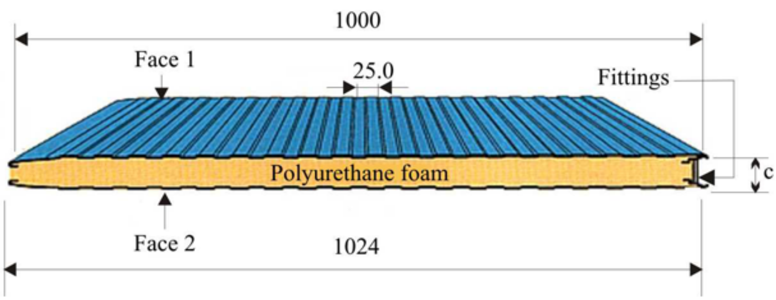
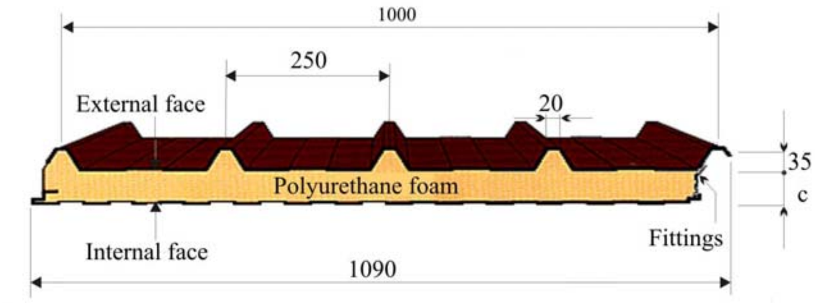


Figure 1. Sandwich panel for wall



(Dimensions in mm)

Figure 2. Sandwich panel with deep profiles for roofs

The program consists of two modules for the hygrothermal study:

- the design of the building thermal performances, that enables the evaluation of the following characteristics:

  − the average thermal resistance with and without consideration of the influence of thermal bridges;
  − the coefficient for heat loss corresponding to the area and volume;
  − the net coefficient for heat loss corresponding to the area and volume;
  − the energy consumption for heating during the cold season, reported to the area and volume;
- the thermal design of the buildings elements, that enables:

  − the design of the thermal transfer coefficient (U) and the temperature distribution given the condition of the stationary thermal regime;
  − the distribution of the partial pressures of the water vapour through Glaser method and the condensation risk on the internal surface of the external construction elements or in their structure;
  − the evaluation of the annual humidity balance, [5].

The heat loss for a multifunctional building, with a height regime *P+1E* (ground floor and one level), with a metallic framing system (column and girder beam), *12 cm* monolithic concrete plates, the external walls and the roof made the sandwich panels, (figure 3) has been determined. The building's ground floor is meant to belong to an industrial hall while the first floor is meant to belong to an administrative headquarters (offices). The building has a span of $L=9\ m$, 5 bays with $t=5\ m$, the height at the eaves level of $H=7\ m$, and the intermediary plate quota is at $+4.0\ m$. The first bay of the building is free, so the intermediary plate is developed only on four bays from the five of the strength structure. The slope angle of the ridged roof is $\alpha=7^o$.

The specific dimension and the thermal characteristic corresponding to the sandwich panels, used to realize the envelope of the studied building are presented in table 1.

The sandwich panels are lightweight building elements, with a good thermal resistance, and aesthetically attractive but due to their reduced self-weight, they have a limited thermal inertia (the damping factor is smaller than *1.6*). Therefore, the overheating phenomenon may occur during the warm season, [1].

The heat losses have been determined with the help of the ISOVER program. The value of rated heat loss, determined in the normative, is equal to *0.77*, diminished with *10%* (in case of new buildings), and the rated coefficient of thermal isolation is $G_N=0.7$.

The heat loss calculated for sandwich panels, with metallic external faces and core made of high performance insulation materials from the thermal isolation point of view, that have a thickness of $c \geq 80$ *mm*, is less than the value prescribed by the actual normative. Consequently, in the case of the studied building there is no overheating phenomenon that could lead to a major thermal discomfort, [2].

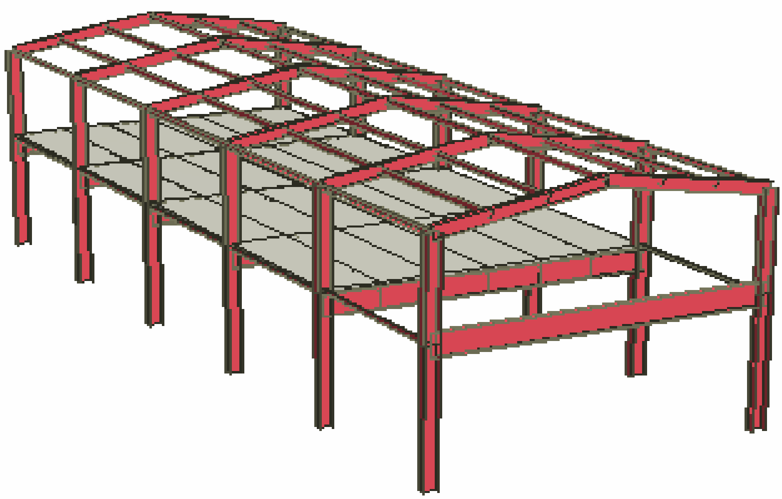


Figure 3. The framing system of the studied building

4. ANALYSIS OF ACOUSTIC PERFORMANCES OF THE SANDWICH PANELS

The design of a sandwich panel with a high acoustic insulation implies the identification of the following variables: the sources of noise, the sound's way (aerial, structural), the frequency of the sound and the geometry of the layered structure penetrated by sound, [3].

In case of low frequencies (less than *500 Hz*), the sandwich panels tend to vibrate and become resonant structures. The most certain way to prevent the resonance is achieved stiffening of the layered structure by using a material with a high density and a greater thickness for the core.

At intermediate and high frequencies (more than *1000 Hz*), the panels should be less rigid, so that the diminishing of the sound energy during the crossing of the panels is higher. The attenuation of the high frequencies is realized by using a material with a low density for the core, or an elastic material for the external layers (facings).

In order to improve the acoustical properties of the sandwich panels it is necessary to identify their resonant frequency, which is determined according to the geometric features of the panel. Generally speaking, it is almost impossible to control the size of the sandwich panels, taking into account a good acoustical behaviour, [1].

In the case study presented in the paper, the sound reductions are calculated for the sandwich panels utilized for wall and roof cladding with the characteristics given in table 1. The analyzed panels have the facings made of steel, or of Al, with *0.6 mm* and *1.2 mm* thickness.

It was following through the variation of the sound reduction according to: the core thickness (figure 4), the external layers thickness (figure 5) and the nature of the material from the external layers (figure 6).

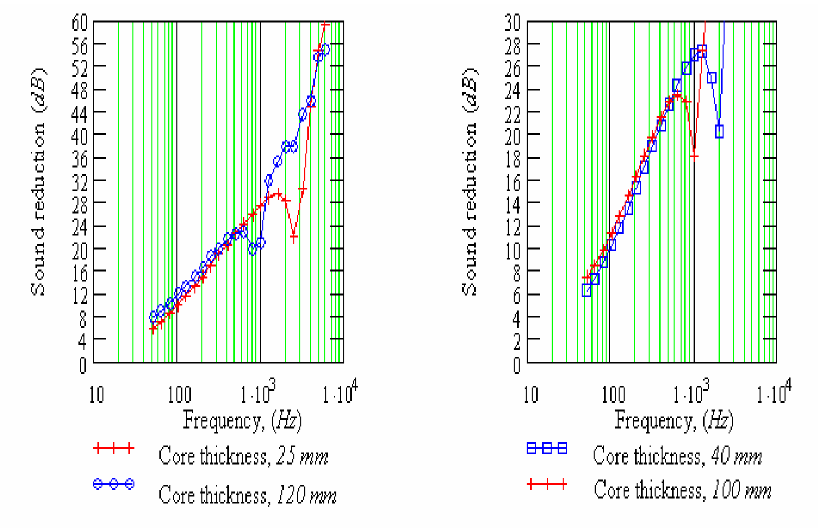


Figure 4. The variation of the sound reduction for sandwich panels, with steel facings 0.4 mm thick and polyurethane foam core, with various thickness

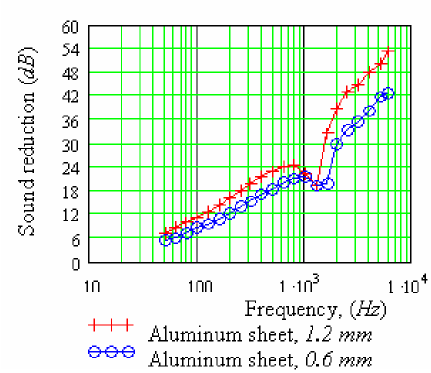


Figure 5. The variation of the sound reduction for sandwich panels, with different thickness faces

The WinFLAG 2.0 software package has been utilized to evaluate the sound absorption and sound reduction for various building making-ups (sandwich panels, perforated and micro perforated panels), [6].

Figure 5 presents the variation of the sound reduction for a sandwich panel with 0.6 mm and 1.2 mm Al facings respectively. Again the increase of the sound reduction with the faces thickness and the movement of the resonance frequency with 1/3 octave can be noticed. In the area of high frequencies, the rise of the sound reduction is more significant.

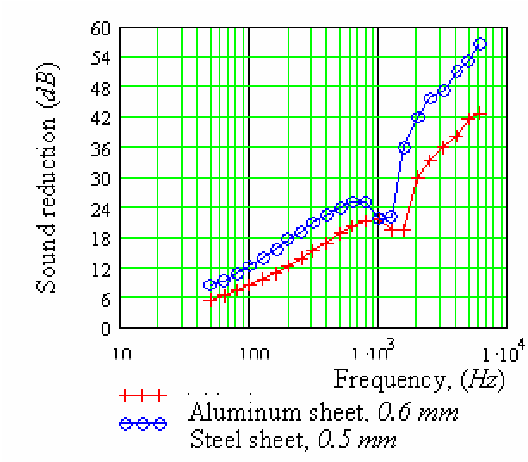


Figure 6. The variation of the sound reduction according to the nature of the face material

The behaviour of the sandwich panel according to the nature of the face material is illustrated in figure 6. A superior sound reduction in the case of steel face panel, even for smaller thickness, in comparison with the panel that has the external layers made of *Al* can be noticed. Also, it can be noticed that the sound reduction is more significant in the higher frequency field of study.

5. CONCLUSIONS

The ISOVER program enables the hygrothermal design of the building envelope made of sandwich panels in the first stage of design. At the beginning of the solution selection, the building cladding elements are dimensioned from hygric and thermal point of view, and then the heat loss is determined, at the values that might occur during the exploitation period. It must be underlined that the thermo-technical checking demanded by the present legislation includes: the checking of the minimum resistance that is necessary to the thermal transfer and the checking of the risk for the condensation phenomenon possible to affect the building elements that make up the building envelope. The sandwich panels with metallic facings and polyurethane foam core are frequently used for envelope of the industrial buildings, where the level of the noise produced by the equipment used in the technological processes is high. When the sound attenuation is an important demand, when choosing a panel we have to take into account the frequencies field of study, the nature of the sound source and the noise way.

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TENTH NATIONAL AND
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iNDiS 2006

NOVI SAD, NOVEMBER 22-24,2006.

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FILLER BEAM DECK - SOLUTION FOR TWO DIFFERENT BRIDGES

Summary: *The paper presents the utilization level of this type of structure for bridges and overpasses, on the road network from the west side of Romania. The main and typical characteristics of a finished structure and of other two designed structures will be presented. Because of many advantages, the filler beam decks represents a good alternative for the classical presstressed concrete superstructures. In order to use this kind of superstructure in the design of bridges and overpasses for the Romanian road and motorway network, our design team has finished the afferent documentation for some over passing structures, each having different parameters and specific functionality. From those we are mentioning as follows: a discharging viaduct in the major bed of the Timis river at Albina, nearby Timisoara and a railway overpass at Gataia, Timis county. For the two above mentioned works characteristic constructive parameters will be presented, which show their resemblance and the contrast at the same time.*

Key words: *filler beam deck, viaduct of discharge, overpass, rehabilitation*

ISPUNA KOLOVOZNIH GREDA - REŠENJE KOD DVA RAZLIČITA MOSTA

Rezime: *U radu je prikazano korišćenje ispune između greda kolovoza kod mostova i nadvožnjaka na mreži puteva Zapadne Rumunije. Prikazane su najvažnija tipične karakteristike završene konstrukcije i za dve projektovane konstrukcije. Zbog mnogih prednosti, ovakva kolovozna konstrukcija predstavlja dobru alternativu klasičnim prethodno napregnutim betonskim rasponskim konstrukcijama. Da bi se koristile ove rasponske konstrukcije pri projektovanju mostova i nadvožnjaka mreže puteva i autoputeva u Rumuniji projektantski tim je uradio dokumentaciju za neke konstrukcije nadvožnjaka, pri čemu svaki ima različite parametre i posebnu funkciju. Iz toga formulisan je stav: uklanjanje vijadukta u glavnom koritu reke Timiš u Albini, blizu Timišvara i železničkog nadvožnjaka u mestu Gataia u Timis pokrajini. Za dva pomenuta objekta su prikazani konstrukcijski parametri, koji istovremeno pokazuju njihovu sličnost i razlike.*

Ključne reči: *ispuna kolovoznih greda, zamena vijadukta, nadvožnjak, rehabilitacija*

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1. ALBINA OVERPASS

The structure, made in the first decades of the last century, is set on a county road, near Timisoara and assures the continuity of this road over the right major bed of the Timis river, between the localities Albina and Cheveresul Mare, playing the role of a viaduct for discharging high floods.

The statically system of the viaduct is a frame with four spans as follows 16,50 m + 2 x 2,50 m + 16,50 m (Fig. 1). The cross section consists of four reinforced concrete beams (C6/7,5) having a width of 30 cm (Fig. 2). In the knots the girders have straight haunches. The height of the girder varies from 1,38 m in the field up to 1,98 m at the piers, respectively up to 1,83 m at abutments. The cross rigidity is assured by means of bridging on the abutments and with reinforced concrete ribs in the field (Fig. 2). The abutments and the piers are made of concrete belonging to the class C 2,8/3,5, indirectly founded on 15 piles of reinforced concrete having a square section of 30 x 30 cm (Figure 3).

Figure 1. Front view

Figure 2. Intrados view

Figure 3. Pier founded on piles

The concrete classes, out of which the elements of the superstructure and substructure, mentioned earlier, are realized, were determined by expertise the structure. It should be considered that the degradation process of the structure has evoluted in a time span of 9 years, passed after the moment the expertise was done (Fig. 4).

Figure 4. View at intrados

At this moment the gauge of 7,40 m includes a runway of 6,00 m and two footways of each 0,70 m.

The realization of a new viaduct of discharge, dimensioned from hydraulic point of view, is placed in a new emplacement, on the discharge direction of the water at very high levels.

This solution supposes the realizing of a new structure in another location, the execution of a traffic alternative, demolition of the old bridge, rearrangement of the earthwork in the area of the demolished bridge and the reestablishing of the riverbed. The location was chosen based on photographic recordings of the water flow in the major bed at high floods in correlation with the configuration of the land, showed using topographical measurements. In this situation the direction of the discharge of the waters will be perpendicular on the bridge axis, in contradistinction to the actual situation, reducing that way the level of the local scouring at the substructure elements.

Considering the amplitude, complexity and the duration of the works development for the designed structure, the traffic continuity will be assured on an asphalted traffic alternative realized upstream. This provisional alternative has one traffic lane and the traffic is being directive by traffic lights.

The static schema of the designed bridge is a continuous concrete slab with three spans: 23,0 m + 29,0 m + 23,0 m (Fig. 5).

In order not to modify the discharge conditions of the high floods, when designing the new structure, the level of the intrados of the old viaduct was maintained and an enlarged outlet was realized by reducing the substructure number and extending of the total length to 88 m.

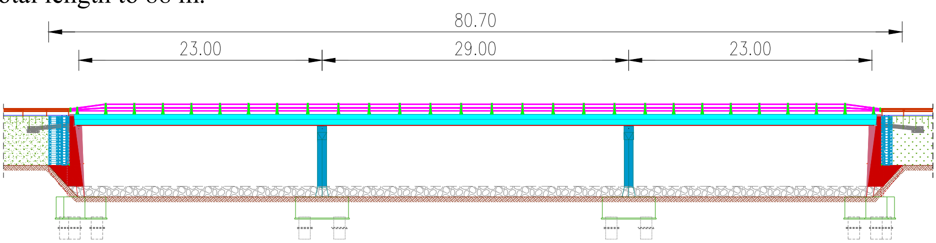

Figure 5. Side view

For the continuous superstructure, the solution with 12 HE beams is being suggested. The embedded beams are 75,40 m long. For the sustaining of the embedding concrete when casting, a lost shuttering of precasted slab will be used (Fig. 6).

The total width of the superstructure is 9,96 m realizing a gauge of 9,00 m, specific for the bridges placed outside of localities (without footways) (Fig. 6).

The runway has longitudinal reradiating directional pavement marking and luminescent buttons having a free distance of 2,00 m.

The roadside obstructions on the bridge, it is protected against corrosion by zinc coating, is conceived in a metallic solution, provided with structural elements in order to offer safety for the traffic and pedestrians.

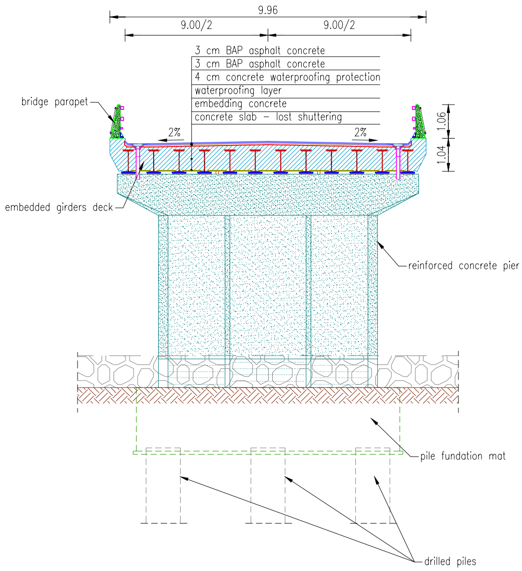

Figure 6. Cross section

The discharge of the meteoric waters from the bridge is achieved through gullies, respectively by means of side ditches at the extremities of the viaduct.

The bearings are made of reinforced neoprene and are put under each metallic beam.

The substructure consists in two abutments and two piers.

The elevation of the abutments is going to be made of plain concrete in classical solution, having reinforcement only at the joint elevation-foundation.

The piers have the elevation made of reinforced concrete giving the opportunity for significant slenderness.

Due to hard foundation conditions, the option was for indirect foundations on piles with a large diameter (columns).

Because the statically system is being chosen as a continuous structure with three spans, the expansion joints are used only at the extremities of the structure (at the abutments).

Considering the high difficulty level concerning the realization of the consolidation of the existing structure, regarding the technical and technological solutions possible to be put through, as well as the controvertible efficiency of these solutions dealing with a very high degraded existent concrete having also a low quality, it is considered that the entire replacement of the old structure is necessary.

2. GATAIA OVERPASS

The designed overpass is placed in the Timiş county belonging to the western part of Romania, on the national road which assures connection between the cities Timişoara and Reşiţa.

The designed work aims the replacement of two intersections at same level with the rail road situated at a distances of approximately 200 m, with an overpass. Alternatives with prestressed concrete superstructure respectively enclosed steel girders were studied. Both solutions did not include footways and the infrastructure was founded directly or indirectly. Further on the solution with the enclosed steel girders and the direct foundation system, adopted by the administrator of the national roads, will be detailed.

2.1. INFRASTRUCTURE

The indirect foundation system realized with cast in place concrete piles of big diameter (columns). The columns are about 9 m long, distributed on two parallel rows, namely 6 pieces at the abutments and 8 pieces for each pier, at the top being embedded in a reinforced concrete girder.

The elevation of the piers, the abutment wall, the retaining wall the bearing bench and the bolsters will be made out of concrete. The static schema of the infrastructures is a frame with two piers, embedded in the foundation mat in which the ends of the columns are embedded.

The piers realized as frames with two pillars, are designed in such manner as the distance between the piers permits the development of traffic under the overpass, between these (Fig. 3).

2.2. SUPERSTRUCTURE

The static schema of the designed overpass, in this variant, is a sequence of 3 continuous slabs with 5, 8 respectively 5 spans, having the following dimensions:

- 21,00 m + 3 x 25,00 m + 21,00 m = 117 m;
- 21,00 m + 6 x 25,00 m + 21,00 m = 192 m;
- 21,00 m + 3 x 25,00 m + 21,00 m = 117 m.

Figure 7. Top view towards the Timişoara slope

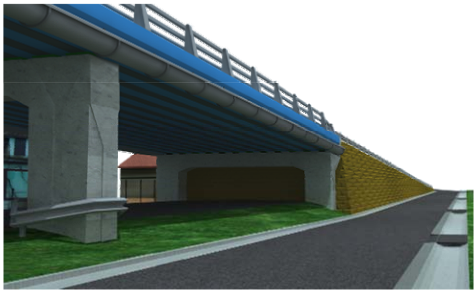

Figure 8. Bottom view towards the Timişoara slope

The overpass is designed curved in space with radiuses between 1300 and 4500 m vertically, respectively 540 m horizontally (Fig. 7, Fig. 8).

In this configuration the length of the superstructure is 428,50 m, and the entire length of the overpass (including the approach slopes) is 687,80 m.

The resistance structure consists in 12 HEA steel girders, embedded in the concrete slab (Fig. 9).

Due to the big length of the continuous girders, these will be junctioned through welding on the building yard from sectors of 25 m, in the sections with the lowest efforts. For supporting the coating concrete, for the pouring it will be used lost formwork out of 4 cm thick precasted slabs. The total height of the superstructure is 0,77 m.

The bearings are of reinforced neoprene and are placed under each steel girder, being different on the piers and the abutments being concordantly with the movements from each bearing. Due to the length of the over passing structure and to the adopted statical schema (sequence of three continuous structures), is necessary to mount four lines of contraction joint covering devices capable to support the movements of the bridge deck, two of them being placed at the end of the overpass (at the abutments) and two of them at the joint between the continuous structures.

The total width of the superstructures is 9.96 m out of which 9,00 m covers the gauge (Fig. 9).

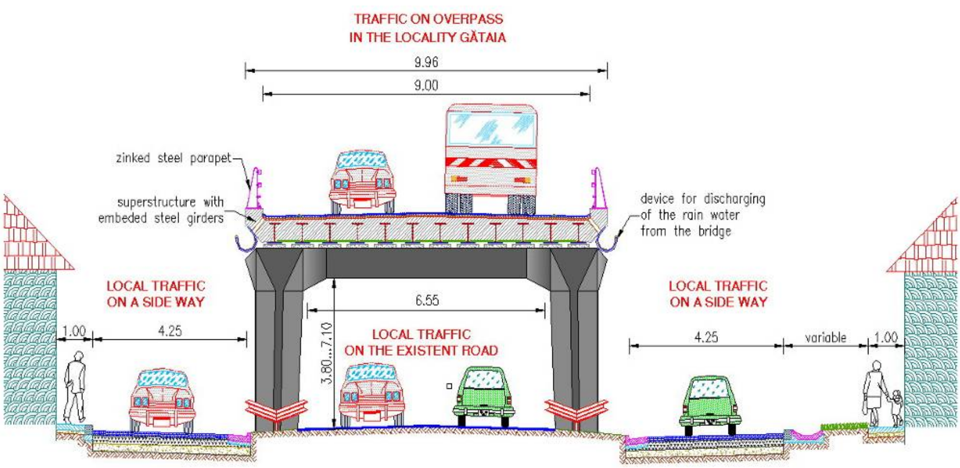

Figure 9. Cross section overpass

The parapet on the bridge, with anticorrosive protection through zinking, is designed in steel structures, endowed with structural elements which shell provide more safety for the auto and pedestrian traffic.

The cross section profile on the overpass is in roof shape with a slope of 2%. The carriageway is separated longitudinally with horizontal reflectorizing marking and luminescent buttons situated in free distances of 2,00 m.

The discharge of the waters from the bridge is solved with outflows placed on each direction in a distance of maximum 8 m, 2 x 2 on each span. It will be discharged in a discharge pan placed on each side of the overpass. These will be discharged by means of PVC tubes in the collecting and discharging system for used waters in the city (Fig. 9).

2.3. LOCAL TRAFFIC

Due to the fact that the national road goes through the locality and also due to the fact that on the current emplacement of the national road the approaching slopes will be executed, the execution of roads for the local traffic towards the land in the area and also to make the houses from both sides of the road, after the intersection with the rail road Gătaia-Buziaş, accessible is imperative.

The local traffic will develop under the overpass, between the piers, on a minimum free distance of 7,00 m, 6,00 m being the gauge. The minimum gauge for free passing is 3,80 m, while the maximum gauge reaches up to 7,10 m (Fig. 7, Fig. 8, Fig. 9).

In this conditions, on both sides of the approaching slope will be normal traffic conditions for the local traffic (pedestrians and vehicles) (Fig. 7, Fig. 8, Fig. 9).

2.4. ADVANTAGES OF THE PROPOSED SOLUTION

By studying the possible variants and simultaneously analyzed, the following can be stated:

- The big height of the prestressed concrete superstructure represents an optical discommode for the inhabitants from the Timişoara extremity of the overpass;
- The difference between the height of the superstructure with prestresses concrete girders and that with embedded steel girders is of 0,48 m, which leads to:
  - an increase of the length of the approach slopes with 225 m and of the earthwork volume with 27%;
  - an increase of the height of the approach slopes implicit of the retaining walls;
  - increase of the earthwork volume for realizing of the intersections with the adjacent streets;
  - arranging of the affected streets on a larger surface;
- due to the fact that the local traffic is assured as well for pedestrians as also for vehicles, on areas at the terrain level (near the Timişoara approach slope and under the overpass), an eventually designed footway, would be not used developing into an discommode source, leading to premature damages of the superstructure of the bridge and also generating additional costs for maintenance;
- the superstructure with embedded steel girders in the solution without footways is the most advantageous one from technical economical point of view because it shows some additional advantages:

– transport facilities, manipulation and execution for the main resistance elements;
– simplicity for maintenance;
– higher durability considering the corrosion of the main resistance elements (prestressed reinforcement suffers degradation faster than a laminated profile with anticorrosive protection).

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TENTH NATIONAL AND
FOURTH INTERNATIONAL
SCIENTIFIC MEETING
iNDiS 2006
NOVI SAD, NOVEMBER 22-24,2006.

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POLYMERIC COMPOSITES FOR SEISMIC REHABILITATIONS - STUDIES AND RESEARCH

***Summary:** The paper presents the theoretical studies and the experimental tests, made at the "Politehnica" University of Timisoara, on the behaviour of the structural elements made of concrete and brick masonry and consolidated with polymer composite materials. There are described the types and the properties of the component materials of different composites, their advantages and disadvantages, and the modes and the systems for the utilization, respectively. During the research programme, carried out along several years, we studied the polymeric composites with carbon fiber and glass for retrofitting certain structural walls. The research team studied six pairs of masonry structural walls subjected to shear forces. After the first experimental test up to failure, the walls were consolidated by applying certain glass or carbon fabrics and then retested. The displacement performances and the load bearing capacity were compared before and after consolidation. The tests pointed out the load bearing brought by the consolidated elements and the failure mode after retrofitting. In the case of the concrete structural walls, the first stage consisted in studies made on the behaviour at horizontal loadings of the walls with monotonic and staggered openings. In the second stage the structural walls were consolidated with carbon fiber fabrics and then retested. Not only their behaviour was studied after retrofitting, but the load bearing capacity and the failure mode as well. The conclusions of the studies and of the experimental tests resulted in a series of recommendations for the calculus, the design and the specific technology for the use of the composites at the seismic rehabilitation of the structural elements.*
***Key words:** composite materials, experimental tests, rehabilitation, structural masonry walls.*

POLIMERNI KOMPOZITI ZA SEIZMIČKU REHABILITACIJU - STUDIJA I ISTRAŽIVANJE

***Rezime:** U radu su prikazani teorijska studija i eksperimentalni rezultati sa Politehničkog Univerziteta iz Temišvara, o ponašanju konstrukcijskih elemenata od betona i zidanih zidova učvršćenih polimer kompozitnim materijalima. Opisane su vrste i svojstva kompozitnih materijala, njihove prednosti i nedostaci, kao i načini i sistemi njihovog korišćenja. Tokom višegodišnjeg istraživanja proučavali smo polimerne kompozite sa karbonskim i staklenim vlaknima za pojačavanje nekih zidova. Istraživački tim je proučavao šest parova zidanih konstrukcijskih zidova izloženih smičućim silama. Nakon prvog eksperimentalnog ispitivanja do loma, zidovi su sanirani primenom staklenih i karbonskih vlakana i ponovo ispitani. Pomeranja i nosivost zidova je upoređena pre i nakon sanacije. Zaključci studije i eksperimentalnog ispitivanja rezultirali su serijom preporuka za proračun, za projektovanje i za specifičnu tehnologiju upotrebe kompozita za seizmičku rehabilitaciju konstrukcijskih elemenata.*
***Ključne reči:** kompozitni materijali, eksperimentalna istraživanja, rehabilitacija, konstrukcijski zidani zidovi.*

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1. INTRODUCTION

The development of the composite materials allowed the development of certain technologies, both for the consolidation and the execution of buildings with a special characteristic. The dynamic of the use of composites in constructions is increasing, nowadays they are more and more used in the process of the consolidation of the old buildings. The composite retrofitting are made by applying certain fibrous materials, resin impregnated, on the surface of different elements in order to remake or increase the load bearing capacity (at bending , shearing , compression and/or torsion), without a significant influence on their stiffness. The fibers used in the consolidation applications can be made of glass, aramide or carbon. The composites have different shapes, they can be pultruded plates, dry or pre-impregnated fabrics, and recently reinforcements. The consolidated elements can be made of: concrete, brick, wood, steel and stone, and from the structural point of view they can be: beams, walls, columns, and slabs, and lately, they are also used for beam-column joints.

The composite consolidations have a lot of advantages. Among these, we would like to mention the most important ones:

-low weight, approximately 20% of the steel weight, thus the costs for transport and the usage (do not need any support) are reduced considerably. This advantage recommends them for situations in which one cannot apply significant permanent loadings at the retrofit rehabilitation of a structure or of a structural element.

- high ultimate tensile strength, at least three times higher than the steel tensile strength;

- very high strength- to-weight ratio, the composite could have less than 10% of the steel weight, at the same ultimate strength;

-possibiliy to direct the maximum resistance capacity, by choosing the position, orientation and the fiber volume;

- high endurance, adequate to be used in aggressive mediums;

- dimensional stability, low thermal conductivity and low thermal extension coefficient;

-magnetic and radar transparency, being recommended for special requirement applications;

-do not need maintenance; the maintenance cost being reduced for the entire life of this system;

- possibility of precompression;

-large variety of the systems, being able to be produced at any required length. They even can be fixed in layers to coat the required element;

-low execution time, minimizing thus the costs resulted out of the production/ traffic interruptions;

-possibility to use them in restricted areas, due to the low thickness of the systems;

- high resistance to impact/explosions.

The advantages offered by the retrofitting with composite materials lead to long term and short term savings. The usage of these types of materials, on larger and larger scales, leads to a decrease of the production price and to quick solutions for the consolidation of the structural elements.

The key to the efficient use of the composite materials consists in the use of fibers in accordance with the objective followed, by applying the fibers in the right positions and orientations, and the realization of an optimal fiber proportion in accordance with the adequate matrices.

2. STATE OF THE ART OF COMPOSITE RETROFFITTING

There are some recommendations for the composite retrofitting calculus, but because of the large variety of the systems used, both from the fibers point of view and that of the matrices, they need corrections and they are still developing. Many institutions all over the world work in groups and commissions with one objective in view i.e. devise certain norms/standards for the design and the execution of the consolidation/ reinforcement works with composites that are to be accepted internationally. So far the main recommendations or norms used are the following:

- Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures, ACI 440.2R-02, 2002
- Technical Report in the Design and Use of Externally Bonded FRP Reinforcement for Reinforced Concrete Structures, fib TG 9.3, 2001
- Technical Report on Continuous Fiber Reinforced Concrete, JCI TC 952, 1998
- IBCO ES "Acceptance Criteria for Concrete and Reinforced and Unreinforced Masonry Strengthening Using Fiber Reinforced Polymer (FRP), Composite Systems", AC125, 2001
- Guide to the Structural Use of the Adhesives, Institution of Structural Engineers, 1999

It is worth mentioning that, in the field of constructions, the use of composites is the branch with the most spectacular development in the recent years. This fact can also be demonstrated by the number of the research papers published, which definitely show a continuous growth over the years.

3. THEORETICAL STUDY AND EXPERIMENTAL TESTS ON MASONRY CONSOLIDATED WITH COMPOSITES

The programme had as a main objective the theoretical and the experimental study of walls made of clay brick masonry, under the shearing force, and consolidated on one side with FRP composites. The main stages of the programme were: the finite elements modelling of the walls under the shear force (Figure 1), the testing of the simple walls, the consolidation of the walls on only one side with composites and then retesting them (Figure 2), the comparison of the experimental results and then drawing some final conclusions.

On the basis of the results obtained, we formulated the following observations and conclusions:

- composite retrofitting of the cracked walls, made of plain masonry, leads to a significant increase of their shear load carrying capacity (actually, the load bearing capacity of the cracked walls is zero);
- recordings of the specific strains in composites demonstrate their contribution to the behaviour of the consolidated masonry and the concurrence of the composite with the brick masonry;

- failure of the consolidated walls was caused by extensive openings of the cracks, followed by debonding of the composite in the cracking area, and not to the tensile failure or to the shear of the composite.

- maximum horizontal displacement of the consolidated walls have increased at least twice compared to the initial ones, fact that demonstrates the increase of the ductility and the energy absorption capacity of the consolidated walls;

- the best compatibility between the composite and the masonry wall proved to be found at the glass fiber.

The simple masonry walls under the seismic forces have a very fragile behaviour and fail without warning. By consolidating these types of non ductile structural elements with composites, which also have a fragile behaviour, the characteristics of the elements can be considered rather ductile.

The research on the retrofitting masonry with composite materials is being carried out, other elements being prepared to be tested, by using composite systems and different layer types. We also consider carrying out some investigations to study the behaviour of the elements at the masonry-composite interface.

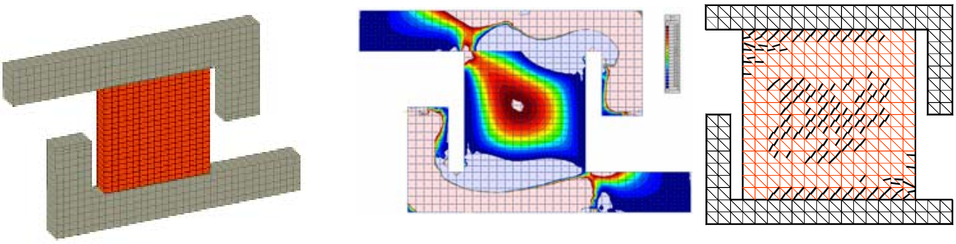

Figure 1. Finite element meshes

Figure 2. Testing the masonry walls consolidated with fiber carbon and glass composites

4. EXPERIMENTAL TESTS ON REINFORCED CONCRETE STRUCTURAL WALLS

The aim of the programme was the investigation of some reinforced concrete structure walls (Figure 3) consolidated with polymer composite materials, increasing thus the bending load bearing capacity and their shearing force. The main stages in the research programme were the modelling of the experimental walls with finite elements, the testing of the reinforced concrete structure walls up to failure (Figure 4), the consolidation of the walls on one side with composites and then retesting them (Figure 5), and in the end the comparison of the experimental results and drawing some final conclusions.

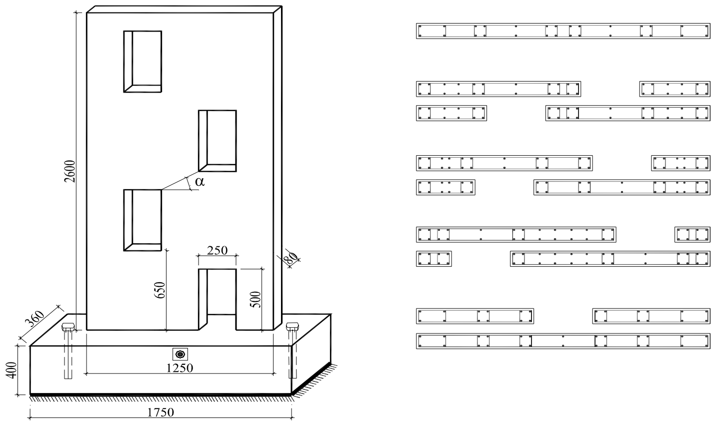

Figure 3. Geometry and the details of the experimental elements

Figure 4. Reinforced concrete structural walls tested

Figure 5. Reinforced concrete structural walls retrofitted and tested

Based on the results obtained, the following conclusions were drawn:

- composite retrofitting of the concrete structure walls determines a significant increase of their ultimate load bearing capacity (actually, the load bearing capacity of the walls tested was insignificant).

- the strain in the composite indicates the major contribution of the CFRP in the load bearing capacity of the walls, the average recorded value being 0.54 ÷ 0.84%.

- the failure of the retrofitting elements was made by the gradual opening of the existing cracks, by the debonding of the composite in the compressed area, then in the extended area at the base of the sidewalls, followed by the tension failure or sometimes by compression.

- maximum horizontal displacement of the consolidated walls were usually higher, or at least identical to the reference walls.

The results obtained depend to a great extent on the initial state of the consolidated element (number and the openings of the cracks, the quantity of the strengthening material used, the method and the materials used for the rehabilitation) and on the evaluation method used, respectively. The method used to evaluate the mechanical characteristics led to the following observations:

- stiffness of the elements decreased by an average of 54%;
- ductility of the elements decreased by an average of 61%;
- value of the elastic limit force of the walls increased by an average of 48%;
- maximum load increased by an average of 46%;
- composite specific deformations had values between 0. 54-0, and 0.84%;
- the anchorage system behaved extremely well, without degradations or local failures.

The concrete walls under seismic forces exhibit a ductile behavior. By consolidating these types of ductile structural elements with composites, which are materials with non ductile behaviour (without leakage level), the ductile behaviour of the elements can be maintained, but at maximum loading the failure is fragile.

5. THE STUDY OF THE CONCRETE BEAMS WITH STRUCTURAL DISCONTINUITIES

This programme had as aim the theoretical and the experimental study of the behaviour of the concrete beams with structural discontinuities, consolidated with composite materials. Internationally, this field is relatively less studied. There are no references or recommendations not even in international calculus normatives (ACI, JCI, FIB) . We studied thoroughly the case of a dapped-end beam supported on column.

We analysed the possibility of increasing the load bearing capacity at the dapped-end beam by retrofitting it with composite materials. The theoretical calculus for the unretrofitted elements was made both in the elastic (Figure 6), and in the post elastic (Figure 7) domains correlated with the results obtained through strut-and-tie models (Figure 8). Experimental tests were made on elements at the scale 1:1 (Figure 9). The first element was tested up to failure, and the other three up to the elastic limit. Then followed the consolidation of the elements by using different solutions and finally they were retested.

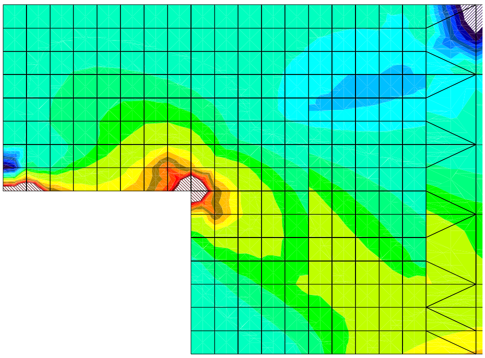

Figure 6. The distribution of the main unit stresses (Axis VM)

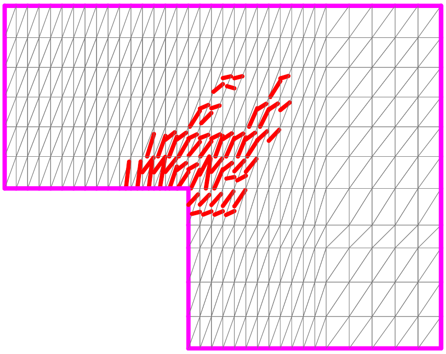

Figure 7. Crack propagation (BIOGRAF)

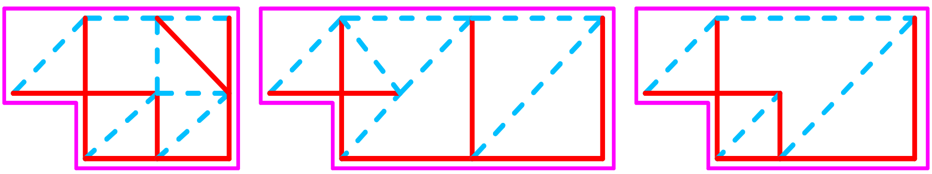

Figure 8. Models of the bars used

The retrofitting was conceived to lead to an increase of 20% of the service load, in terms of displacements and efforts in the steel reinforcement, without a significant change of the stiffness.

Four experimental elements were tested (dapped-end beams), the aspect after testing them is shown in Figure 10.

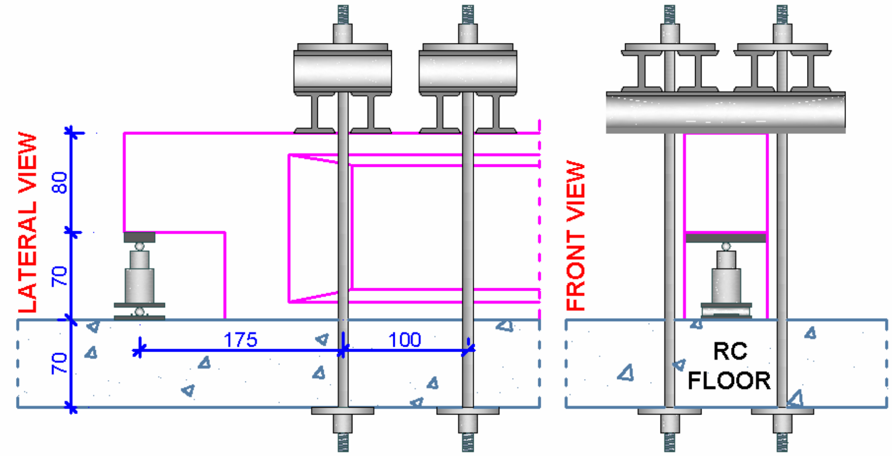

Figure 9. Loading scheme (side and front view)

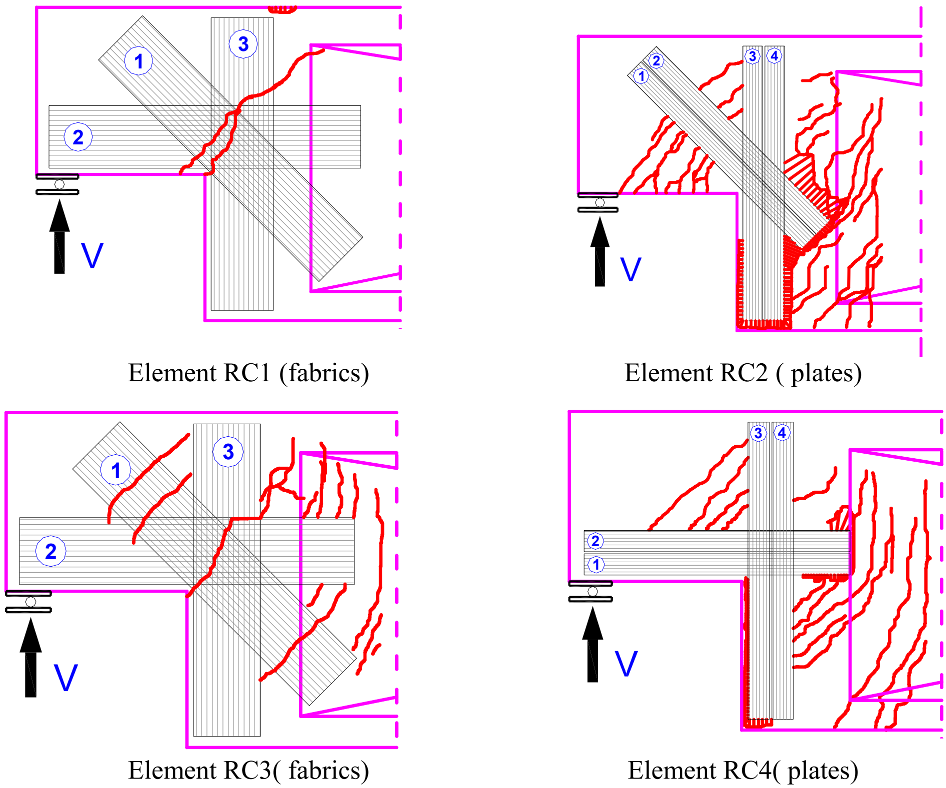

Figure 10. Experimental elements tested

On the basis of the tests made, we can conclude:

- theoretical models approximate rather well the actual behaviour.
- FRP system proved to be a correct choice for these types of applications, the recorded increase of the service load being of 25% for RC3, 40% for RC4 and of 45% for RC2.
- ultimate load bearing capacity increased by 11% for RC1, 10% for RC2, 6% for RC4 and stayed the same for the element RC3.
- elements strengthened with fabrics exhibit a more ductile behaviour than those retrofitted with plates.
- the elements consolidated exhibit a delay in the opening of the cracks, the failure being made by the breaking of the fabrics fibers or by the peeling-off of the plates.
- with respect to the baseline specimen C1, the maximum displacement was approximately identical for the fabric retrofitted elements, but 30% lower than in the case of the plate retrofitted elements.

6. REINFORCED CONCRETE BEAMS RETROFITTED ON ONE SIDE WITH POLYMER COMPOSITE MATERIALS

The paper had as objective the bending test for a concrete beam retrofitted with carbon fiber, applied laterally. The first stage consisted in testing the non retrofitted beam up to the elastic limit (Figure 11). The beam was then unloaded and prepared for the application of the carbon fiber fabrics. The second stage consisted in retesting the retrofitted beam, up to failure (Figure 12) .

The theoretical modelling was made by using the programmes AxisVM and BIOGRAF.

Even if intuitively it seems a less efficient solution, the viability of technique was also proved experimentally, thus obtaining an increase of the load bearing capacity of 28% up to the accepted distortion limit, and of 35% up to failure. It is worth mentioning the brittle failure mode .

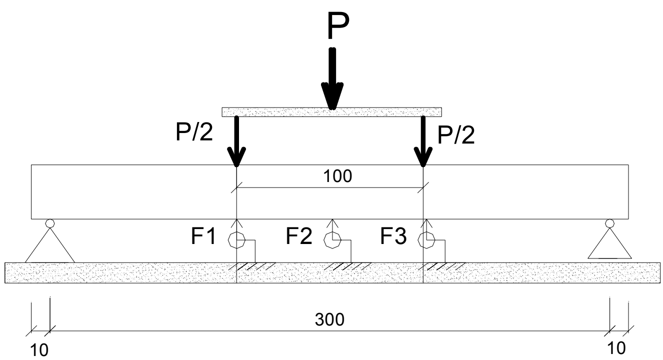

Figure 11. The scheme of the experimental stand

Figure 12. The consolidated beam, prior to testing

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GEOTECHNICAL CONDITIONS FOR BUILDING A NEW TUNNEL INSTEAD OF THE OLD ONE TROUGH THE PETROVARADIN FORTRESS HILL

Summary: *In the peaper briefly are presented aspects of widening of the old existing tunnel or conditions for building of a new one with enlarged cross-sectin. In smaller part the tunnel would be built in soft rocks, while the greater part would be built in diabases of the Fortress, in which specific quasi-homogenous zones with building conditions are separated, while the classification system also considers building conditions from time when the existing tunnel was built. Tthe Fortress has great cultural-historical significance so that it was necessary to consider how builing of a tunnel would affect the objects of on the surface. To that end into comparatively detailed analysis, made according to parameters available in this examination phase, several cross-sections of the tunnel have been introduced. Data here presented are certainly interesting to wider professional public, from the geotechnical aspect, but also due to the fact that in the near past it was the first tunnel-bridge which connected Novi Sad and Petrovaradin.*

Key words: *tunnel, diabases and soft rocks, zoning in tunnel and excavation classes*

GEOTEHNIČKI USLOVI IZGRADNJE NOVOG TUNELA UMESTO STAROG KROZ BREG PETROVARADINSKE TVRĐAVE

Rezime: *U radu su ukratko prikazani geotehnički aspekti proširenja starog postojećeg tunela ili uslovi izgradnje novog povećanog poprečnog preseka. Manjim delom tunel bi se gradio u mekim stenama, a većim delom bio bi izgradjen u dijabazima Tvrdjave, u kojima su izdvojene odredjene kvazihomogdne zone prema uslovima gradnje, a u sistem klasifikovanja je uvedeno razmatranje uslova izgradnje postojećeg tunela iz vremena izgradnje. Tvrdjava ima veliki kulturno-istorijski značaj pa je bilo neophodno razmatrati i uticaj izgradnje tunela na nadzemne objekte na površini. To je učinjeno tako što je u relativno detaljnu analizu, prema parametrima koji su se posedovali u ovoj fazi ispitivanja, uvedeno nekoliko poprečnih preseka tunela. Podaci koji se prezentiraju zasigurno su interesantni širokoj stručnoj javnosti, upravo sa geotehničkog aspekta, a i prema tome da je to u bliskoj prošlosti bio prvi tunelsko – mostovski objekat koji je spajao Novi Sad i Petrovaradin.*

Ključne reči: *tunel, dijabazi i meke stene, zoniranje u tunelu i klase iskopa*

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1. INTRODUCTION

In the General Urbanistic Plan of Novi Sad till 2021, it is planned to re-establish traffic communication between the Backa part of Novi Sad and Petrovaradin through the Petrovaradin Fortress. The old tunnel will be widened or a new one will be built. The length of the tunnel through Fortress is 354,0 m.

The whole entity of the Petrovaradin Fortress includes overground and underground space where numerous objects are on the ground but also underground. Buildings have extremely great cultural-historical significance. Rather vast existing documentation has been viewed: geodetic, archeological, geological, structural, urbanistic et al., because of two basic reasons:

- to precise conditions and possibilities of technical-technological solution of building of a tunnel through Fortress, wthout any endanger for the existing objects on the surface (buildings, existing supporting walls made of bricks etc.),
- by widening of the existing tunnel, or a new tunnel next to the now exiting do not destroy or damage any parts of existing underground objects (tunnels, war well etc.).

The Petrovaradin Fortress is a cultural-historical monumental complex of the highest rank. As an extremly significant object, in 1947 it was put under protection of State, and it is registred among the first objects, i.e. number 3. For affirmation of both cultural-historical values of the Fortress, and to use its potentials, a very detailed urbanistic plan of this area has been made. Its provision provides necessary preconditions to begin professional works for valorization of all potentials, to perform works of: revitalization, renewal, protection and rehabilitation of already built objects; to design new objects, like for instance the said tunnel, etc., have a real insight into possibilities of corresponding economy branches, and especially to make the Fortress more accessible and more atractive to tourists and city ihabitants.

2. RESULTS OF EXAMINATION

2.1. Composition of the aera

The oldest preserved data about geological composition of the Fortress are from 1818, by the author F.S.Beudant. After that author the Fortress is settled down on serpentinit. Those data have exclusively historical significance, because today on disposal are many more precise and detailed data about terrain composition of the narrower and wider Fortress area. Critically observing, and based on detailed terrain mapping and inspection of sprouts of those rocks in Petrovaradin, it is certain that on all open sprouts on the overall Fortress there are diabases, not serpentinit. From the geotechnical aspect, diabases are considerably better environment than serpentinit for construction.

Diabases are storngly cracked into blocks of different size. Generally, some are of meter dimensions, while many are smaller. Lawfullness related to their cracks, blocks size, stability particularly in steep kerfs and their falling part in the zone of the road R-107, are examined in details on the open sprouts on the slope and also in non-wall-supported parts of the old railroad tunnel.

Diabases are moulds of vulcanic gabro, of Triassic-Jurrasic age. Hence, in form of open sprouts they are in the kerf of the road R-107, on the upper plateau of the Fortress they are practically on the surface in several places of the terrain, and also were found in the Danube's riverbed and that from the slide on the Danube slope under the middle plateau up to the bridge "Duga"("Rainbow").

Apart from those on the surface in the Fortress zone, diabases are accessible to detailed examination and calculations along the considerable part of the said, already performed old tunnel through the Fortress. Namely, by inspection of the tunnel, as well as by inspection of the vast documenation about it, their presence is determined. From the entrance portal in the Danube zone, i.e. from St.769.28m up to for about 1050m, the terrain is composed of diabases. Diabases are dominantly green in colour. Certain sub-zones and parenthetics are in tones of dark redish. Generally, and based upon overall investigations of diabases, it is for sure seen that diabases within terrain are in the form of two huge geotechnical units. The first is from the entrance portal on the Danube up to length of 100.7m in tunnel. Those diabases are less cracked, less tectonic and create extremly suitable and good environment for building the tunnel, also are suitable from the stability point of excavation site and objects on surface. That geotechnical unit is also seen clearly in the open part of the slope in the Fortress zone. The second geotechnical unit of diabases form those more cracked, and within tunnel they are distributed from St. 869.98m till St. 1050m. The whole massive is, except in colour, anisotropic regarding cracking, fisures, characteristics of physical-mechanical properties of that rocky mass.

Contacts of diabases and pliocene clay-sandy-marly sedimenats are clear. Videlicet, those sediments are discordantly deposited around those rocks. The same applies to quartarian sediments of loess. Relations between the Triassic-Jurrasic sediments, when diabases are also impressed, have not been precisely defined by detailed examinations, and that, from point of view of these investigations, has no influence and significance.

Ispection of the tunnel showed presence of undergraound water in the tunnel. When the tunnel walls were examined the sudden snow melting on the surface terrain happened so that it was favourable for bringing conclusions about undergraund water. Namely, if was established that water infiltrated from the terrain surface moves along characteristic cracks and consequently as filtered water appears in the tunnel. The most intense appearing of the underground water happened some 40m from the entrance portal of tunnel in the Danube zone and that as moistening and intense dripping especially from the tunnel's vault and characteristic places. In that zone, as well as along complete length of that part of tunnel from the portal up to length of 100.7 m the tunnel is unsupported. Presence of water in form of moistening and scattered dripping from the tunnels vault are especially intensive in the pre-portal part of the entrance tunnel in the Danube zone and up to walled in niche in the tunnel about 125m far from the entrance portal. From that niche up to the exit from the tunnel in Petrovaradin it is dry, there are no visible appearance of the underground water. That does not mean that broaken structure of the sptout in terrain is not present but that assuredly can be stated that around the walled in tunnel, there is drainage by which the underground water is introduced into drainage and carried out into the tunnel floor and that with fall towards Petrovaradin. Greater number of cracks of diabases in these sections provide huger quantuty of underground water. Yet, having in mind the cracks narrow as well as comparatively small surface from where the water is infiltering into the terrain, it is certain that the matter is in small quantity of underground water, which is actually neglectable. From the stand point of influence of underground water upon stability of certain unstable blocks, even that small quantity is important and influenced determination to build support to some zones with intensified moistening and water incoming.

2.2. Division of rock mass, blocks hinge caracteristics

The existing tunnel through Fortress, as well as the future widened existing one or separate new one, will be carried out from the entrance portal in the Danube zone up to St.1077.2m in diabases, and from St. 1077.2m up to exit portal in Petrovaradin in semi-bound and non-bound rock masses. Division of diabases into blocks of different size as well as their relationship are the most important natural elements for evaluation of the stability of the tunnel and objects. Due to that the special stress is given to investigation of those structural elements and to investigation on open sprouts, in the tunnel and drilling core.

Division of rocky mass of diabases is very expressed and accessible to direct investigation. Therefore, needed inspection of the practically undiscovered Danube slope in the wider zone of the entrance portal was made, also in the tunnel part from the entrance portal up to St. 869.98m and another two non-wall supported zones in the tunnel and which are on the diabases.

According to degree of division of rocky mass, there are in diabses two clearly distinguished geotechnical zones and within them sub-zones. Manner and degree of division in them essentially defines geotechnical conditions of building a tunnel of that existing tunnel, but especially a new one if is of the wider cross-section.

The ruptures in diabases are classified into four classes. They are ruptures fault which intersect the whole tunnel; long cracks which also intersect the whole tunnel or its greater part; cracks of meter dimensions; and short cracks. From this classification it is clear that defining elements of cracks length, but in all made analyses the other specific characteristics of ruptures were, also, taken into account. So for example the cracks of first order, i.e. fault are always with thinner or fatter zone of crushed rocky mass and there are mainly appearances of some unstability in the excavate.

Basic zoning of the area has been done based on kinds of present anvironment, and in that respect are present two basic entities. The first is from entrance portal on the Danube till St. 869.98m. The second is from St. 869.98m till St. 1077.2m. Within the area from 869.98m to 1077.2m there are several sub-zones. Basic are those that are not supported and among them zones with performed sub-support.

2.3. Geotechnical classification of rocky masses

Known procedures of N. Burton (Q procedure), Z. I. Bieniawski (RMR classification) and M.Vasića (RBR procedure) served for geotechnical classification of rocky massive.

The most important parameters for classification have been determined by means of detailed inspection of walls of excavation of the old tunnel, in the sections where it is not wall-supported. Those data could be considered real and reliable. Differently there are sections where the excavations accessible to vision and study, there are sections in the tunnel, and that rather big length, which are walled in and where walls are visible. Based on actually collected data about diabases of Fortress on the uncovered sprouts on the terrain surface it could be foreseen that there also are such zones in areas of the excavation of old tunnel. That means that based on inspection of the tunnel, which is in diabases on non-wall-supported sections, inspection of these diabases on the uncovered sprouts of the Fortress, also by inspection of sprouts on the terrain surface, which correspond to wall-supported sections in the old tunnel the real state of the rocky masses actually could be précised in the overall tunnel. The paper presents only a classification example for one characteristic sub zone in the Danube's zone up to 100.7m.

Q classification system

Q = 0.024458 * EXP (0.073123*RQD) ; R = 0.808
RQD = 95.0000 ---------------> Q = 25.42892
Q = 51.623856 * EXP (-0.379581*In) ; R = 0.658
In = 6.0000 ---------------> Q = 5.29360
Q = 0.074376 * (RQD/In) ∧ 1.666818; R = 0.786
RQD/In = 15.8333 ---------------> Q = 7.42851
Q = 0.203793 * EXP (1.323819*Ir) ; R = 0.580
Ir = 4.0000 ---------------> Q = 40.63483
Q = 11.570654 * Ia ∧ (-2.249515) ; R = 0.832
Ia = 1.0000 ---------------> Q = 11.57065
Q = 2.787576 * (Ir/Ia) ∧ 1.498533 ; R = 0.840
Ir/Ia = 4.0000 ---------------> Q = 22.25530
Q = 9.209769 * Iw ∧ 3.212126 ; R = 0.469
Iw = 1.0000 ---------------> Q = 9.20977
Q = 32.848607 * EXP (-0.556793*SRF) ; R = 0.570
SRF = 2.5000 ---------------> Q = 8.16557
Q = 21.948076 * (Iw/SRF) ∧ 1.203177 ; R = 0.467
Iw/SRF = 0.4000 ---------------> Q = 7.28793
Qsr = 15.98108

RMR classification

RMR = 39.135795 * P ∧ 0.154050 ; R = 0.828
P = 15.0000 ---------------> RMR = 59.39515
RMR = 9.301854 * EXP (0.117552*RQD) ; R = 0.923
RQD = 19.0000 ---------------> RMR = 86.80825
RMR = 4.419010 * F1 - 6.922980 ; R = 0.908
F1 = 18.0000 ---------------> RMR = 72.61920
RMR = 3.444811 * F2 - 0.251581 ; R = 0.874
F2 = 27.0000 ---------------> RMR = 92.75832
RMR = 4.860290 * V + 5.207934 ; R = 0.868
V = 13.0000 ---------------> RMR = 68.39170

RBR classification

$$RBR = 5.790593 \cdot \ln Vb + 67.9588$$
$(Vb$ - je u $m^3)$
$$RBR = 8.68589 \cdot \ln Fb + 67.9588$$
$(Fb$ - je u $m^2)$
$$RBR = 17.371779 \cdot \ln Lb + 67.9588$$
$(Lb$ - je u $m)$
$$RBR = 2\varphi - 10$$
$$RBR = 100 \cdot Jr / Ja - 10$$
$$RBR = 28.853901 \cdot \ln \sigma_P - 52.877124$$
$(\sigma_P - MN / m^2)$
$$RBR = 80 - 12.426699 \cdot \ln W$$
(W- parameter dependent on inflow of undergaud water)

Block size	10.0000	(m3)	29.81	81.29	A
Sher resistence	57.0000	(°)	23.00	100.00	A
Hardness against compression	230.0000	(MN/m²)	29.00	100.00	A
Inflow of underground water	1.0000	(l/min/10m)	12.00	80.00	A
SUMMERIZED INFLOW			93.81	93.81	A

Based on comparative value of geotechnical classifications, and which are essentially based on empirical data of designed building of tunnel-tunnels and other underground structures the following has been concluded:

- For diabases from entrance till 100.7m class Q is 13th.. To provide stability it is required to apply unsystematic anchors, mesh and thin layer of torque concrete of 5cm layer.
- For more cracked diabases from 100.7 to 307.9m the class is 17th. To provide excavation stability it is required to apply systematic anchors, mesh and torque concrete of 10 cm.
- For fault zones and very intensively cracked diabases from 100.7 to 307.9m class is 30th. It is needed to provide for the support to apply systematic anchors at a distance of 1m, wire mesh and torque concrete of some 15 cm layer.

2.4. Influence of tunnel on objects

Analyses have been made for the design solution with one tunnel, which actually would be the widened existing one, and for the design solution for two independent tunnels. In agreement with performers of preceding traffic analyses it is adopted to have width of the newly designed tunnel, for case of one tunnel *B=16m*, and height *H=9m*, while in case of two tunnels *B=8m*, and height *H=8m.*

This analysis introduced:

- Changes of strain-deformation states,
- Deformations and shears in terrain where the important objects are on the Fortress terrain surface and their possible damage,

- Variants of tunnel building,

  Numerical analysis for one tunnel (16m x 9m) consists of:
- *analysis I:* strain-deformation state of the existing tunnel without widening and covering,
- *analysis II:* strain-deformation state of the new widened tunnel, in the place of old tunnel,
- *analysis III:* strain-deformation state of the tunnel with performed covering.

  Numerical analysis for two independent tunnels (8m x 8m) consists of:
  - *analysis I:* strain-deformation state of the existing tunnel without widening and covering,
  - *analysis II:* strain-deformation state of the new tunnel, in the place of old tunnel, with performed covering,
  - *analysis III:* strain-deformation state of two new tunnels, with performed covering.

All analyses have been carried out for characteristic *A*, *B* and *C* sections, based on the geotechnical terrain cross-section along the tunnel. Given cross-sections represent separated quasi-homogenous zones.

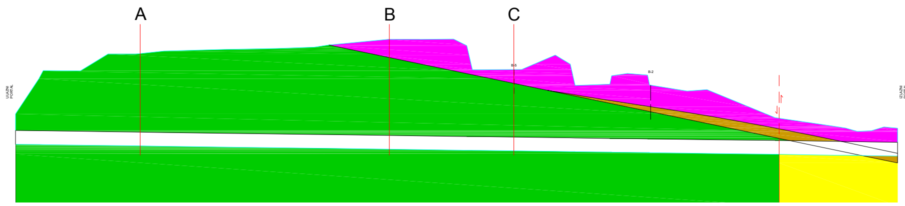

Fig. 1. Characteristic A, B and C sections (Karakteristični A, B i C preseci)

Analyses have been done based on estimated values of elasticity module *E* and *Poisson's* coefficient *ν*, for diabases, while also has been taken into account the tunnel's covering with real stiffness.

section	E (GPa)	ν	γ (kN/m³)	d (cm)
A	40	0.19	26	15
B	25	0.23	25	30
C	0.1	0.26	22	40

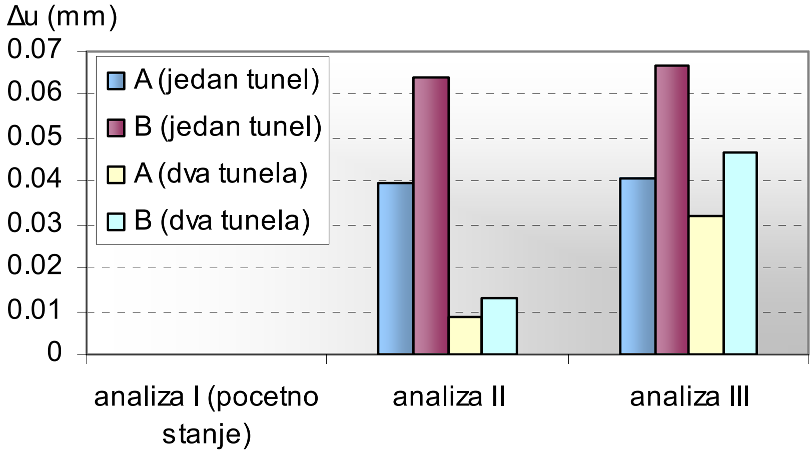

Fig. 2. Vertical shear of the knot K in function of gravitation load for cross-sections A and B (Vertikalno pomeranje čvora K u funkciji gravitacionog opterećenja za preseke A i B)

Shear values were calculated in relation to zero deformation state, i.e. in relation to deformation of the so far existing tunnel. Figure 2 depicts shear of the knot *K* in sections *A* and *B*. Figure 3 depicts shear of the knot *K* in section *C*. It is evident that for the rocky massive, i.e. diabases (*E=40GPa*), the sizes of vertical shear on the surface are neglect ably small. This also indicates that the differential shears are also very small. By reducing of elasticity module, deformations are greater, i.e. deflections on the terrain surface, what is well depicted in Figure 3. It is necessary to emphasize, based on overall analyses, that distribution of environment as it is in the section C is extremely small. Essentially the matter is in two split zones with estimated length of 1m to 2m.

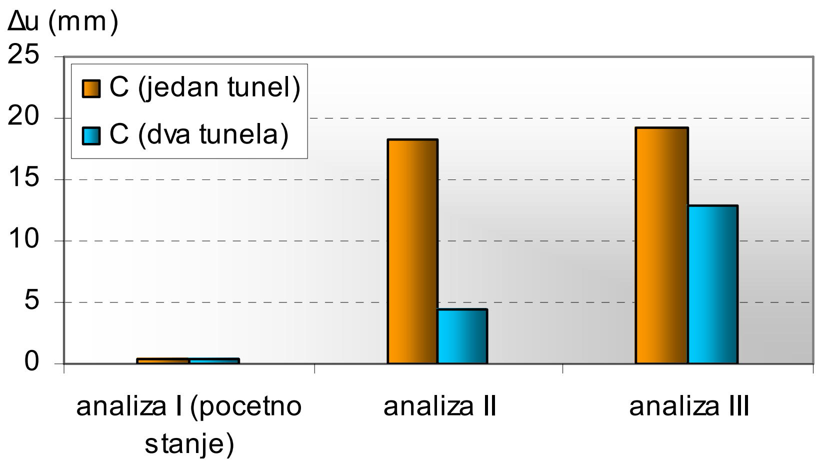

Fig. 3. Vertical shear of the knot K in function of gravitation load for cross-section C (Vertikalno pomeranje čvora K u funkciji gravitacionog opterećenja za presek C)

3. CONCLUSION

Paper presents summarized survey of preceeding-Prethodnih geotechnic-al analyses carried out for needs to begin production of technical documentation of the traffic communication that also includes tunnel under the Fortress. The tunnel will in its greatest part will be made in diabases with favourable properties, while in its minor part in semi-connected and non-connected rocky masses and that in open excavation. Particularly is stressed need for considering ofimpact the tunnel upon all objets-facilities within the Fortress, as well as analyses of widening of the existing tunnel or building of two completely separate tunnels. In coming phases of design the geotechnical data will be completed with new ones, with higher quality.

4. REFERENCE

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THE DIFFERENCES BETWEEN FORCE BASED AND DISPLACEMENT BASED DESIGN

Summary: *Here are compared two methods of seismic design: traditional-force based and the new one-direct displacement based design. In the paper that follows are briefly describe the fundamental differences between these two design philosophy and conceptual problems for force based design starting from the stiffness that is accounted into the calculation, damping, distribution of shear forces, proper spectra's that are taken and other problems which are often not recognized by engineers.*

Key words: *Force based design, direct displacement based design, differences*

RAZLIKE IZMEĐU PRORAČUNA BAZIRANOG NA SILAMA I BAZIRANOG NA POMIJERANJIMA

Rezime: *Ovdje su poređene dvije metode seizmičkog dizajna: tradicionalni-koji proračun baziran na silama i novi-koji proračun bazira na pomijeranjima. U radu koji slijedi ukratko su opisane i date osnovne razlike između ove dvije filozofije kao i konceptualni problemi sa kojima se susreće proračun baziran na silama, počevši od krutosti koju uzima u proračun, pa preko prigušenja, raspodjele smičućih sila, odgovarajućih spektara I drugih problema koji često nisu ni prepoznati od strane inženjera. .*

Ključne reči: *Proračun baziran na silama, direktan proračun baziran na pomijeranjima,razlike*

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1. THE INTRODUCTION

In recent years, many new approaches for assessment and design of structures have been presented. They are very opposite to well-known forced based design and they criticize the old method but it should be mention that: this “old” approach is still used by the most civil engineers, and that forced based design is perfectly acceptable approach that can be expected to result in safe strictures, but doesn’t directly address performance criteria at the initial stage of the design. Considering all this, current design approaches try to modify existing force-based design to include displacement consideration, rather than rework procedure on a more rational displacement basis. However, there are some conceptual and philosophical problems associated with force-based/displacement based design:

1. Initially, earthquakes were considered in terms of inertia forces. Also the engineers are more comfortable designing for forces rather than for displacements. For elastic response force and displacement are directly equivalent. But for inelastic response, a displacement approach is more logical.
2. In DDBD there is no additional force at the roof level what is commonly adopter in force based design, but recent studies for tall flexible frames indicate improved performance if 10% of the base shear is located at roof level.
3. While force based design use elasting damping, DDBD (Direct Displacement Based Design) characterize the structure by equivalent viscous damping that is combined elastic damping and the hysteretic energy absorbed during elastic response.
4. Traditional seismic design has been based on the acceleration spectrum, Figure 1. and the direct displacement based design (DDBD) is based on displacement spectrum, Figure 1.

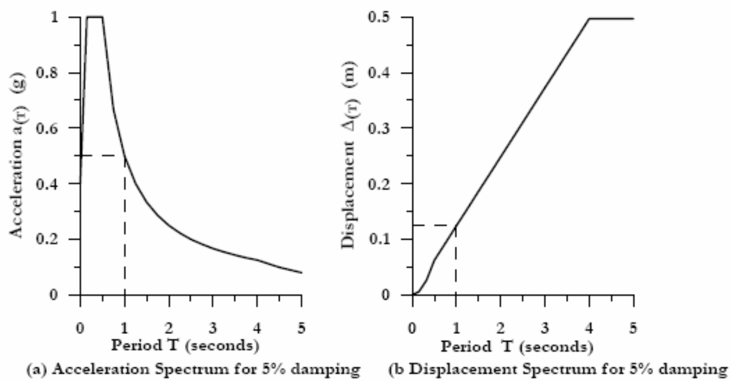

Figure 1. Equivalent acceleration and displacement response spectrum

This is the major difference between these two approaches. Displacement-period spectrum is given for the different levels of equivalent viscous damping, and acceleration-period spectrum for 5% damping, see Figure 2.

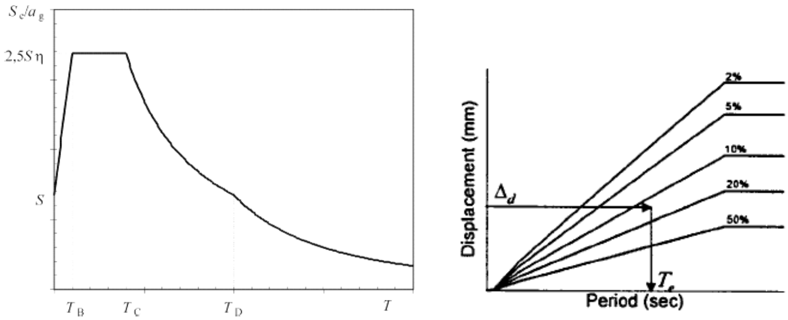

Figure 2. The acceleration response spectrum for 5% damping and displacement response spectrum for 2,5,10,20 and 50% damping

5. Since the structural period of the substitute structure is no longer than that for the elastic structure ($T_e=\mu^{1/2}\,T_i$ where T_i is the initial, elastic period) it is necessary for the displacement spectra to continue to longer periods than that of acceleration spectra. It is also appropriate to place a cap on peak response displacement since at long periods, structural displacements tends to reduce usually equaling to the peak ground displacement.
6. Although the engineers are not comfortable with a new approach, the procedure for performing displacement based design is simpler and requires less steps compared to forced based design. For elastically SDOF the response acceleration $a_{(T)}$, corresponding to the fundamental period T is found and the corresponding force F and displacement also Δ. So, for forced based design there are 5 operational steps, and it is necessary to calculate stiffness, period, acceleration, force and displacement.

   $$F = m \cdot a_{(T)} \cdot g \qquad \Delta = F / k$$

   where k is the system stiffness, m is the system mass and g is acceleration due to gravity.

   During the displacement based design, there is one operational step less because the displacement spectrum could be used directly: it is needed to calculate only stiffness, period, force and displacement.

   $$F = k \cdot \Delta_{(T)}$$

   where $\Delta_{(T)}$ is response displacement.

   In both cases the elastic period must be first calculated, but in displacement approach the mass is not needed
7. Forced based design requires the specification of initial stiffness of structural members, and forces are distributed between them on the basis of initial stiffness. It is also used for the estimation of the period. This is sometimes taken to be gross stiffness and sometimes as a reduced one to represent the influence of cracking in concrete and masonry structures. In displacement based design, the effective stiffness that corresponds to ultimate displacement is the one that is used during the calculation, what is illustrated in Figure 3.

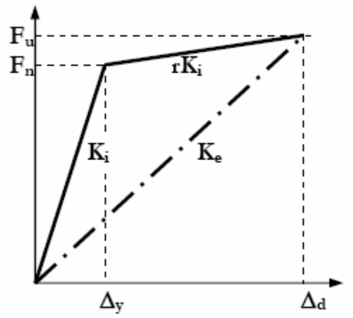

Figure 3. Definition of initial and effective stiffness

8. In force based design, if the dimensions of the structure are known, the stiffness can be directly estimated and it is independent of strength and that yield displacement or yield curvature is directly proportional to strength as shown in Figure 4a. But this assumption is invalid: stiffness is directly proportional to strength and the yield displacement or curvature is independent from strength as shown in Figure 4b.

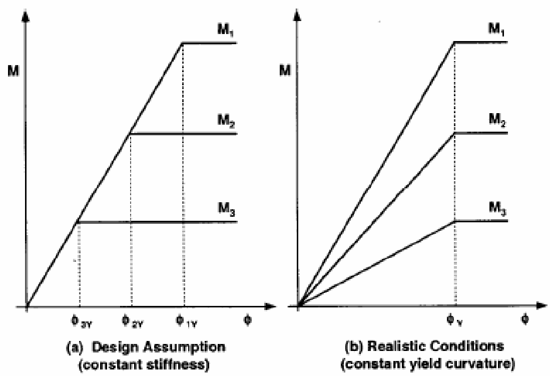

Figure 4. Influence of strength on moment-curvature relationship

9. As a consequence of these findings it is not possible to perform the accurate analysis or to define the structural period until the member strengths have been defined. Since these aspects are fundamentals for force-based design some iteratiation must be carried out before the adequate elastic characteristics of the structure have been found. This will increase the design effort. It should be noted that the elastic characteristics of the building is not the best indicator of its inelastic performance.
10. At the same time it is clear that the increasing the computational effort of the design by 3D analysis is not likely to result in better characterization of structural response then more simplified SDOF representation unless the structural response is elastic.
11. If there is a building with a number of shear walls of different length, the conventional force based design would apportion the base shear between walls in proportion to the cube of he wall length. This is based on invalid assumption that the walls could achieve simultaneous yield. The consequence of this approach is

that the longer walls will have more heavily reinforcement for flexure than the shorter walls. Rational decision is to apportion the base shear between the walls in proportion to the square of the length, what will further on result in constant reinforcement ratios between the walls.

12. With some difficulties it is possible to incorporate the influence of the foundation flexibility, but it is rarely checked. In displacement based design, the design displacement will be increased by the elastic displacement if the limit state being considered is strain limited, what is not the case if it is code drift limited.
13. From the point 8. yields the influence on effective damping: in DDBD, both foundation and the structure will contribute to the damping, as presented on following Equitation's and Figure 5.

$$\xi_f = \frac{A_f}{2\pi V_B \Delta_f}$$

$$\xi_S = \frac{A_S}{2\pi V_B \Delta_S} \text{ and}$$

$$\xi_e = \frac{A_f + A_S}{2\pi V_B (\Delta_f + \Delta_S)} = \frac{\zeta_f \Delta_f + \zeta_s \Delta_S}{\Delta_f + \Delta_S}$$

where ζ_f, ζ_S and ζ_S are damping of the foundations, structure and system, A_f and A_S are hysteretic areas within the loop for foundation and structure respectively.

So, the peak response displacement will be: $\Delta_d = \Delta_s + \Delta_f$ where the displacement Δ_s is the component from the structure and the displacement Δ_f the component from the foundation.

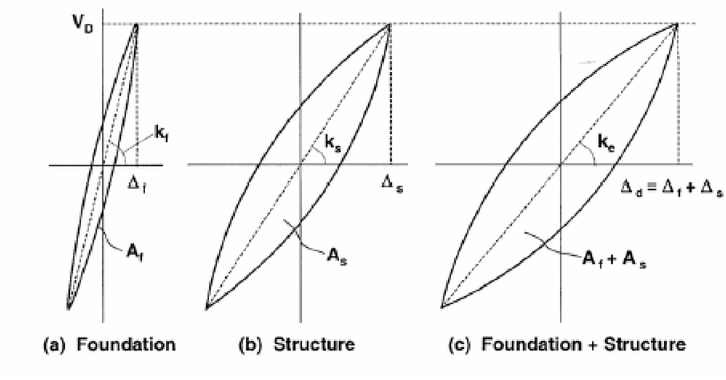

Figure 5. Damping contribution on foundation and structure

14. In a force-based design there are no uniform risk structures, this methodology keep the risk of a structure below an acceptable threshold although undefined. But it can be shown that risk (the annual probability of reaching a given level of damage) for structures designed to force based criteria can very from structure to structure. Thus two different structures designed to the same code and with the same force-reduction or ductility factors may experience different levels of damage under the same earthquake.

15. There is different structural sensitivity to seismic intensity related to DDBD and force based design. Figure 6. illustrates acceleration and displacement spectra for two seismic zones with different intensities but with assumption that the spectral shapes are identical and that the design spectra in different zones is found by multiplying a base spectrum by the zone intensity factor Z. The assumption is that the structures are designed to satisfy the seismic requirements for both zones, also assuming that the structural geometry (member sizes but not the content of the reinforcement) is the same for the two buildings. Force based design is giving the same fundamental periods for both buildings. So, required base shears V_{b1} and V_{b2} will be related by:

$$V_{b2} = V_{b1}\frac{Z_2}{Z_1}$$

If it is assumed that the geometry of the buildings is the same direct displacement based approach will give the same yield displacements and the limit state design displacement. Hence, the ductility and the effective damping will be the same too, but different effective periods related to the intensity of the zones:

$$T_{b2} = T_{b1}\frac{Z_1}{Z_2}$$

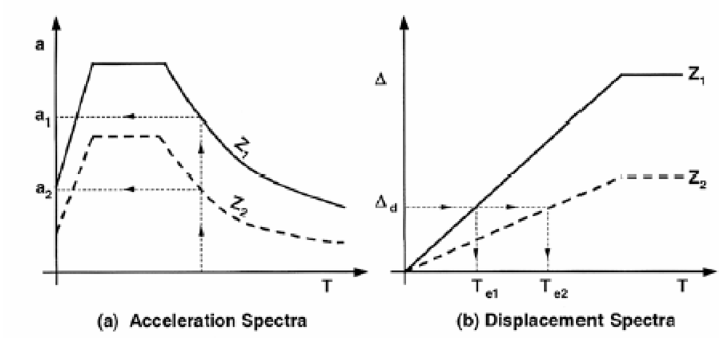

Figure 6. Influence of seismic intensity on design base shear force

Having on mind that the effective stiffness k_e is:

$$k_e = \frac{4\pi^2 m_e}{T_e^2}$$

where m_e is the effective mass of the structure participating in the fundamental mode of vibration, now we have this relation:

$$k_{e2} = k_{e1}\left(\frac{Z_2}{Z_1}\right)^2$$

Further on, the ratio of base shear forces if the design displacements are the same (and they are) will be:

$$V_{B2} = V_{B1}\left(\frac{Z_2}{Z_1}\right)^2$$

what means that the base shear is not linear proportional but proportional to the square of the seismic intensity.

As a summarization of all these (similarities and) differences, there is a Table 1. where are stressed out the most important ones.

FORCE BASED DESIGN	DISPLACEMENT BASED DESIGN
Effect of earthquakes explained thru inertia forces	Effect of earthquakes explained thru displacements
Use initial stiffness	Use secant stiffness
Use acceleration response spectrum	Use displacement response spectrum
Elasting damping	Equivalent viscous damping
There is additional force at the roof level	There is no additional force at the roof level
Five (5) steps till the end of designing process	Four (4) steps till the end of designing process
Need the mass for the calculation of the elastic period	Doesn't need the mass for the calculation of the elastic period
It is not necessary to plot longer periods in spectra	It is necessary to plot longer periods in spectra
Very rarely incorporate the influence of the foundation flexibility	Incorporate the influence of the foundation flexibility if the limit state being considered is strain limited
Non structural damping force is not force dependant	Non structural damping force is displacement dependant
Linear proportional of the seismic intensity	Proportional to the square of the seismic intensity
Base shear proportional to the cube of the shear wall length	Base shear proportional to the square of the shear wall length
The stiffness is independent of strength	The stiffness is directly proportional to strength
No uniform risk structures	Uniform risk structures

Table 1. Differences between force-based and direct displacement based design

2. REFERENCE

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TENTH NATIONAL AND
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iNDiS 2006

NOVI SAD, NOVEMBER 22-24,2006.

Venera Vukašinović[1]

ADAPTIVE CAPACITY SPECTRUM METHOD

Summary: *The re-examination of the fundamentals of seismic design has intensified in recent years, with a great number of conflicting approaches being advocated. One of the major developments in seismic design over the past 10 years has been increased emphasis on limit state design, now generally termed Performance Based Engineering. One of the techniques are: the Capacity Spectrum Approach, the N2 method, Direct Displacement Based Design .and so on. Here will be explanted the fundamentals of he Adaptive Capacity Spectrum Approach.*

Key words: *Capacity Spectrum, Adaptive Approach*

SPEKTAR KAPACITETA: ADAPTIVNA METODA

Rezime: *Preispitivanje osnova seizmičkog dizajna se naročito intenziviralo u zadnjih nekoliko godina, sa velikim brojem sukobjavajućih pristupa. Jedan od najvećih razvića u polju seizmičkog dizajna koji se zbio u zadnjih 10 godina je to što je dao naglasak na proračun po graničnim stanjima, koja su danas oslovljana kao "inženjerstvo bazirano na performancama".Neke od tehnika su: metoda spectra kapaciteta, N2 metoda, proračun baziran na direktnim pomijeranjima i druge. U ovom radu biće izložene osnove adaptiranog pristupa metodi spektralnog kapaciteta.*

Ključne reči: *Spektar kapaciteta, adaptivni pristup*

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1. THE METHODOLOGY

Pushover is simply an analysis algorithm that allows the computation of the capacity of the structure (unlike nonlinear dynamic analysis where both demand and capacity are estimated). Hence, the application of pushover analysis in the assessment of existing structures requires its introduction within a process that allows the estimation of the demand, against which the capacity is then compare with.

Several important methods have been proposed recently, such as:

1. Capacity Spectrum Method (Freeman, ATC-40),
2. N2 Method (Fajfar, EC 8),
3. Modal Pushover Analysis (Chopra and Goel),
4. Displacement Coefficient Method (Miranda, FEMA-356),
5. etc.

The procedure that describes the Capacity Spectrum Method is consisted from these following steps:

1. Create the structural model,
2. Determine adaptive capacity curve of equivalent SDOF system:
   2.1. Run pushover analysis (preferably adaptive),
   2.2. Define equivalent system displacements $\Delta_{sys,k}$ at every analysis step,

$$\Delta_{sys,k} = \frac{\sum_i m_k \Delta_{i,k}{}^2}{\sum_i m_k \Delta_{i,k}}$$

where is:

m_i floor mass,

$\Delta_{sys,k}$ floor displacement at step k

   2.3. Define equivalent system mass $M_{sys,k}$ at every analysis step,

$$M_{sys,k} = \frac{\sum_i m_k \Delta_{i,k}}{\Delta_{sys,k}}$$

   2.4. define equivalent system acceleration $S_{a\text{-}cap,k}$ at every analysis step,

$$S_{a-cap,k} = \frac{V_{b,k}}{M_{sys,k}\, g}$$

where is:

$V_{b,k}$ base shear at step k,

g acceleration of gravity.

Determination of these sub-steps is presented in Figure 1.

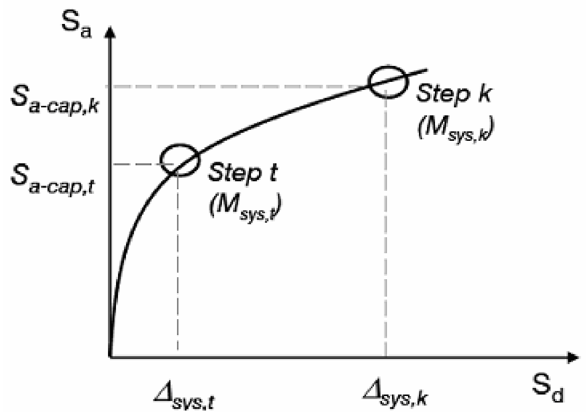

Figure 1. Diagram acceleration-displacement (S_d-S_a) at different steps: t and k

3. Computation of Performance Point using the demand spectrum:
   3.1. Assume a given initial viscous damping value (e.g.10%) and calculate its corresponding spectral reduction factor:

$$\eta_{iniz} = \sqrt{\frac{10}{5+\xi_{iniz}}} \geq 0{,}55$$

   3.2. Represent, in the same plot (Figure 2.), the damped response spectrum and the capacity curve, so as to individuate the Performance Point-PP as intersection of two curves,

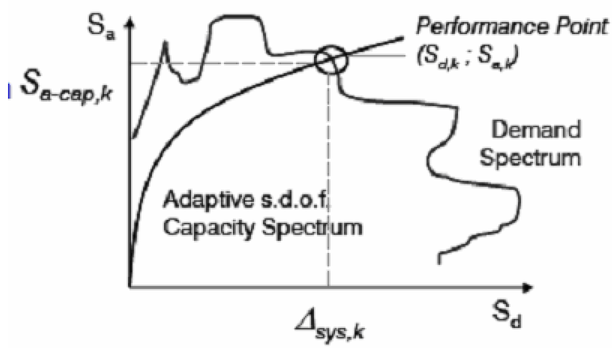

Figure 2. Defining of Performance Point-PP

   3.3. Bi-linearise the capacity curve using approach of equal areas, as given in Figure 3.

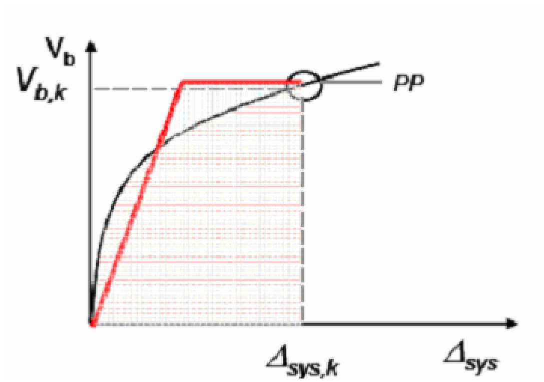

Figure 3. Bi-linearization of capacity curve

3.4. Calculate ductility μ_{sys} and effective equivalent viscous damping $\xi_{sys,eff}$ at Performance Point. There are many ways to calculate the damping, one of them is:

$$\xi_{sys,eff} = 5 + \frac{100}{\pi}\left(1 - \frac{1}{\sqrt{\mu_{sys}}}\right)$$

3.5. Check if obtained damping is equal to that initially assumed. If:

$\xi_{sys,eff} = \xi_{iniz}$ the Performance Point is well defined,

$\xi_{sys,eff} \neq \xi_{iniz}$ restart from 3.1. with $\xi_{sys,eff} = \xi_{iniz}$ until convergence in dumping values. Few iterations (typically 3) are usually suffice.

4. Computation of response quantities of interest, in correspondence to the Performance Point deformation level (see Figure 4.)

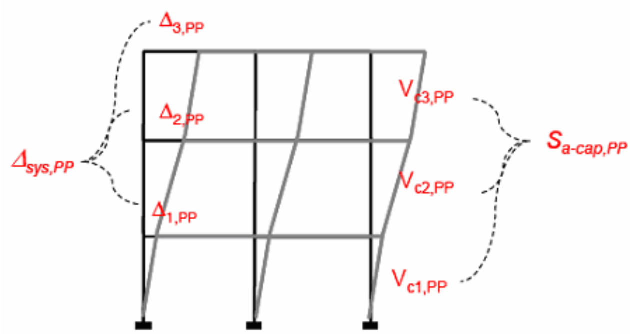

Figure 4. Response quantities of interest in correspondence to the PP

After basic theoretical steps, herein will be exposed one practical example, dealing with structural model consisted from 3 stories with 10 meters height totally. Performance level (target) was collapse limit state.

1. Determine adaptive capacity curve of equivalent SDOF system,
2. Run pushover analysis (preferably adaptive):

.oad Factor	n121	n131	n141	n221	n231	n241	n321	n331	n341	Total Support Force
	SPOSTAMENTI									TAGLIO ALLA BASE
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-0.009
0.001	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	-8.183
0.002	0.000	0.001	0.001	0.000	0.001	0.001	0.000	0.001	0.001	-16.336
0.003	0.000	0.001								
0.004	0.001	0.002								
0.005	0.001	0.002								
0.006	0.001	0.003								
0.007	0.001	0.003								
0.008	0.001	0.003								
0.009	0.001	0.004								

TAGLI SULLE COLONNE

col1111	col1211	col1311	col2111	col2211	col2311	col3111	col3211	col3311
1.618	2.524	6.575	-0.004	-0.009	0.078	-1.623	-2.546	-6.434
-1.010	0.301	5.417	-2.957	-2.562	-1.410	-4.237	-4.763	-7.593
-3.632	-1.923	4.257	-5.902	-5.113	-2.901	-6.843	-6.980	-8.754
-6.254	-4.147	3.098	-8.845	-7.663	-4.391	-9.447	-9.195	-9.915
-8.876	-6.370	1.940	-11.786	-10.211	-5.880	-12.048	-11.408	-11.075
-11.498	-8.592	0.782	-14.724	-12.756	-7.367	-14.646	-13.620	-12.235
-14.119	-10.814	-0.374	-17.659	-15.299	-8.854	-17.241	-15.830	-13.394
-16.740	-13.034	-1.529	-20.592	-17.840	-10.339	-19.833	-18.037	-14.552
-19.360	-15.254	-2.683	-23.521	-20.378	-11.824	-22.422	-20.243	-15.709
-21.980	-17.474	-3.836	-26.448	-22.914	-13.306	-25.008	-22.446	-16.866

3. Define displacement, mass and acceleration of the equivalent system at every analysis step,

Masse	(t)							
n121	n131	n141	n221	n231	n241	n321	n331	n341
10	10	10	20	20	20	10	10	10
				mΔ				
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0.001	0.004	0.005	0.003	0.008	0.010	0.001	0.004	0.005
0.003	0.008	0.010	0.006	0.017	0.020	0.003	0.008	0.010
0.004	0.013	0.015	0.008	0.025	0.030	0.004	0.013	0.015
0.006	0.017	0.020	0.011	0.034	0.040	0.006	0.017	0.020
0.007	0.021	0.025	0.014	0.042	0.050	0.007	0.021	0.025
0.008	0.025	0.030	0.017	0.051	0.060	0.008	0.025	0.030
0.010	0.030	0.035	0.019	0.059	0.070	0.010	0.030	0.035

Adapt Cap. Spec.

Δ_{eq}	M_{eq}	A_{eq}
0.000	71.630	0.000
0.000	100.694	0.081
0.001	100.625	0.162
0.001	100.602	0.243
0.002	100.591	0.324
0.002	100.585	0.405
0.003	100.581	0.486

4. Computation of Performance Point, using demand spectrum:
   4.1. Initial damping value was proposed of 10%→η=0,52
   4.2. Performance Point, Figure 5.

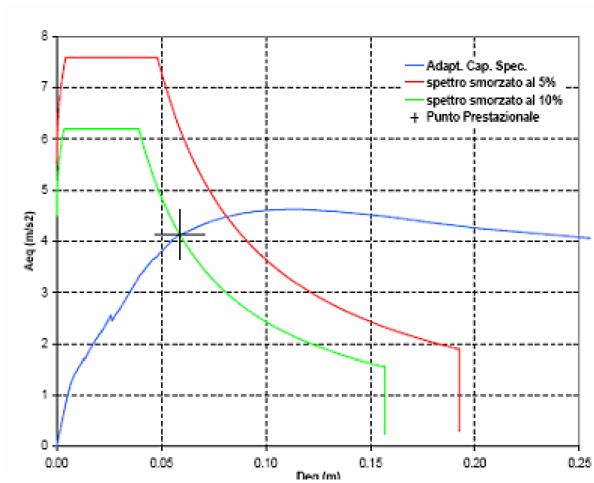

Figure 5. Defining of Performance Point

4.3. Bi-linearization of capacity curve, Figure 6.

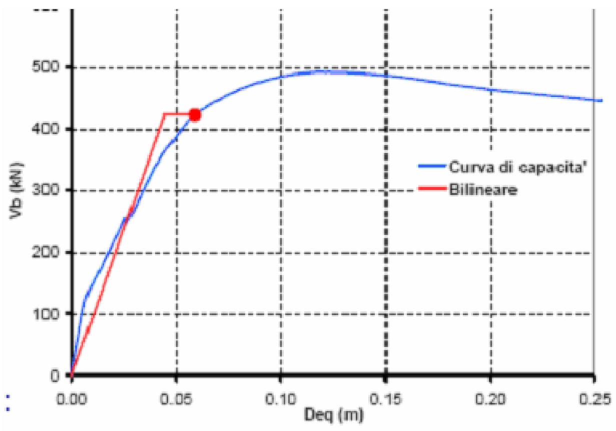

Figure 6. Bi-linearization of capacity curve

4.4. Ductility and damping at PP:

$$\mu_{sys} = 1{,}32$$

$$\xi_{sys,eff} = 9{,}14\% \neq 10\%$$

4.5. Restart from with new damping until convergence in damping values… and upon convergence:

$$\mu_{sys} = 1{,}32$$

$$\xi_{sys,eff} = 9{,}37\%$$

Load factor obtained in correspondence to Performance Point PP = 0,142.

5. Determine all response quantities of interest in the analysis step corresponding to the load factor = 0,124:

UNITS: m, kN, t, m/s2

Load Factor	n121	n131	n141	n221	n231	n241	n321	n331	n341	Total Support Force
	SPOSTAMENTI									TAGLIO alla BASE
...	...	...	...	...	...	...	...	...	...	...
0.138	0.020	0.062	0.069	0.021	0.062	0.069	0.024	0.062	0.069	-424.127
0.139	0.020	0.062	0.069	0.021	0.062	0.069	0.024	0.063	0.069	-425.328
0.140	0.020	0.062	0.070	0.021	0.063	0.070	0.024	0.063	0.070	-426.226
0.141	0.020	0.063	0.070	0.022	0.063	0.070	0.025	0.064	0.070	-427.126
0.142	*0.021*	*0.063*	*0.071*	*0.022*	*0.064*	*0.071*	*0.025*	*0.064*	*0.071*	*-428.116*
0.143	0.021	0.064	0.071	0.022	0.064	0.071	0.025	0.065	0.071	-428.875
0.144	0.021	0.064	0.072	0.022	0.065	0.072	0.025	0.065	0.072	-429.883
...	...	...	...	...	...	...	...	...	...	...

col1111	col1211	col1311	col2111	col2211	col2311	col3111	col3211	col3311
TAGLI INTER-PIANO								
...	...	...	...	...	...	...	...	...
-107.052	-93.256	-38.318	-174.510	-120.829	-62.125	-146.325	-92.133	-35.726
-107.520	-93.391	-38.170	-175.076	-121.084	-62.171	-146.535	-92.341	-35.741
-107.841	-93.547	-38.275	-175.521	-121.280	-62.318	-146.707	-92.600	-35.777
-108.270	-93.702	-38.400	-175.828	-121.489	-62.477	-146.912	-92.818	-35.798
-108.729	*-93.853*	*-38.529*	*-176.194*	*-121.687*	*-62.662*	*-147.117*	*-93.033*	*-35.816*
-108.956	-94.003	-38.683	-176.567	-121.735	-62.863	-147.319	-93.240	-35.805
-109.423	-94.150	-38.844	-176.938	-121.965	-63.050	-147.532	-93.448	-35.827

6. Determination of displacements, inters story drifts and base shear, as presented in Figure 7 and Table 1:

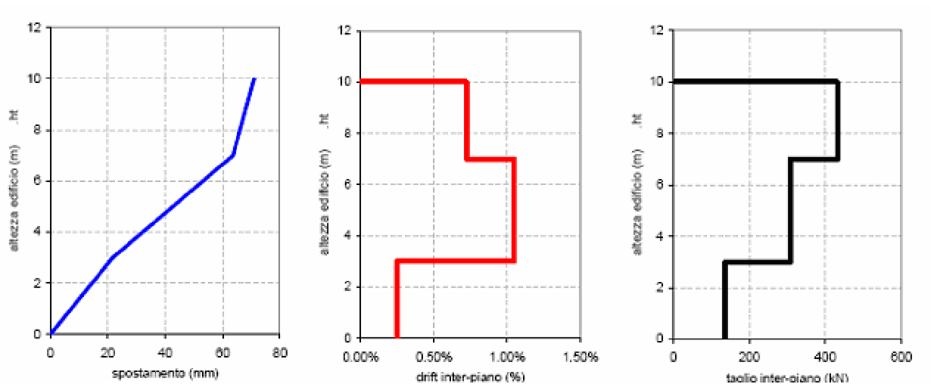

Figure 7. Graphical presentation of obtained displacements, interstory drifts and interstory shear forces

Story	Displacement (mm)	Inter-story drift (%)	Inter-story forces (kN)
First	70,96	0,24	432,04
Second	63,62	1,05	308,57
Third	21,71	0,72	137,01

Table 1. Numerical values of displacements, inter-story drifts and shear forces

2. REFERENCE

1. The manuscripts from the course “Advanced Earthquake Analysis” that was held in Italy, Udine, July 2006.

TENTH NATIONAL AND
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iNDiS 2006

NOVI SAD, NOVEMBER 22-24,2006.

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Dan Dubina[2]

FIRE DESIGN OF STEEL AND COMPOSITE STRUCTURES USING THE EUROCODES

Summary: *Lately, there is a growing demand in Romania for steel structures, especially for industrial and commercial objectives, where erection speed is critical in the choice of structural solution. The main problem of steel structures is their low fire resistance. Composite steel-concrete solutions have the advantage of increased resistance in case of fire. The fire resistance of steel and composite structures may be determined using simplified methods, based on analytical formulas or tables, provided in the corresponding Eurocodes for fire design. For special situations or for complex structures it may be necessary to perform an advanced analysis, using special purpose programs for the analysis of structures under ambient and elevated temperature conditions, as well as to establish a realistic fire scenario, based on „natural fire" models. The paper presents the principles of fire design and gives some examples of application by the authors of the advanced methods, for some existing buildings in Romania.*
Key words: *fire design, steel, composite steel-concrete, advanced analysis*

PRORAČUN ČELIČNIH I KOMPOZITNIH KONSTRUKCIJA NA POŽAR KORIŠĆENJEM EVROKODOVA

Rezime: *U poslednje vreme prisutan je povećan zahtev za čeličnim konstrukcijama u Rumuniji, pogotovo za industrijske i poslovne objekte, gde je vreme gradnje kritično za izbor rešenja konstrukcije. Glavni problem čeličnih konstrukcija je njihova mala požarna otpornost. Rešenja sa kompozitima čelik-beton imaju prednost zbog povećane otpornosti u slučaju požara. Požarna otpornost čelika i kompozitnih konstrukcija se može odrediti korišćenjem jednostavnih metoda, baziranih na analitičkim formulama ili tabelama, koje su date u Evrokodu za proračun na dejstvo požara. U specijalnim situacijama ili za kompleksne konstrukcije može biti potrebno da se sprovedu neke napredne analize, korišćenjem specijalizovanih programa za analizu konstrukcija u uslovima ambijentalnoj i više temperature, kao i da se uspostavi realan požarni scenario, baziran na modelima "prirodnog" požara. U radu su prikazani principi proračuna na požar i dati su neki primeri primene autora napredne analize za neke postojeće objekte u Rumuniji.*
Ključne reči: *proračun na požar, čelik, kompozit čelik-beton, napredne analize*

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1. INTRODUCTION

According to the Directive of the European Commission issued on 21 December 1988, the construction works must be designed and built in such way that in the event of an outbreak of fire:

- the load bearing capacity of the construction can be assumed for a specific period of time;
- the generation and spread of fire and smoke within the works are limited;
- the spread of fire to neighbouring construction works is limited;
- occupants can leave the works or be rescued by other means;
- the safety of rescue teams is taken into consideration.

In order to reach these objectives, passive and active protection measures are necessary. The active measures are generated by all the devices activated in case of fire, as for example the automatic extinguishing systems. Concerning the passive measures, the following may be highlighted: the adequate number of escape routes function of the specific and particularities of the building, the elimination and the protection of the potential fire sources, the limitation of fire spread through an adequate partitioning of the building and, last but not least, the appropriate design of the resistant structure of the building, in order to maintain its strength and stability under elevated temperatures, for a specified period of time. The paper presents the principles of fire design and gives some examples of application by the authors, for some existing buildings in Romania

2. PRINCIPLES OF FIRE DESIGN

The basic principle in determining the fire resistance of a structural element is that the elevated temperatures produced by the fire reduce the materials strength and stiffness until possible collapse. Figure 1 shows the reduction factors for the stress-strain realtionship of carbon steel at elevated temperatures. When the temperatures on the cross-section of a structural element produce the reduction of the element resistance bellow the level of the effect of actions for fire design situation, it is considered that that element lost its load-bearing function under fire action.

The fire resistance of steel or composite steel-concrete structures is calculated according to EN1993-1-2 [5] and EN1994-1-2 [6] respectively. Three methods are available in order to evaluate the fire resistance: the tabulated data method, the simple calculation models and the advanced calculation models.

The tabulated data method is based on observations resulted from experimental study. It is the easiest to apply, but it is limited by the geometrical conditions imposed to the composite cross-section. The tabulated data method is not available for steel structures.

The simple calculation models compute the ultimate load of the element by means of formulas or design charts, established on the basis of experimental data.

The advanced calculation models supose an advanced numerical analysis of the elements or of the entire structure under fire, using specialized software for the mechanical analysis of structures under elevated temperatures.

There are several fire models, accepted by the European Standard EN1991-1-2 [3], which describes the thermal and mechanical actions to be must be considered for a structure under fire.

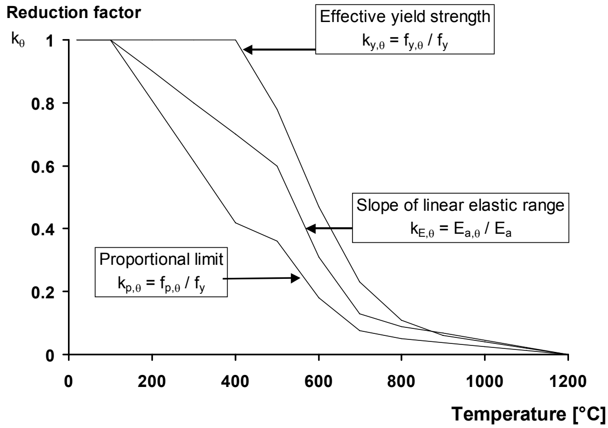

Figure 1. Reduction factors for carbon steel at elevated temperatures

The nominal standard temperature-time ISO model does not take into account any physical parameter, and can be far away from reality. From the beginning, the nominal model supposes that the entire compartment is in the flashover phase and the temperature is increased continuously, without taking into account the cooling phase.

The parametric fire model considers the cooling phase and gives the temperature-time curve function of the fire load density and openings. This model is, however, limited to the surface and the height of the fire compartment considered, and supposes that the temperature is the same on the entire compartment, from the beginning of the fire.

A modern fire model approach is the combined `Two Zone` and `One Zone` model. In this natural fire model, in the pre-flashover phase, the fire compartment is divided in a hot upper zone and a cold inferior one. For each zone, with uniform temperature, mass and energy equations are solved. Complex equations describe the air movement in the fire plume, the radiative exchanges between the zones and the gas movements on the openings and adjacent compartments. After the flashover, the temperature is considered uniform and is determined by solving the equations of mass and energy of the compartment, taking into account the walls and openings. In the frame of the ECSC research "Natural Fire Safety Concept" [1] it was considered necessary to develop a computer program for this model. This objective is now reached, a computer program called ***OZone*** is available in order to determine the temperature-time curve by means of the `Two Zone` and `One Zone` concept, and was built at the Liege University, Belgium, in collaboration with the “Politehnica” University of Timisoara.

The fire is considered an accidental situation which requires, with some exceptions, only verifications against the ultimate limit state. The combinations of actions for accidental design situations are given in the European Standard for basis of structural design EN1990 [2] by the following formulas:

$$G_k + P_k + \Psi_{1,1} Q_{k,1} + \sum_{i>1} \Psi_{2,i} Q_{k,i} \qquad \text{or} \qquad G_k + P_k + \sum_{i\geq 1} \Psi_{2,i} Q_{k,i} \tag{1-2}$$

G_k, P_k, Q_k are the characteristic values of the permanent, variable and prestressing action. According to the European Standard for actions on structures exposed to fire

EN1991-1-2 the representative value of the variable action Q_1 may be considered as the quasi-permanent value $\Psi_{2,1}Q_{k,1}$, or as an alternative the frequent value $\Psi_{1,1}Q_{k,1}$.

The following verifications are possible for a steel or composite steel-concrete structure, substructure or element, as considered, in fire situation:

- Verification in the time domain, in which is has to be verified that the time of failure t_f is higher than the required fire resistance time t_{req}:

  $$t_f > t_{req} \tag{3}$$

  The failure time is the time for which the resistance of the structure (or substructure, or element, as considered) under elevated temperatures reach the effect of actions for the fire design situation, considering the combinations of actions presented above.
- Verification in the load domain, in which it has to be verified that the resistance of the structure (or substructure, or element, as considered) for the fire design situation at time t_{req}, $R_{fi,d}$, is higher than the effect of actions for the fire design situation, $E_{fi,d}$.

  $$R_{fi,d} > E_{fi,d} \text{ for } t=t_{req} \tag{4}$$

  This is the standard verification proposed in both EN1993-1-2, EN1994-1-2 [5,6].
- As an alternative, the verification may be carried out in the temperature domain, in which it has to be verified that, at the corresponding required fire resistance time t_{req}, the temperature of the structure θ_a (or substructure, or element, as considered) is lower than the temperature that leads to the failure, called critical temperature $\theta_{a,cr}$.

  $$\theta_a < \theta_{a,cr} \text{ for } t=t_{req} \tag{5}$$

  for $t=t_{req}$

  This verification is available for some limited situations. For steel structures, this verification may be performed for cases in which deformation criteria or stability phenomena have not to be taken into account.

3. EXAMPLES OF APPLICATION FOR SOME BUILDINGS IN ROMANIA

In the following, two examples of fire design made by the authors for buildings in Romania are presented. Advanced calculation models were considered, by means of the SAFIR computer program [7], which is a special purpose program for the analysis of structures under ambient and elevated temperature conditions. The program, developed at the University of Liege, accommodates various elements for different idealization, calculation procedures and various material models for incorporating stress- strain behavior under elevated temperatures. The analysis of a structure exposed to fire consists of two steps. The first step involves predicting the temperature distribution inside the structural members, referred to as 'thermal analysis'. The second part of the analysis, termed the 'structural analysis' is carried out for the main purpose of determining the response of the structure due to static and thermal loading.

The first example [9] presents the calculation of the fire resistance for the columns of a three-storey framed building structure for the LINDAB-Romania Company Headquarters, in Bucharest. The office structure has three levels (ground floor, 1st floor, and an attic floor), two spans of 6m each, and 7 bays of 5m, with a total area of 1308 m^2. Taking into account the specific of LINDAB – Romania (systems of steel industrial buildings) the special demand was that the resistance structure must be visible steel, made by circular columns. Because for this type of building, according to Romanian fire regulations, the columns must have 2 hours of fire resistance, the solution of reinforced concrete filled CHS columns was chosen.

Columns concrete filled CHS cross-section is presented in Figure 2, while Figure 3 presents the numerical model of this section. Due to obvious reasons of symmetry, only a quarter of the cross-section was represented. The round reinforcing bars are represented by quadrilateral elements, with equivalent area. Figure 4 shows the temperature distribution after 2 hours of standardized ISO fire. It may be observed that for the CHS profile, the temperatures are superior of 1000^{o}C, so the steel profile exhausted its loading capacity. In the same time, the temperatures of the reinforcing bars are around 500^{o}C and there is an important core of concrete with quite low temperatures.

The column, loaded with the axial force and bending moment, corresponding to the fire situation, was modeled with 2D beam elements. Conservatively, the buckling length of the column was considered as the system length (L_{ei}=1.00L), i.e. the height of the relevant storey. Equivalent imperfections according to EN1993-1-1 [4] were considered. As the characteristic time-displacement demonstrates (Fig.5), after 2 hours of ISO fire (7200 sec) the column is still able to resist to the imposed static loads, due to the bearing capacity reserve provided by the concrete core and the reinforcing bars. The collapse of the column is produced after around 3h of ISO standardized fire.

The second example presents the calculation of the fire resistance for a composite column of „Bucharest Tower Centre" structure, which will be the tallest building in Bucharest for the moment. The building has 3 basements, one ground floor, 21 floors, 3 technical floors at a total height of 106.3m. The columns are made by cruciform cross sections made of hot rolled european profiles, partially encased in reinforced concrete, in order to increase strength, rigidity and fire resistance. According to Romanian fire regulations, considering the specific and particularities of the building, the columns must have 2 ½ hours of fire resistance. The example presented here is part of an ongoing study. Figure 6 and 7 show, respectively, the cross section of one of the columns of the building, with european profiles HEB1000 and HEB500, and the numerical model of this cross-section. Due to symmetry of the cross-section, as in the previous example, only a quarter of the cross-section was represented. The round reinforcing bars are represented by quadrilateral elements, with equivalent area.

Figure 8 shows the temperature distribution after 2 ½ hours of standardized ISO fire. It may be observed that the steel profiles flanges exhausted practically their load capacity, having temperatures greater than 900 ^{o}C, while the profiles webs and the reinforcing bars have much lower temperatures and there is an important core of concrete with quite low temperatures. Consequently, after 2 ½ hours of standardized ISO fire, the section has a reserve of load capacity.

The column, loaded with the axial force and the bending moments on both principal cross-section axes, efforts corresponding to the fire situation, was modeled with 3D beam elements. As in the previuos example, the buckling length of the column was considered as the system length (L_{ei}=1.00L), and equivalent imperfections according to EN1993-1-1 were considered on both directions of the principal cross-section axes. As the characteristic time-displacement demonstrates (Fig. 9) after 2 ½ hours of ISO fire (9000 sec) the column of the ground floor, with a height of 5.4m, does not resist to the imposed static loads, having a resistance time of 143 minutes. At the level of first floor, the column with the same cross-section, but having a height of 4.2m, presents, under the imposed static loads (lower than for the ground floor) a resistance time of 189 minutes and thus fulfills the requested resistance time of 2 ½ hours (Fig. 10). All the columns from the upper storeys, with the same composite cross-section, present a better resistance time due to the lower imposed static loads and also due to the reduced height (4.0m from the fourth floor). Consequently, an adequate fire protection is requested, for this type of composite cross-section, only for the columns of the ground floor. The ongoing study seems to generalise this conclusion for all the composite columns of the building.

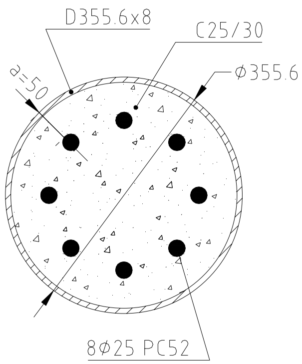

Fig. 2 Composite CHS column

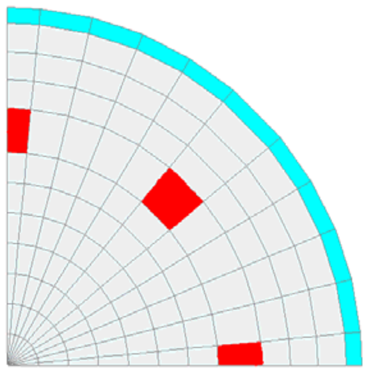

Fig. 3 Numerical model of composite CHS

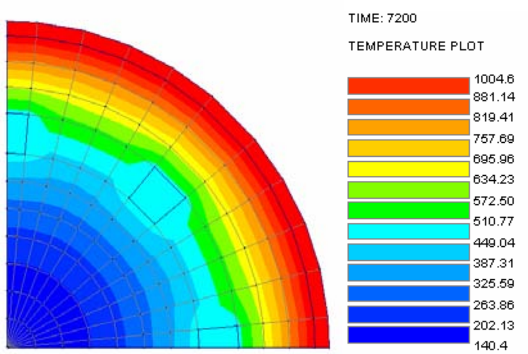

Fig. 4 Temperature distribution after 2h of ISO fire in composite CHS

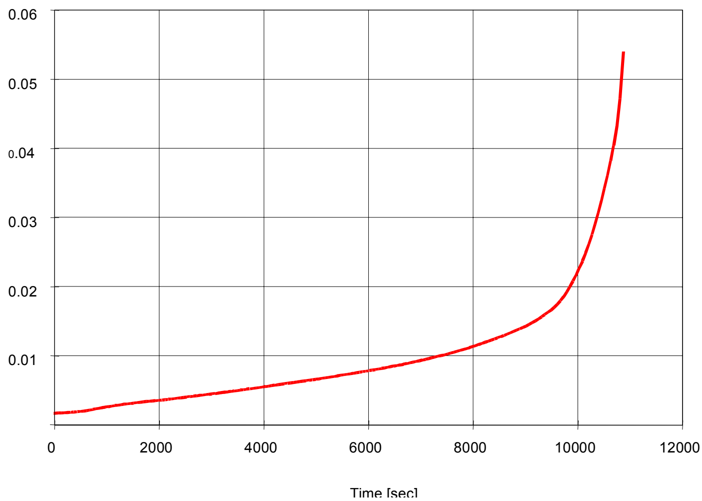

Fig. 5 Time [sec] – displacement [m] characteristic for composite CHS column

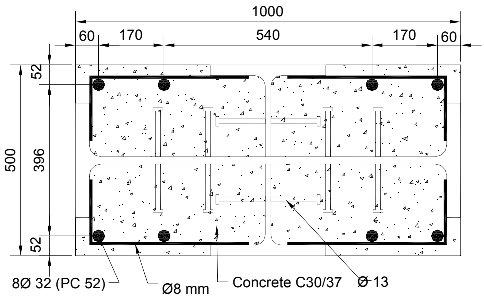

Fig. 6 Composite cruciform cross-section

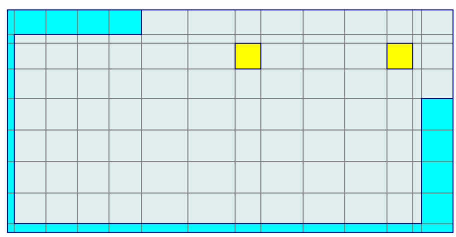

Fig. 7 Numerical model of composite cruciform cross-section

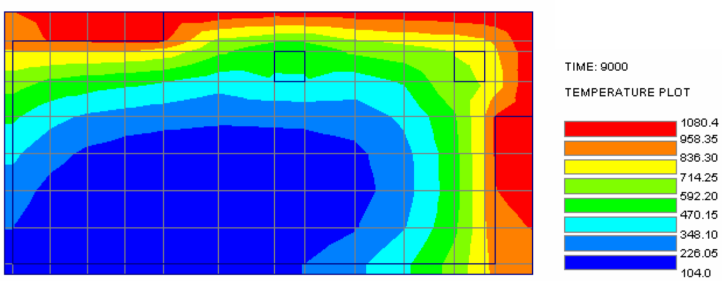

Fig. 8 Temperature distribution after 2½h of ISO fire in composite cruciform cross-section

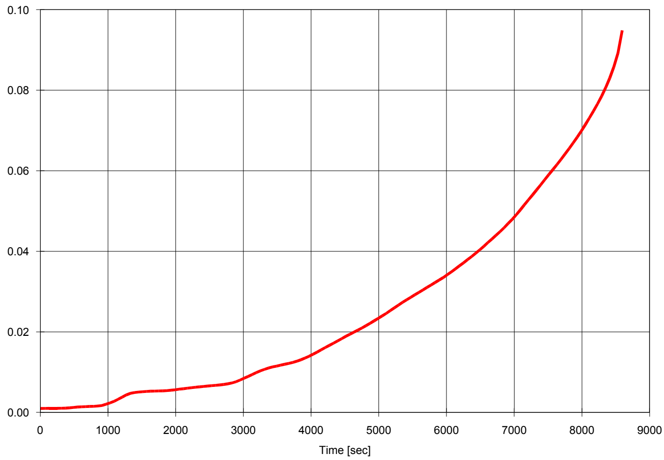

Fig. 9 Time [sec]– displacement [m] characteristic for ground floor column

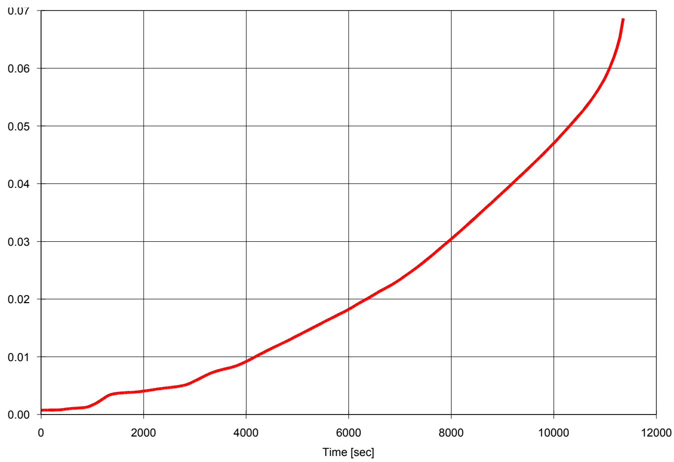

Fig. 9 Time[sec] – displacement [m] characteristic for first floor column

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NOVI SAD, NOVEMBER 22-24,2006.

Nadja Kurtovic-Folic[1]

PHYSICAL AND CULTURAL FORCES AFFECTING BUILDING CONSTRUCTION THROUGH HISTORY

Summary: *Because building construction involves considerable time and expense, and because most buildings are expected to be long-lived, frequent and sudden failure cannont be tolerated. Throughout history builders have always learned, sooner or later, from mistakes and failures. It may be therefore instructive for today to list some of the causes of failure that have resulted in either partial or total collapse. With respect to the past, building regulations or bylaws provide one source of information in standards that imply the existence of undesirable practices. Another source is the assessment that have now and then been made of the causes of a structure collapse, particularly as a result of earthquakes. Ocasionally, there are even direct accounts that report on some specific instance of jerry-building practice.*
Key words: *natural forces, human causes, maintenance, repair, reconstruction, laws, rituals, standards, jerry-building.*

UTICAJ FIZIČKIH I KULTURNIH SILA NA GRADJENJE OBJEKATA TOKOM ISTORIJE

Sažetak: *Gradjenje objekata zahteva odredjeno vreme i troškove, očekuje se da većina gradjevina bude dugotrajna. Ovi zahtevi ne dozvoljavaju tolerisanje čestih i iznenadnih graditeljskih promašaja. Tokom cele istorije graditelji su, brže ili sporije, učili na osnovu grešaka i rušenja. Smatra se da bi navodjenje nekih uzroka promašaja koji su izazvali delimično ili potpuno rušenje bilo i danas poučno. Graditeljska pravila i propisi mogu biti izvor informacija o standardima koji ukazuju na postojanje loše prakse. Drugi izvor su procene koje su ponekada zabeležene o uzrocima kolapsa gradjevine, naročito nakon zemljotresa. Povremeno, postoje i neposredni izveštaji koji opisuju pojedine slučajeve nesolidne gradnje.*
Ključne reč*: prirodne sile, ljudski uzroci, održavanje, popravke, rekonstrukcija, zakoni, rituali, standardi, nesolidna gradnja..*

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1. INTRODUCTION

There have been many failures along the history way. For in all but the rudest and most minimal examples, building is a complicated activity that must accommodate many condititions and meet many requirements. From the very beginning the primary, universal function of man's building has been to ensure his survival by creating an environment that could shelter him from the hostile elements as well as from his human and animal enemies. Whether the hostilities of nature manifested themselves in excesses of heat, cold, wind, or rain, or any combination of these and other inimical forces, man as a builder has had to work out solutions that could protect him from their baneful effects and provide him with a viable interior habitat. As his state of civilization developed, he had to create a more and more ideally controlled habitation that went far beyond the stark needs of mere survival, preservation, and the perpetuation of life.

During the building history jerry-building has been one way of the human forces affecting the failure of the building construction. That is because jerry-building may be defined as shoddy practices that result in unsafe and dangerously substandard structures, that flout existing building regulations by substituting faulty or unferior materials or by utilizing adulterated, damaged, inadequate or defective components, or that are the consequence od inept or incompetent workmanship. As no systematic coverage of this subject has ever found its way into print, there are only indirect and partial sources that reveal some idea of the nature and extent of jerry-building practicies, despite their existence throughout the world and throughout history.

2. FAILURE FROM NATURAL FORCES

The titanic forces of nature are one cause of destruction. Take, for example, the thick, smothering overburden of hot ash and mud from the A.D.79 eruption of Vesuvius that embalmed Pompeii and Herculaneum; the period floods that have caused such wide spread misery and devastation in China, India, Check Republic, even Serbia and Roumania in last two years; the earthquakes that have brought appaling ruin to many places in the world over the centuries; the forest fires, out of control over hundreds of acres, that in this century have brought recurring desolation and wholesale property damage in the timbered mountain landscapes of Meditarenian costs; and the earth slides in northern Italy that have repeatedly obliterated whole villages, created or eliminated lakes, and even changed the course of rivers.

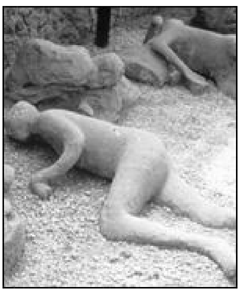

Fig.1 The human bodies in the ruins of Pompeii after the eruption of Vesuvius

One would think that localities susceptible to such catastrophes would be shunned by men, once and for all. But not so. Perhaps these cataclysmic disasters occur too infrequently in places that are otherwise highly desirable and/or productive. In any event, man seems to be a perennial optimist when it comes to building in dangeroulsy vulnerable locations. And at least when the devastation is not total and the damage is reparable, means have been devised to couteract some of the fury destructive forces or to lessen the damage they cause, if builders will but heed the lessons of previous destruction and build accordingly.

With respect to one of these destructive forces, earthquake action, the following account comments of the inertia and the carelessness of those resposible for building in areas known to be at high risk.

"In 1798, Spallanzani, an Italian traveler visiting Messina, which had been completely destroyed in 1783, wrote down as good a rule for earthquake-proof structures as I have ever read. He said that in order to meet the horizontal thrust directed at the base of a structure, it was necessary either to provide that the entire structure move as a unit under the thrust, or that it have elasticity enough to carry the thrust from its own center of gravity when the shock came. He advised against building structures more than two stories high (at that time of course the steel structures of today were not known). The brick building of that day and those of stone above two stories in height he knew would be dangerous in the severe earthquakes that visit the Calabrian Straits. He also recognized the increased danger to all structures erected upon the sandy shore. He then gave plain warning to those who should rebuild the city of Messina that certain building rules must be observed. The city of Messina has since been rebuilt and again destroyed, and it is now being rebuilt for the third time."[2]

Fig.2 The town of Messina after the earthquake in 1908

The account continues with an updating of the tendency to ignore the clear warnings of past experience:

„ After the great earthquake (of San Francisco) in 1906, the city building code was revised to insure an increased wall strenght, through which to meet horizontal thrusts.There was no method then known to accomplish this except in terms of wind-load specification was raised in 1907 from 15 lb. to 30 lb. per sq. ft. In 1925,however, it is rather curious to note that without anybody being able to tell just when, why orhow, the wind-load somehow found its level at 15 lb. again. In addition to that, the allowable limit

[2] A.L.Day (Director, Geophysical Laboratory, Washington, D.C.), *Earthquakes and Their Effects on Buildings* (1926), p.75

of maximum load for structural steel had also been increased in the meantime 12 ½ per cent, so that San Francisco, on that day of our visit last summer, was less well prepared to meet another shok of severity such as it had once been through, than it was in 1906...[3]

Fig.3 The houses in San Francisco after the eartquake in 1906

3. FAILURE FROM HUMAN CAUSES

A much more constant and common category of failure in building construction consists of man-made predicaments, the result of misjudging the behavior of materials or miscalculating the degree of strain and stress that a building may undergo in the normal course of events. These problem include inadequate foundations, insufficient bracing against wind pressure, and unsatisfactory framework for meeting the structural requirements for strength and stability. Failure may result from vibration, caused by the ringing of bells in a lofty tower[4] or by the rhythmic tread of a military detachment (which contributed, along with the blasts of the priests' trumpets and the great shouting of the Israelites, to the destruction of the walls of Jericho)[5].

[3] Idem, ibidem.
[4] A.D.R. Caroe, *Old Churches and Modern Craftsmanship*, London, Oxford University Press, (1952), pp. 163-173.
[5] Jos. 6:3-20.

Fig.4 The painting showing the fall of the Jericho walls caused by the blasts of the priests' trumpets

Structures may collapse from the overloadingof walls and foundations due to the addition of stories they were not originally designed to receive. „After the great earthquake of 1783, it was duly recognized that the buildings must be low, of one story if possible...but as the years went on and the population incresed, stories were added one after another even to five stories while the retaining walls were not strengthened in proportion...“[6] Quite adequately designed structures may collapse because of inferior material, as in the case of the original choir of Beauvais Cathedral,whose failure was at least partly due to the poor quality of the stone that could not take the compressive loads of the lofty superstructures. „part of this edifice fell down less than a cenutry after the completion of the choir; and yet it was designed in such a way as to enable it to stand for centuries. This dissaster, which has completely altered its character, was due to the indifferent execution, the lack of rigid support, or their too slight resistance and ecpecially to the nature of the materials, which were neither large nor solid enough...“[7] With respect to more universal conditions, too casual and infrequent maintenance may result in failure where weathering, rusting, or rotting has been allowed to proceed unchecked,[8] where serious cracks or fissures in the fabric of the building have been ignored, and where periodic cycles of expansion and contraction have weakened the fabric though metallic fatigue or the disintegration of proper adhesion between the components of the masonry.[9] Sometimes, moreover, drastic failures occur while the building is under construction, as when formwork is removed prematurely, when the mortar or concrete is still green Finally, failure may follow from the culpable negligence, outright dishonesty, or fraudulent practices of the builder.

4. MAINTENANCE, REPAIR AND RECONSTRUCTION

In building construction of all sorts, a very considerable effort over the years goes into repaira and reconstructions. In buildings made of organic materials, such as thatched roofs of grass or walls of plaited palm fronds, the materials themselves dry out, become britlle, and are no longer effective protection against rain or wind. Clay or mud plaster extends the life span of these materials, but such coatings spall off in time and need to be patched. Wood-attack caused serious damage to the timberwork of many medieval roofs, including the unique octagon at Ely Cathedral (the deathwatch beetle in this case). In Japan the most ancient wooden pagodas have been periodically taken down, when the age and weathering of the old materials have threatened their survival, and then rebuilt in new materials as exact replicas in every minute detail.[10] In the case of lofty structures, high

[6] “The Messina Earthquake”, *Scottish Geographical Magazine* 26 (1910), pp.95-96

[7] E.Viollet-le-Duc, “Costruction” in *Dictionnaire raisonné de l'architecture française*

[8] A.R. Powys*, Repair of Ancient Buildings*, (1929), p.72

[9] Makyš, Oto, *Rekonštrukcie budov, Technológie,* Jaga group Vydavateľstvo, Bratislava (2000)

[10] N.Kurtovic-Folic,“Novo tumačenje doktrine zaštite graditeljskog nasledja”, *Prostorno planiranje, regionalni razvoj i zaštita životne sredine 3* (1997), (ur. N.Spasić), IAUS, Beograd, pp.153-172.

winds sometimes topple exposed features and bolts of lightning cause major damage that requires extensive repairs. The scouring of rivers, causing shifts in the channel, sometimes endangeres the integrity of pier foundations and abutments, necessitating deeper and more protected footings. In wintertime ice builds up and sometimes breaks man-made structures, which then need to be repaired. Both fire and earthquake may cause very extensive damage, imposing the need for comprehensive repair, if not total rebuilding. Movements or shifts in the subsoil water table may compromise the stability of foundations and require underpinning to forestall subsidence. Particularly in jungle areas, the insinuating roots of vines and even great trees constantly disintegrate stone buildings. To a lesser extent, weeds lodging in crevices in the walls and moss forming on the roofs of buildings, must be periodically removed and the affected areas repaired. In our day atmospheric pollution is not only fouling the appearance of but actually disintegrating stonework. Shingled roofs need replacing at least every twenty years; clapboarded walls need repainting every few years, with cracked or warped walls or rotted clapboards replaced as required.

Fig.5 The famous octogon of Ely Cathedral was seriously damaged by the deathwatch beetle

Very extensive repairs and rebuildings come about not only in consequence of the forces of nature but by man's own destructiveness, as in the seemingly indeterminable recurerrence of warfare. An enormous amount of money has been expended for reconstruction necessitated by the devastation of World Wars I and II, not to mantioned local wars and revolutions, particularly in Europe. Even cosmetic repairs to these buildings usually involve the use of equipment, such as scafolding of some sort, and are both time-consuming and costly.

Over the years, as social needs and institutions have changed, repairs to some buildings have involved shifts in the building's use or occupancy. A well-known Roman example is the central concourse of Diocletian's great Imperial Baths, which become a Christian church, *Santa Maria degli Angeli*, in the fifteenth century. Again, the thick-walled donjon was built as the place of impregnable refuge in the castles of the early Middle Ages; later, its very indestructibility brought about its use as a prison when times became more settled and life on a medieval barony more civilized. Were it not for such changes in use, most of the monumental buildings of former areas would long since have perished through neglect and ruin or through the removal of their materials to build entirely different structures.

Fig.6 Interior of Diocletian's Imperial Baths transformed in the church Santa Maria degli Angeli by Michelangelo in XVI Century

5. CONCLUDING REMARCS – THE PERSISTENCE OF TRADITION IN BUILDING

Tradition in native practices and customs has always been a dominant and very powerful force among peoples and cultures in former times. Innumerable examples come to mind, embracing language and its local dialects, dress and ornament, food and its preparation, social relationships, in fact, all those charachteristics of belief and conduct, of attitude and habit, that particularize each human culture and distinguish one tribe or people from another. Building has been no exception. It too has quite universally shown its own adherence to time-honored custom, and even more strongly so, perhaps, because of its constant and highly visible physical presence.

There have been, of course, significant breakthroughs from time to time; but most have been gradual and incremental rather than sudden and spectacular. Colored glass, for example, was produced and applied in tiny pieces to small, infrequent perforations in thick walls many centuries before the extensive, resplendent achievements of thirteenth-century stained-glass windows were created. Again, the cantilever principle was utilized in native Himalayan bridges in stepped, overlapping projections of timber beems centuries before steel made possible the astonishing triumphs of the Firth of Forth and the Quebec bridges. But at the very least, where changes has indeed occurred, the record indicates that man has often taken a long time to accept innovation, to come to terms with and adopt practices, however advantageous, that defy conservative tradition and local custom. Until quite recently, even in the Western world, the building industry has reputedly been one of the most conservative businesses, least subject to progressive practices, least disposed to abandon traditional methods. In the past this persistent conservatism generally accounted for continuity in the life style of a people, even in the face of hostile invasion, constant warfare, or grinding exploitation. It gave to the inhabitants, perhaps quite unconsciously, and indispensable sense of community, participation, and shared identity.

It is important, then, to try to retreive credible information of the construction methods and building techniques of the past. For the traditional native crafts and processes represented stable usages and time-honored practices. It is clear that they indicated an accepted and viable way od adjusting to the climate and of meeting the requirements of living in a particular community. Wherever this established practices had time to develop independenly, they were perfected in terms of the materials employed, the tools developed to obtain, prepare, and fashion them, and the procedures followed in their assemblage or erection. The various steps and operations adopted in customary building operations were therefore recognized by the whole community. That is, each culture had one "style" of building, one way of creating a habitation or a temple, a storehouse or a means of defense. With tools, materials, and physical needs in common, the members of every native community in former times produced common results in the sphere of building activities. These shared building products identified and defined all such communities and their life style, both physically and visually. Certainly among what in modern times have been called "underdeveloped peoples", these indexes of sustained differentiation are clearly evident in the building forms they followed and the practices they perpetuated.

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Igor Boyko[2]

NUMERICAL ANALYSIS OF THE STRESS STRAIN STATE EVOLUTION OF LANDSLIDE-PRONE TERRITORY

Summary: *The modeling concept is presented. The evolution of the stress state of earth mass at different stages of landslide process development and its changes under the influence of major factors is investigated. The main factor causing landsliding was determinate using numerical modeling.*
Key words: *slope, landslide-prone territory, landslide, factors, modeling, stress-strain state, nonlinear deformation, slip surface.*

NUMERIČKA ANALIZA NAPONSKO-DEFORMACIJSKOG STANJA RAZVOJA POTENCIJALNOG KLIZIŠTA

Rezime: *Prikazan je koncept modeliranja. Istražen je razvoj naponskog stanja zemljane mase u različitim fazama razvoja klizišta i promene usled uticaja glavnih faktora. Glavni uzrok klizišta određen je numeričkim modeliranjem.*
Ključne reči: *kosina, potencijalna klizišta, klizište, faktori, modeliranje, naponsko-deformacijsko stanje,nelinearna deformacija, klizna površina.*

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1. INTRODUCTION

For today increase of the investments into construction in complicated ground resources conditions is more often lead to use landslide-prone territories. The main terms for such territories for the construction period and during exploitation are maintenance of slope stability and durability of superstructures. Thus it is necessary to take into account possible changes of soil parameters under the action of different factors (seasonal fluctuations, additional loading, etc.).

Such detailed analysis taking into account action of all factors together or separately is possible to execute by using numerical modeling. It enables to analyze evolution of the stress-strain state of an earth mass on landslide-prone territories that is an important problem.

2. NUMERICAL MODELING CONCEPT

It is the principal task to provide the earth mass stability during the construction works as well as the superstructures strengthening during the operations on the landslide-prone territories. Thus, it is necessary to take into consideration the probable changes of soil parameters under the influence of different factors (seasonal influences, technology of the underground works, extra load, etc.).

To prevent the development of the landslide processes and to preserve the lands for the future generations, it is necessary to carry out the detailed analysis of the stress-strain state of the earth mass on the landslide-prone territories at all stages of the slope loading.

The updated model by Prof. I.P. Boyko within the framework of the dilatancy theory by Nikolaievskij is adopted for the simulation of soil elastoplastic behavior. For a soil satisfying the plasticity condition the total strain increment $\hat{\varepsilon}$ is additively composed of elastic $\hat{\varepsilon}^{e}$ and plastic $\hat{\varepsilon}^{p}$ components:

$$\hat{\varepsilon} = \hat{\varepsilon}^{e} + \hat{\varepsilon}^{p}. \tag{1}$$

The modified Mises-Schleicher-Botkin and Coulomb-Mohr condition is used as plasticity criterion:

$$f = \left| \begin{array}{ll} T+\sigma_m tg\psi-\tau_s=0 & for\, \sigma_m \geq p_o; \\ T+p_o tg\psi-\tau_s=0 & for\, \sigma_m \,\pi\, p_o, \end{array} \right. \tag{2}$$

where $T= (\frac{1}{2}s_{ij}s^{ij})^{1/2}$, $s_{ij}=\sigma_{ij}-\sigma_m g_{ij}$
is the tangential stress intensity, ψ is the friction angle on an octahedral plane, and $H=\tau_s /tg\, \psi$ is the ultimate resistance to volumetric extension. In the principal stress space condition corresponds to a combined limiting surface which incorporates two figures – a cone and a cylinder.

The plastic stresses are corrected on the basis of the non-associated flow rule:

$$d\hat{\varepsilon}^{p} = d\lambda \frac{\partial F}{\partial \hat{\sigma}}, F \neq f, \tag{3}$$

where F is a plastic potential function and $d\lambda$ is a small non-negative scalar multiplier determining the absolute value of $d\hat{\varepsilon}^{p}$.

The model represented above is realized in automatic system of scientific researches (ASSR) «VESNA».

3. GEOLOGICAL CONDITIONS

The landslide-prone territory is situated on the left bank of river Lybid, where the difference of heights is more, than 30 metres. The geological cross-section is represented by filled-up soil from above depth to 4 m and has distribution at the bottom of the slope. There are sand and sandy loam under this layer. Clay occurs at depth about 30 m.

The finite element model is shown in Figure 1.

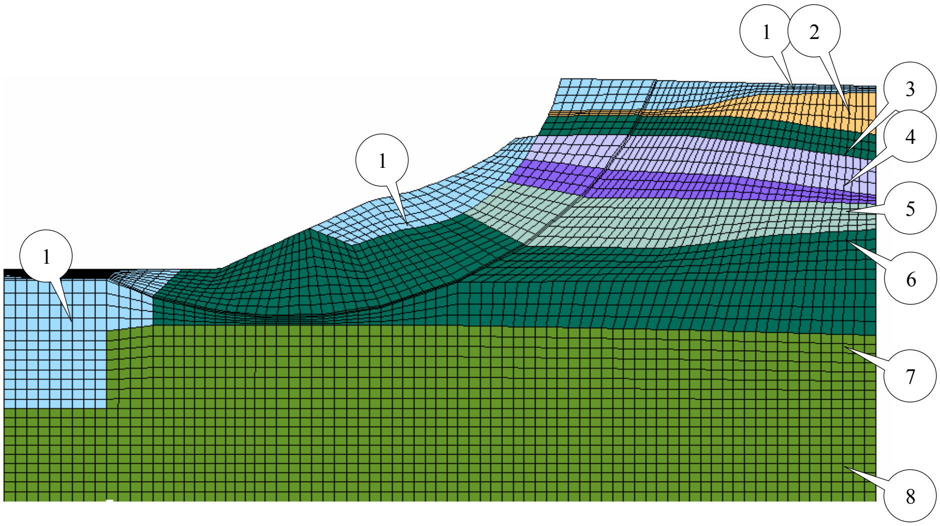

Figure 1. Finite element model

The effective parameters for slope stability are given in Table 1, where the following symbols are used: E – deformation modulus [MPa], ν – Poisson ratio, φ – friction angle [º], c – cohesion [kPa], γ – unit weight [kN/m^3].

№	Material	E	ν	γ	c	φ
1	Filled-up soil	5	0,3	17,0	5	20
2	Sand, medium-grained	30	0,3	19,8	0,5	36
3	Sand, medium density	20	0,3	19,4	1	32
4	Sandy loam, fluid state	10	0,32	18,9	6	16
5	Sandy loam, plastic state	12	0,31	19,5	5	18
6	Sandy loam, plastic state	15	0,31	19,3	7	19
7	Loam, solid state	20	0,35	19,1	28	22
8	Clay, solid state	24	0,3	18,9	40	18

Table 1. Soil parameters

4. RESULTS

The calculated stress strain state evolution is shown in Figures 2, 3 and 4. For each case we fix zones of the plastic deformation distributions which we can observe in the characteristic areas.

Figure 2 demonstrates the initial state of the slope. Figure 3 features the changes of the plastic deformation zones after modification the soil parameters in the potential slip surface. Figure 4 the displacement vectors cumulated during the excavation at the bottom of the slope are depicted.

In such a way was determinate the main factor causing landsliding.

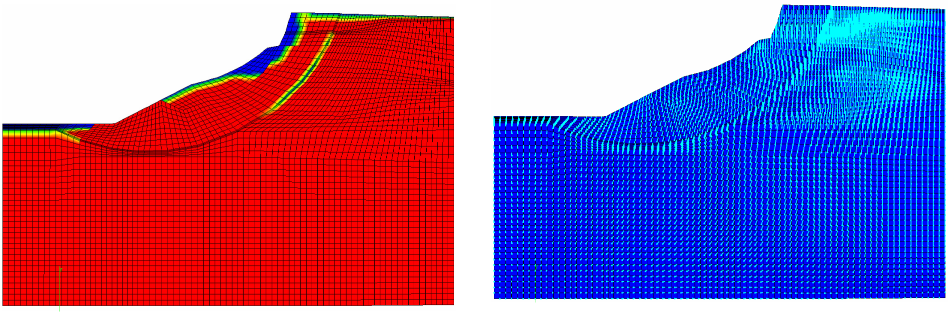

Figure 2. Distribution of the plastic deformation zones and displacemen vectors

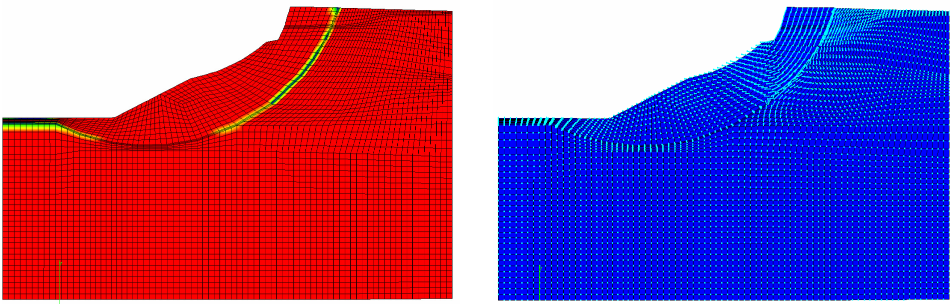

Figure 3. Distribution of the plastic deformation zones and displacemen vectors

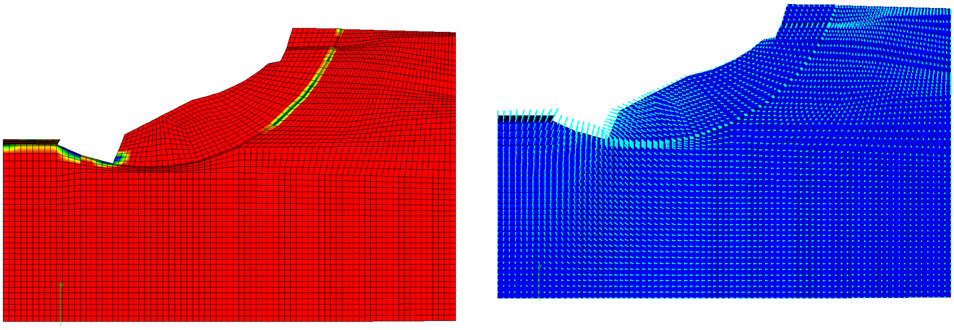

Figure 3. Distribution of the plastic deformation zones and displacemen vectors

5. CONCLUSIONS

1. The use of numerical modeling gives us an opportunity to observe for the stress strain state evolution under the action of different factors.
2. The evolution of the stress state of earth mass at different stages of landslide process development and its changes under the influence of major factors are investigated on the basis of «VESNA» programmed complex. This programmed complex is orientated to solve nonlinear 3D geotechnical tasks.
3. Fixation of the development of plastic deformation zones determinates the slip surface location.
4. Received results provide a chance for the optimal solution for the landslide territory stabilization.

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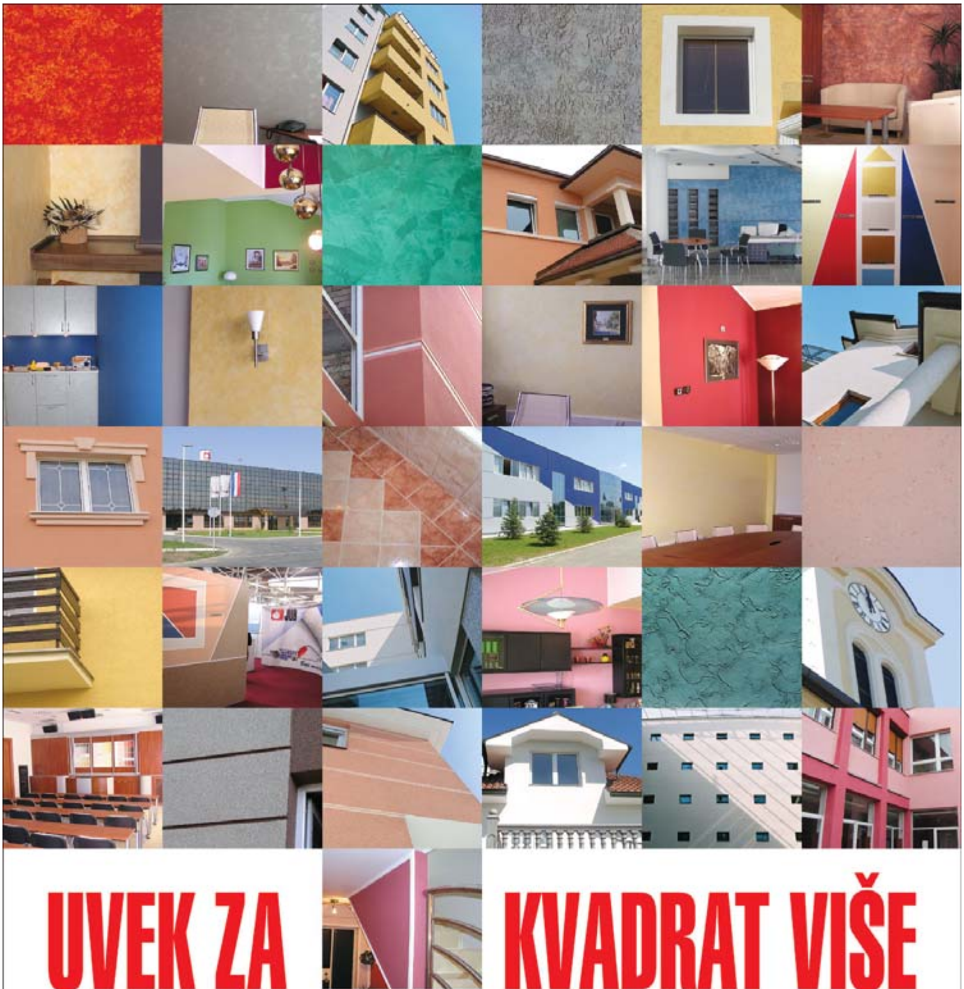

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Firma MIKOM inženjering osnovana je 1999.godine, u sklopu kompanije MIKOM, sa sedištem u Novom Sadu - SCG. Osnovna delatnost MIKOM inženjeringa je projektovanje, izvođenje i montaža svih vrsta čeličnih konstrukcija, izrada čeličnih polufabrikata, aluminijumske bravarije i strukturalnih fasada, kao i izgradnja objekata po sistemu "ključ u ruke". Pri izgradnji se koristi sopstvena tehnologija proizvodnje i montaže čeličnih elemenata. Trenutno je zaposleno 50 radnika, među kojima je velik broj visokostručnih kadrova

Generalni smo zastupnici poznatog italijanskog proizvođača termoizolacionih panela - RWP Venecija (web site: www.rwpanel.com) za područje Srbije i Crne Gore i BiH.

Proizvodnja čeličnih polufabrikata

MIKOM inženjering pored proizvodnje i montaže čeličnih konstrukcija vrši: uslužno sečenje, savijanje, bušenje, probijanje, zavarivanje i druge srodne operacije, kao i proizvodnju poluproizvoda iz programa obojene metalurgije.

Hladno oblikovani profili se izrađuju hladnim oblikovanjem, polazeći od osnovnog materijala-hladnovaljane i toplovaljane trake svih kvaliteta, pogodnih za preoblikovanje hladnim postupkom.

Sečenje traka pogodnih za hladno oblikovanje se vrši hidrauličnim makazama na dužini od 6000 mm i debljini 13 mm.

Pojedinačna izrada profila velike duzine - do 6m izvodi se na specijalnim mašinama za profilno savijanje, koje su poznate pod nazivom prese za profilno savijanje ili apkant prese. Na ovim mašinama se izrađuju profili L, U, Z, C.... Pri oblikovanju trake u profil, granice plastičnosti i čvrstoće se povećavaju, dok se izduzenje smanjuje.

Rezanje limova debljine od 13mm do 200mm, pravolinijskom i krivolinijskom putanjom rezanja, izvodimo na fotomatu sa duzinom radnog zahvata 3m.

Savijanje cevi izvodimo na profilisanom trovaljku i to:
cevi kružnog poprečnog preseka fi88,9mm i kvadratnog poprečnog preseka do 80x80mm.

Probijanje otvora kružnog i ovalnog oblika izvodimo na mašinama za probijanje i prosecanje.
Bušenje otvora i rupa izvodimo na radijalnoj bušilici sa prečnicima alata do 55mm.

Vršimo zavarivanje elemenata konvecionalnim postupcima zavarivanja, kao što su: REL, MAG, MIG, TIG, kao i reparaturna zavarivanja svih vrsta čelika: L, SL i obojenih metala.

Rezanje pripremaka pod uglom od 90 stepeni iz punog profila,kao i debelozide cevi većeg poprečnog preseka izvodimo na kružnoj testeri sa radnim zahvatom max 500mm.
Rezanja cevi na meru sa mogućnošću ukrajanja pod uglom od 30,45,60 i 90 stepeni.

Proizvodnja aluminijumske bravarije i strukturalnih fasada

Raspolažemo sa najsavremenijom opremom za obradu aluminijumske bravarije i aluminijumskih fasada (mašine firme Elumatec - Nemačka).
Koristimo serije profila firme FEAL, Široki Brijeg:
- serije hladnih profila
- serije profila sa prekinutim termičkim mostom
- profili za fasade

Površinska zaštita profila je eloksiranjem i plastificiranjem, prema RAL karti.

Takođe raspolažemo sa najsavremenijim softverom za proračun repro materijala, izradu krojnih lista i lista za spajanje i optimizaciju rezanja.

MikoM

Stambeno-poslovni objekat Dalton, Novi Sad

BINS - sportsko rekreativni centar, Novi Sad

REFERENCE

Poslovno - proizvodni objekat, Novi Sad

TERMIKA a.d. Zrenjanin

a.d. za proizvodnju termoizolacionih materijala

A) TERMOIZOLACIONI MALTERI

VRSTA MALTERA	ČVRSTOĆA MALTERA [MPa]	ZAPREMINSKA MASA SUVOG MALTERA [kg/m^3]	KOEFICIJENT TEMP. ŠIRENJA α[mm/(m*100K)]	KOEFICIJENT TOPLOTNE PROVODLJIVOSTI λ[W/mK]
SUPERMAL	0,688	395	1,26	0,091
PERMAL	1,8	352	0,76	0,081
TERMIKAfil	termoizolacioni malter za nasipanje u sendvič zidove			0,033

B) TERMOIZOLACIONI BETONI

VRSTA BETONA	ČVRSTOĆA BETONA [MPa]	ZAPREMINSKA MASA SUVOG BETONA [kg/m^3]	KOEFICIJENT TEMP. ŠIRENJA α[mm/(m*100K)]	KOEFICIJENT TOPLOTNE PROVODLJIVOSTI λ[W/mK]
SUPERBET 1	0,5	258	0,9	0,072
SUPERBET 2	1,2	438	1,02	0,084
SUPERBET 3	2,72	578	1,1	0,108

C) OSTALI PROIZVODI

PERLIT P 1	granulati za samospravljanje maltera i betona
PERLIT P 2	
PERLIT P 10	
KRIOGENI PERLIT	granulat za kriogene izolacije
AGROPERLIT	granulat koji ima široku primenu u poljoprivredi
TERMIkol	grupa lepkova za keramiku i stiropor
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