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STEINORT CASTLE – HISTORY, CURRENT STATE, INTERVENTION

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Abstract: *The Count von Lehndorff family, the last German owners of Steinort, is one of the oldest and most important noble families in East Prussia. Steinort Castle (in polish language Sztynort), today in the Polish part of East Prussia, is one of the most important mansions there. In addition, Heinrich von Lehndorff took part in the attempted coup against Hitler on July 20, 1944 and was murdered for it. The rescue and revival of this castle is therefore of exceptional interest. The baroque wooden beam ceilings are characterized by their unusually large span and unique polychrome painting. This paper reports on the minimally invasive replacement of the ceilings (in order to achieve higher load-bearing capacity, the original beam was glued with an additive).*

Key words: *heritage preservation, timber structures, ceiling, strengthening, structural bonding*

1. Introduction

The northeastern Masuria, a land of forests and lakes, is location of castle Steinort, (in Polish Sztynort), a manor house that belonged to the von Lehndorff family for nearly 500 years. Nowadays, it is a cultural monument of German heritage. The core of building (in German: Kernbau) was constructed between 1689 and 1693. In the 19th century there were extensive structural changes expanded the Kernbau, with baroque characteristics, with two wing buildings (1829) and three towers (1860-1880), which were built in the spirit of the time - late classicism.

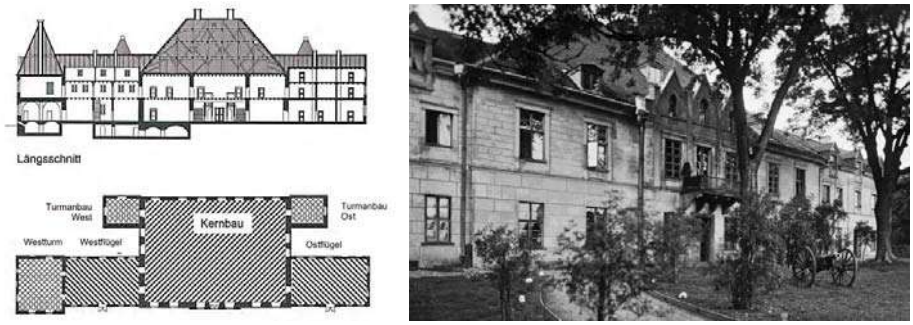


Fig. 1. Steinort Castle: Floor plan and section [1], photo in the 1930ies [2]

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At the end of the 1930s, the castle underwent a thorough renovation after it had been neglected and damp since the First World War. It was in immaculate condition until the Red Army arrived in January 1945. Count Heinrich von Lehndorff lived in one wing of the castle with his family; the field quarters of Foreign Minister Ribbentrop were set up in the other half in 1941.

The name von Lehndorff is best known for Count Heinrich von Lehndorff, the last owner at Steinort Castle, who was involved in the failed Stauffenberg assassination attempt on Adolf Hitler on July 20, 1944 and was executed shortly afterwards in Berlin. After 1990, the former East Prussian estate fell into disrepair at a rapid pace (Fig.2).



Fig. 2. Castle Steinort/Sztynort in 2000

2. Historical value

In addition to the castle complex as a whole, polychrome painted ceiling beams with a span of 10,00 m have been preserved from the time of its construction, reflecting traditional Masurian craftsmanship (Fig. 3). The still preserved, original ceiling construction covering an area of 1400 m² is unique in its size, originality and completeness.



Fig. 3. Polychrome painted ceiling, conditions 2004 [3]

However, for public use as a museum or exhibition space, the existing structure must be reinforced for an imposed load of 5 kN/m². The renovation work is mainly paid for from the German federal budget. Project manager is Prof. W. Jäger, Radebeul, Germany.

3. Methodology and intervention

To meet the structural requirements for public use and the conditions for the preservation of historical monuments (Rule of minimal intervention [4]), the idea for strengthening the ceiling beams is based on transforming the rectangular beam (Fig. 4a) into a T-beam. The former filler made of loose organic material is replaced by a glue-laminated timber over a rectangular cross-section, so that a T-cross-section can be created (Fig. 4b). The analogy to reinforced concrete or steel construction is clearly visible here. The height adjustment plank was glued to the bottom and top with a melamine-urea resin adhesive. This created a T-section with a much larger load-bearing capacity. The necessary tests have been carried out of Prof. W. Jäger´s team at the TU Dresden.

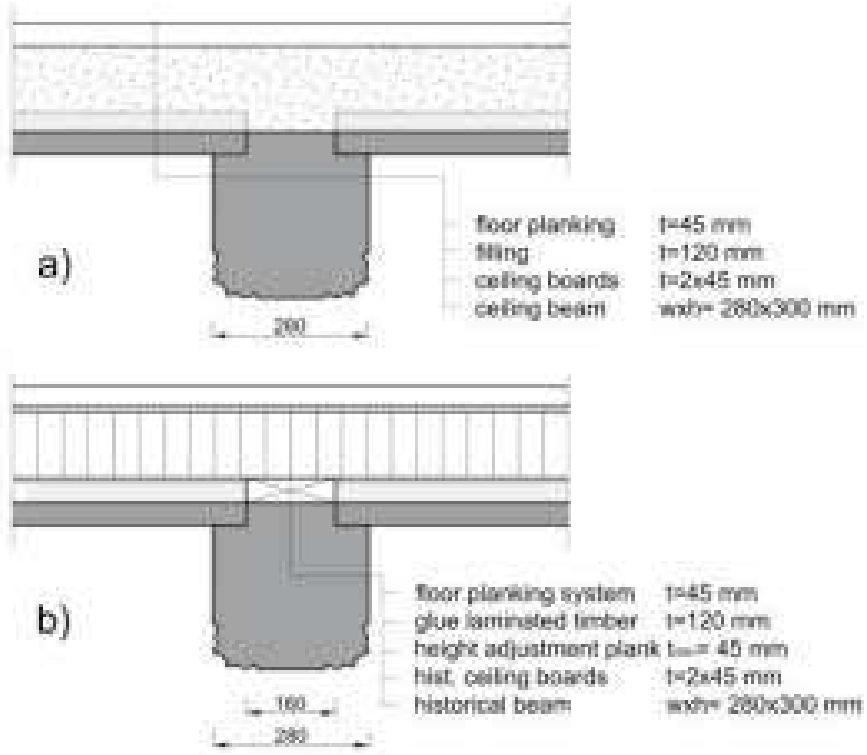


Fig. 4. Ceiling beams a) historical b) reinforced [5]

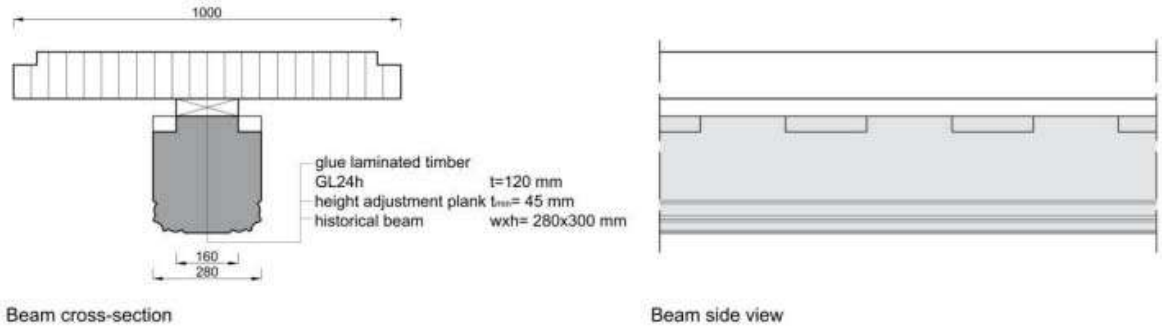


Fig. 5. Cross-section of a T-beam [5]

Finally, the glue-laminated timber panels must be screwed together. Fig. 6 shows a section of the ceiling. The historical boards could be pushed in from below.

4. Discussion

The presented method for material-appropriate strengthening enables the wide-span wooden beam ceilings to be used by the public in a way that traditional solutions cannot achieve. This method is suitable for historic buildings taken into consideration international principles of monument conservation [4]. Worse alternatives to this method are timber concrete composite ceiling and even a reinforced concrete ceiling. In addition, the in situ strength sorting of the historical wooden beams as described in [7] could be useful. The effort and expense of strengthening using structural bonding are justified by the great historical and architectural significance.

The renovation of Steinort Castle is mainly financed from the German Federal Budget. Project manager is Prof. W. Jäger. The renovation work on the castle began in 2014 and will continue over the next few years.

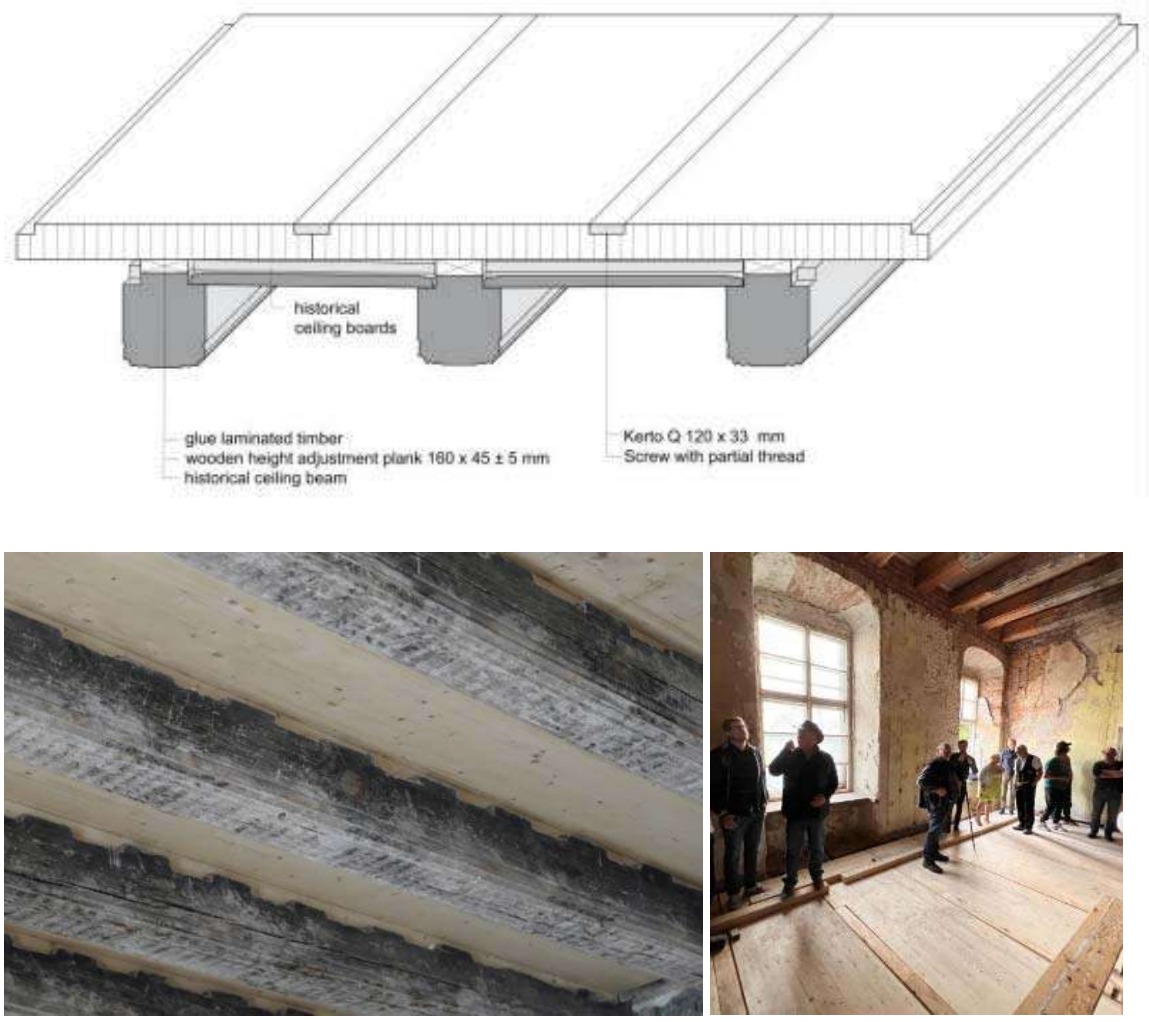


Fig. 6. Section of ceiling [5],[6]

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CONSTRUCTIVE MEASURES TO PRESERVE THE BEARING CAPACITY OF STEEL VERTICAL TANKS IN SERVICE FOR A LONG TIME

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***Abstract**: The tanks are vessels designed for reception, storage, consumption, technological prefabrication and transportation of various liquid products. The most significant part of this large group of engineering facilities, with a wide application in almost all branches of our national economy, are steel vertical tanks. They intensively entered our country in the sixties of the last century, which is connected with the intensive development of the economy. Following the chronological course and development of reservoir construction in our country, three main periods can be defined - Origin, Development and Technical inspection, repair and reconstruction. In Bulgaria, we have already entered a period when the normal operating life of the majority of AST is expiring or even has already expired. Therefore, a period of intensive and responsible technical inspection, repair and reconstruction of the steel tanks in our country is emerging. These activities are aimed at preserving their bearing capacity and even increasing their safety during exploitation.*

***Keywords**: steel tanks, development, bearing capacity, safety, bottom, shell, roof*

1. Introduction

The tanks are vessels designed for reception, storage, consumption, technological prefabrication and transportation of various liquid products, regardless of their shape, material, location and construction. They are known from the deepest antiquity. As an example, clay amphorae for storing wine can still be found among the remains of ancient merchant ships.

The operating conditions of the tanks are very different. This leads to considerable diversity among them. They can be:

a) according to their position relative to the earth's surface – raised, on the ground, semi-buried and buried;

b) according to their shape – cylindrical vertical tanks, see Fig. 1, cylindrical horizontal tanks, drop-shaped, spherical, with a conical bottom, water towers, trench;

c) according to the used materials – clay, wood, metal (copper, aluminium, steel), plastic.

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The largest part of this big group of engineering facilities, with wide application in almost all branches of our national economy, are aboveground steel vertical tanks (AST), which are spatial sheet structures with a cylindrical shape, see Fig. 1.



Fig. 1. Aboveground steel vertical tanks

They have the following advantages over other types of tanks:
- from an economic point of view, they are optimal for storing large quantities of liquids at low pressure;
- the used there constructive solutions have been repeatedly checked in practice;
- the wide variety of types of roofs, shell and bottom, structural elements, of joints, allows the selection the most economically optimal solution;
- they are convenient for prefabrication, mounting and maintenance in the process of exploitation;
- they are suitable for repair and/or structural alterations.

2. Development of tank building in Bulgaria

If 1878 is taken as the starting year for the construction of steel vertical cylindrical tanks, when the first such tanks were designed and built in Russia [4], these facilities will meet their 145th anniversary these days. Five years later, in 1883, *Shukhov* developed and published a method for the calculation of AST, showing their rational constructive form and dimensions [6, 10], ensuring quick and cheap fabrication and assembly with minimal metal consumption.

Steel tanks entered intensively in our country in the sixties of the last century, and their introduction into operation was provoked by the rapid development of the economy. A trend of continuous growth of the total storage tank capacity ΣV in the country has been imposed and developed. This trend leads to two interrelated directions: overall increased tank construction per year and an increase in single volume V_1.

The first direction is related to an increase in oil refining, which in turn requires a significant expansion of the storage capacities of the petrol bases (PB). Another factor here is the rapid development of thermal energy related to the use of liquid fuels (usually fuel oil) for thermal power plants (TPP).

The increase in the volumes of fuels that had to be stored in the PB and fuel oil holdings of the TPP inevitably led to an increase in the single capacity of the tanks. Thus, gradually the V_1 = 1,000 m^3 and 2,000 m^3 tanks, which were typical at the beginning, gave way to V_1 = 5,000 m^3 and 10,000 m^3 tanks. The increase in single volumes V_1 means larger dimensions (diameter D and height H), and therefore – increased loads, respectively bigger thicknesses. The trivial solution of simply increasing the thicknesses leads to an

incommensurable increase in the problems associated with the manufacture and assembly of sheet structures from thick sheet metal. It was clear that a complex approach had to be applied to the issue. Indeed, the first calculations of tanks with large capacities have shown [6] that the known methods for designing AST with $V_1 \leq 2{,}000$ m^3 are not fully applicable for large capacities, and in a few cases completely new and more accurate methods are required. On the other hand, to withstand the high tensile stresses in the lower courses, the traditional BSt3 steel had to be replaced with a qualitatively new type, having increased strength. This is how the so-called bi-steel shells of the tanks [4], in which the lower courses are made of low-alloy steel with increased strength, respectively with decreased thicknesses. Bi-steel shells have been well developed in theory and applied and continue to be applied many times in practice.

Several problems in the design and construction of the tanks on a national level have been successfully solved through the implementation of Standard Working Documentation for AST [11]. This typical series has currently raised the construction of steel tanks in our country to a higher scientific and technical level, with significant industrialization of manufacturing and mounting. Standard tanks were designed with various volumes, with limited numbers of heights and diameters, which could be ordered from a catalogue. In practice, the roll method of mounting was implemented, see Fig. 2. The shell, bottom and roof shields were factory welded. They were transported as separate parts to the site, where they were joined by a minimum number of assembly welds. The execution of most welds in factory conditions has sharply increased their quality, reduced labour intensity and accelerated the mounting of tanks.



Fig. 2. Mounting of aboveground steel tanks by roll method

Currently, the Standard Working Documentation for AST [11] in question has been cancelled. The design instructions [5 and 9] were introduced. Until recently, they were leading documents in the field of steel tank design for Bulgaria, but they are also no longer relevant. Now each tank has to be calculated individually, according to the real operating conditions. In connection with the harmonization of European and Bulgarian regulatory documents, the leading European standards for tank design have been introduced into our practice. They are imperfect, but are a step forward as they reflect the latest developments

in the American and Western European schools of steel vertical cylindrical tank design. Following the chronological course and development of reservoir construction in our country, three main periods can be defined [13], namely:

(I) Origin – tanks with relatively small volume ($V_1 \leq 2{,}000$ m^3), according to foreign projects and materials. It covers the period 1900 ÷ 1950. A typical representative: a riveted steel tank with German design and material delivery, with a volume of $V_1 = 1{,}000$ m^3, built at the Port of Burgas in 1917. The joining of the shell sheets by rivets is shown in Fig. 3.

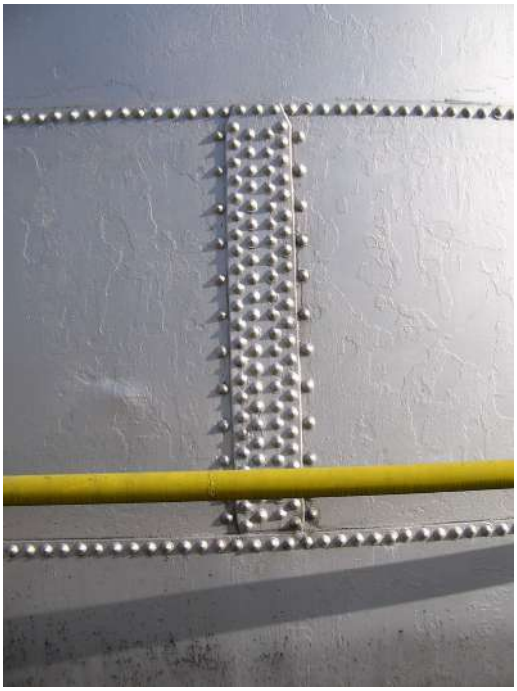


Fig. 3. Joining of shell's sheets with rivets

(**II) Development** – in this period, significant individual volumes ($V_1 = 5{,}000 \div 50{,}000$ m^3) were achieved on our typical and individual projects: on KIPP "Hranproekt" (for light petroleum products with $V_1 \leq 5{,}000$ m^3) from 1967, of NIPPIES "Energoproject" (for fuel oil with $V_1 \leq 10{,}000$ m^3) from 1970 and the Standard Documentation [11]. The projects of the tanks during this period marked the refinement of theoretical methods, optimization and automation. The construction of the reservoirs was largely industrialized through the introduction of the stands for rolls at the NHC - Burgas and later - at the ZLMK - Septemvri. It refers to the period after 1950, as it can conditionally be limited to 1980. Typical representatives are the steel tanks at NHC - Burgas, NHC - Pleven (now "Plama"), TPP "Sofia", TPP "Tr. Kostov" (now "Iztok"), TPP "Plovdiv", OC "Zemlyane", OC "Lyulin" and various oil bases in the country (St. Zagora, Burgas, Sliven, Iliantsi, etc.) with volumes up to 10,000 m^3 – mounted by roll method and from 15,000 m^3 to 50,000 m^3, see fig. 4, built using sheet by sheet method.

(III) Technical inspection, repairs and reconstructions – the maximum values of the two trends in tank building in our country (MAX V_Σ and MAX V_1) were reached in the period 1970 - 1975. On the other hand, according to the working version of the new edition of the standard EN1990, Annex A.4, the service life of a steel tank is taken as follows:

- for facilities of national importance – $t_{life} = 50$ years;
- for tanks that are part of industrial production – $t_{life} = 30$ years.



Fig. 4. Aboveground steel tank with a single deck floating roof and $V = 50{,}000$ m^3 capacity

Clearly, in Bulgaria, we have already entered a period in which the normal operational life of a significant part of the AST expires or has already expired. Therefore, a period of intensive and responsible technical inspection, repair, see Fig. 5, and reconstruction of the steel tanks in our country, which have been in operation for a long time. These activities aim to preserve their bearing capacity and even increase their safety.



Fig. 5. Repairing works on the shell of aboveground steel tank

3. Constructive measures to preserve the bearing capacity and increase the safety during exploitation

Aboveground steel tanks, like all building facilities, are designed, executed and put into operation according to the current regulatory documents. The latter, however, are imperfect, but undergo continuous development and improvement. This means that it is very likely that old facilities, which have been in operation for a long time, do not correspond to the most recent achievements of science and practice. In addition, the aggressive action of the stored product and the environment lead to a continuous thinning of their structural elements. The constructive measures described here aim to preserve the

bearing capacity of facilities that have been in operation for a long time. This will increase the safety of the reservoirs, which will reduce the risk of [12]:

- premature decommissioning of facilities and their replacement with new ones;
- considerable material losses due to product spillage (the price of the stored product many times exceeds the value of the facility itself);
- ecological pollution of the environment;
- risk of fire and human casualties.

3.1. Increase the service life of the bottom

The bottom is one of the elements of the tank that is most intensively subjected to corrosion. In the process of operation, its thickness decreases and the risk of a break in it increases significantly. Considering the accumulated experience of the rate of corrosion in various types of stored products and the need to guarantee a 10-year interval between two complete internal inspections, the following measures can be taken to increase the service life of the bottom:

- putting an internal protective coating on the bottom, for example, special paints, which have a guaranteed service life not less than the period between two complete internal surveys, i.e. 10 years;
- installation of cathodic protection on the bottom.

3.2. Mounting of a double bottom of the tank

Mounting a double bottom does not increase the bearing capacity of steel tanks. It aims at quick indication of a hole appearance in the main bottom and prevention of penetration of product into the soil. In general, double bottoms can be classified according to the following criteria:

a) according to the material of the second (additional) bottom:

- steel;
- from a polymer material.

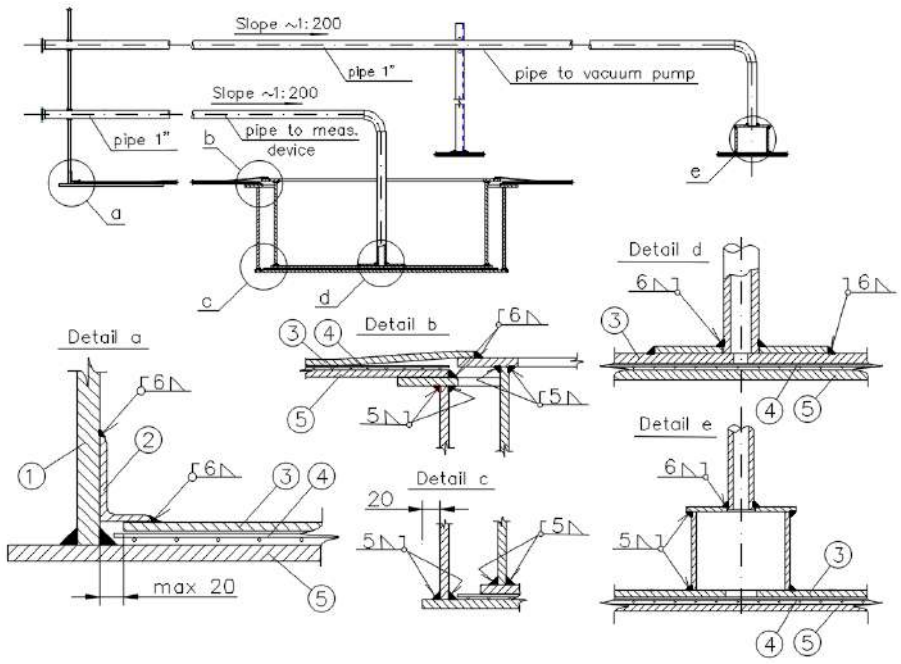


Fig. 6. Double bottom of the tank and active vacuum system

1. Shell of the tank *2. Angle section* *3. Upper (secondary) bottom*
4. Intermediate net *5. Lower (main) bottom*

b) according to the type of leak detection system:
- passive system – it is a system of pipes, through which the product caught between the two bottoms goes outside to a designated, easily accessible place. The liquid moves under the influence of self- weight. For this reason, the detection of a hole in the bottom occurs with some delay;
- a continuously operating active vacuum system - it constantly monitors for an increase in pressure between the two bottoms. Detection of a bottom hole is immediate.

The modern trend is to put a second steel bottom and an active vacuum system, see Fig. 6. The pressure between the two bottoms is decreased by a vacuum pump and its level is continuously monitored. Any increase in pressure between the bottoms is a signal of breaking their tightness. This scheme guarantees continuous control for the presence of holes along the entire bottom.

The sheets of the two bottoms are separated by a steel mesh of spot-welded smooth reinforcing bars with a diameter of Ø3 ÷ Ø4, see Fig. 7. It prevents the tight contact between the sheets and the creation of areas where the free movement of fluids is impossible.



Fig. 7. Mounting of double steel bottom of the tank

The disadvantages of the double steel bottoms discussed in Fig. 6 and Fig. 7, are:
- increased price and time for mounting;
- in case of a hole in the lower (first) bottom, the mass practice is not to remove the upper (inner) bottom, but to place a new, third bottom on it. This leads to more expensive repairs;
- in the case of a hole in the upper (inner) bottom, a product penetrates into the space between the two bottoms, which cannot be completely removed even after the tank is completely steamed. Residual product can catch fire and even explode in fire operations. It follows that in this case fiery works, such as gas flame cutting and/or welding, are inadmissible.

3.3. Removal of the assembly lap weld in tanks with a roll-mounted shell

In a significant part of the tanks in Bulgaria, whose shells are mounted by roll method, the assembly vertical weld was filled with overlap and double-sided welding, see

Fig. 8. This is contrary to the requirement that the vertical assembly joint of the tank shell be realized by butt welding with full penetration. Repairing works on the tanks must include the replacement of the double welded lap joint of the shells.



Fig. 8. Vertical joint in the shell, executed as a lap weld

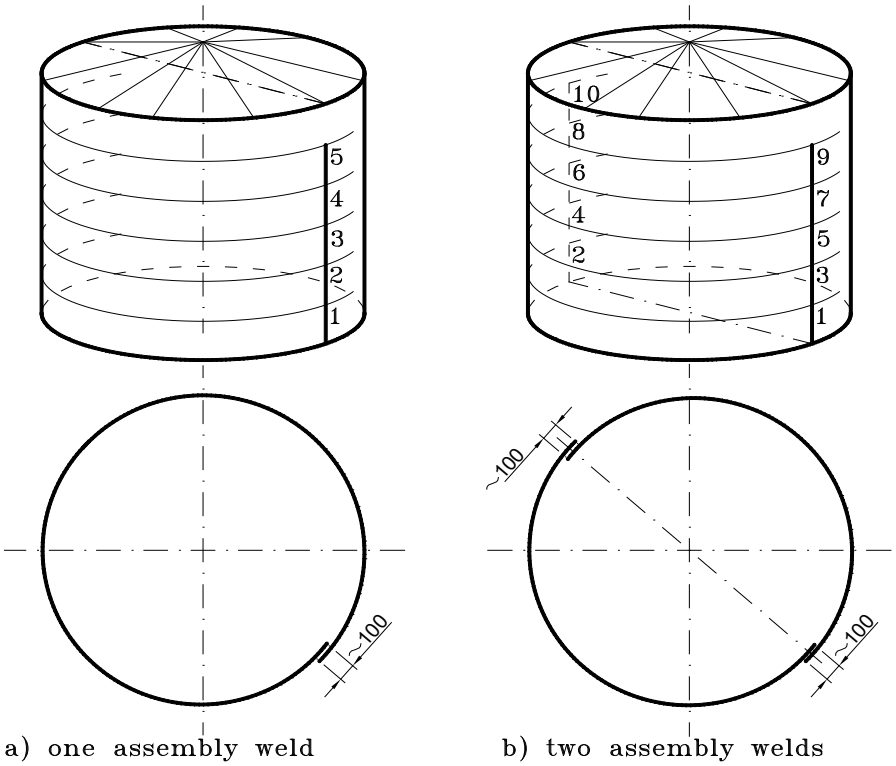


Fig. 9. Location and scope of repair works
along the mounting joints of the shell

The accepted technological sequence for the repairing works on lap welds is as follows:

a) the vertical lap weld along the shell is divided into sections, where repairing works start from the first course, see Fig. 9;

b) a section of the assembly weld is prepared, repaired and controlled;

c) after the final of repairing works and control on this section, the operation is repeated on:

- on the other assembly weld, if there are more than one vertical assembly weld;
- on the same lap assembly weld, above the already repaired section.

The size of a section of the shell where an overlap assembly weld will be removed and a full-weld butt assembly weld will be performed at one time shall not exceed the following values:

a) the length of the section is not greater than 3 m;
b) the section must not cover more than two belts at once;
c) the length of the section should not exceed 1/3 of the shell's height.

3.4. Mounting of intermediate stiffening rings on the shell

The intermediate stiffening rings on the shell, see Fig. 10, are intended to strengthen it and prevent local buckling from wind load and/or negative pressure. They are often applied to tanks that have been in service for a long time and whose shell has been thinned by corrosion processes.



Fig. 10. Intermediate stiffening rings on the tank's shell

Methodologies for determining the maximum distance between them and their minimum moment of resistance are specified in [1, 2, 3, 8]. It is necessary to note that according to the methodologies in [1, 3, 8], the radial rings stiff the shells against buckling from loads in the radial direction (wind and/or negative pressure). The methodologies in question do not consider forces in the longitudinal (axial) direction. There is some development on the matter in [2], where the longitudinal forces in the shell also are considered when determining the locations of these intermediate rings.

3.5. Using internal floating roofs

The installation of internal floating roofs does not increase the bearing capacity of the tanks. However, they are the easiest and most economical way to reduce the product vapour losses in tanks. They can reduce evaporation of stored liquids by up to 98%. According to the materials used, pontoons are:

- steel - their construction is very similar to that of external floating roofs;
- aluminium - the most widespread in practice, see Fig. 11;
- polymeric - have problems with sub-layer fire extinguishing.



a) non-contact roof type



b) contact roof type [7]

Fig. 11. Aluminium internal floating roofs

Other advantages of aluminium and synthetic floating roofs are:

- corrosion protection is unrequired;
- high operational safety;
- the installation of the roofs is carried out by a small number of workers inside the tank in a short period and does not depend on climatic conditions;
- the elements of the roofs are brought into the tank through the cleanout door or the manhole, i.e. it is unnecessary to cut additional holes;
- all mounting connections are bolted, i.e. there is no firework.

The disadvantages of aluminium roofs include:

- the thin roof plates are placed on floats and are not in continuous contact with the liquid. In the space under the sheets, steam is formed, which passes through the peripheral seal and escapes into the atmosphere;
- the construction of these pontoons does not have sufficient rigidity, which imposes restrictions on the flow rate of filling the tank when the roof is in the lower position;
- the small stiffness of the roof structure leads to cyclic fatigue and destruction of the welds that join the floats to the vertical supports.

3.6. Replacement of traditional steel roofs with aluminium

Aluminium domes, see Fig. 12, are taking a bigger and bigger part of the tank's market. This is related to the increased requirements for environmental protection through the reduction of product losses and the reduced price of aluminium. These domes are often applied when covering existing tanks with floating roofs. Another area of application is the replacement of a supported steel conical roof with a self-supporting aluminium dome and an internal floating roof.

The advantages of replacing a steel roof with an aluminium one are as follows:

- reduced self-weight, which leads to lower axial load on the shell;
- they do not need to be painted with protective or aesthetic coatings, as a result of which they are comparable in price to steel roofs in the long term;
- have increased corrosion resistance compared to some stored products;
- can cover tanks with a diameter of up to 120 m, i.e. volume V ≈ 200,000 m^3;
- they are beautiful and have a very good aesthetic impact.

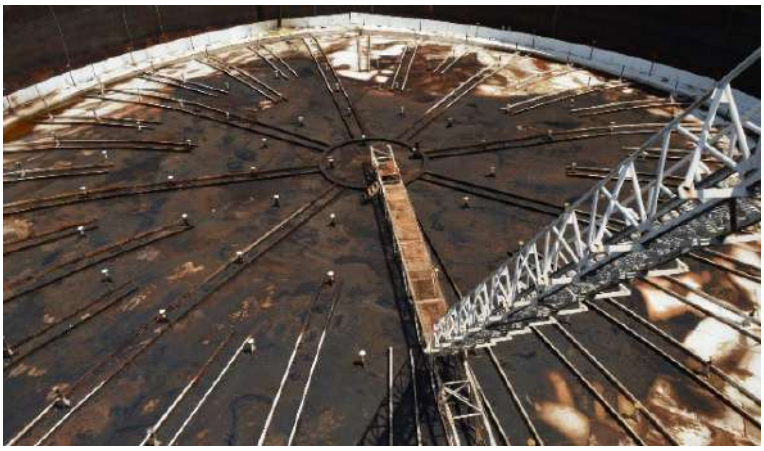


Fig. 12. Aluminium dome roofs on vertical steel tanks [7]

It should be noted that when designing the domes, the possibility of forming bags of snow on them should be taken into account. It is the unevenly distributed snow load combined with wind load that has caused the collapse of several domes in the United States and northern Russia. The key reason for these failures is that the σ-ε diagram of aluminium has no yield strength. This does not allow redistribution of forces between separate elements when one of them is overloaded. The lack of reserve capacity of aluminium as a material necessitates the very precise determination of the load on the dome and the resulting forces in the bearing elements.

3.7. Using additional stiffening elements on the membrane of the floating roofs

As a result of mounting mistakes and long-term operation, the membrane of single-deck floating roofs is deformed. As a result, part of the water does not reach the drainage, remains on the roof and accelerates corrosion processes. On the other hand, as the temperature of the product increases, the vapour pressure under the membrane increases, too. This can cause it to blow up in the middle like a balloon. Partially, the problems with deviations of the membrane from its design form are solved with the use of additional stiffening elements. They can be placed in a radial, see Fig. 13*a* or in a circular direction, see Fig. 13*b*.



a) stiffening of the membrane by radial elements positioned above



b) stiffening of the membrane by circular elements positioned below

Fig. 13. Stiffening of the membrane of single deck floating roof

When the stiffening elements are placed radially, it is more convenient to mount them above the membrane. In this way, all assembly welds are performed in the lower position. In the case when the stiffening ribs are placed in a circle, they must be placed from below the membrane. In this way, the removal of atmospheric water to the drainage is not hindered. But in this case, the mounting welds should be on the ceiling, which is inconvenient to perform. In addition, the implementation of anti-corrosion protection becomes difficult, see Fig. 14.



Fig. 14. Corrosion on the circular elements stiffening the membrane

4. Conclusion

Aboveground steel vertical cylindrical tanks are important facilities that are almost always filled to the maximum level. They often store toxic, flammable and/or expensive products. All this requires us to treat them with increased attention and responsibility. The structural measures described above to preserve the bearing capacity of steel tanks that have been in operation for a long time will significantly reduce the risk of these facilities to collapse, product loss and/or environmental pollution. Compared to the cost of the stored product, these measures do not cost that much at all.

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Относно живота и творческия път на проф. д-р инж.Димитър Димитров, по случай 100 години от неговото рождение

Михаил Цанков [1],

[1] *University of Architecture, Civil Engineering and Geodesy*
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Абстракт: *Докладът е посветен на високо професионалната научна дейност на проф. д-р инж.Димитър Димитров, като изтъкнат и всепризнат учен в областта на стоманените мостове у нас и в чужбина, по случай 100 години от неговото рождение. Направен е опит за представяне на неговото конструктивно творчество в съвременността, залегнали в основата на цялата му научна дейност, практика и публикации.*

Key words: *инж. Димитър Димитров, постижения в проектирането на ортотропни стоманени пътни мостове в република България*

1.Въведение

Проф. Димитър Димитров е роден на 03.10.1923 год. в гр. Ловеч(фиг. 1). Баща му е бил строителен инженер, майка му домакиня. Средното си образование завършва в Германското училище в София с отличен успех (5,72), през 1942 год.. След две години военна служба в общо-войсковия армейски полк в гр. Пловдив и в ШЗО София. Уволнява се със звание старши лейтенант. От 1944 след приемен изпит, проведен в гр.Ловеч, класирайки се на първо място е приет за студент по строително инженерство в София. Дипломира се през 1950 г. като дипломант в катедрата и постъпва на работа в ТРАНСПРОЕКТ, където проектира мостове и други строителни обекти, достигайки до нивото на проектант I степен. В периода 1954-1957г. е редовен аспирант по стоманени конструкции във ВИСИ. Темата на дисертационната му работа е със заглавието: "Влияние на пълзенето при комбинираните стомано-стоманобетонни конструкции". След това продължава да работи в "ТРАНСПРОЕКТ". През1958-1959г. е групов ръководител в Института за типово проектиране. Награден е със златна значка като рационализатор. Постъпва в катедрата като асистент през 1960г. От 1965 до 1968 г. с разрешение на ръководството на института работи към "Техноекспортстрой" като началник на Техническия отдел на строежа на новото международно летище в Багдад. През 1970 г. се хабилитира като доцент. В периода 1971-1972г. по съвместителство работи като директор по техническите въпроси в ДСО "Металстрой". Същата година е награден с орден"Кирил и Методи" II степен. През 1975г. е избран за зам.декан на Строителния факултет. Беше женен с една дъщеря.

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Фиг.1. Проф. Д. Димитров(1923-2009)

Инж. Димитър Димитров защитава своя хабилитационен труд за професор и получава академичната си длъжност на професор през 1976 год. Като преподавател в катедрата в началото е ръководил курсово проектиране на студентите от специалностите ПГС, ПС и ЖС. Бил е ръководител на дипломните работи на много студенти. Като лектор преподава дисциплината "Стоманени мостове" на студентите от специалностите железопътно и пътно строителство. Бил е зам. Декан е на строителния факултет от 1977 до 1980 г. Като проектант на пътни и ж.п. стоманени мостове за първи път у нас внедрява следните модерни технологии: ортотропна плоча, цялостно заводско заваряване и високоякостни болтове за монтажни съединения. Бил е ръководител на аспиранта: Радослав Славов. Бил е ръководител на колектива при разработване на ръководство и следните научни теми:

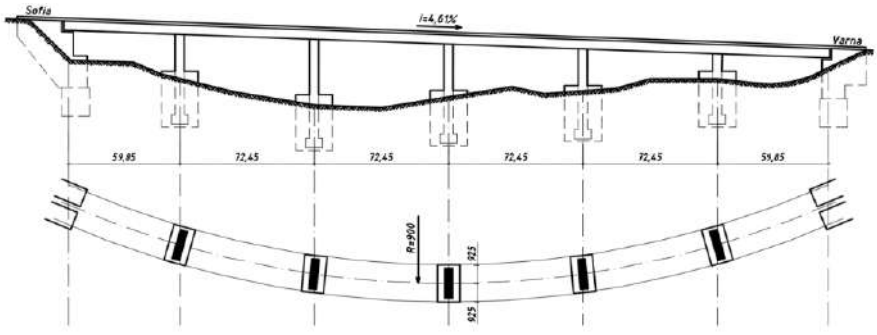
* Ръководство за изготвяне на упражнения по стоманени мостове; Подови конструкции на многоетажни сгради с метален скелет; Изследване на носещите конструкции на цирк "Глобус" и съставяне на технически паспорт за тях. Публикувал е много работи, както в нашата специализирана периодика, така и в германската (Stahlbau 1986, 1988). От известните ни 21 бр. тук са маркирани само следните публикации: Димитров Д. Г., Върху един малко известен подвижен лагер, сп."Строителство", 10/1970 год.; Димитров Д. Г., Избор на стоманата - главно условие за физическата дълготрайност на стоманените конструкции, сп."Промишлено строителство", 8/1972 год.; Димитров Д. Г., Взаимодействие между главни и надлъжни греди при стоманените мостове, Годишник том XXV, св.111. 1972-1974 год.; Димитров Д. Г., Влияние на напречните греди при поемане на усуквателни моменти в гредовите мостове, изд. „Техника”, 1963 год.; Димитров Д. Г., Усукване на стоманени мостове с две главни греди, стоманобетонна плоча горе и прътова връзка долу, Научна сесия 1980 год.; Димитров Д. Г., Подови конструкции на многоетажни сгради с метален скелет, сп. Строителство, 2/1975год.; Димитров Д. Г., Върху изчисляването на високи мостови опори, - хабилитационен труд, 1973 год.; Димитров Д. Г., Взаимодействие между надлъжни и главни греди при липса на спирателна връзка в ж.п. прътови стоманени мостове, сп. Строителство, 6/1970год.; Димитров Д. Г., Дулевски, Е.; Еластично изкълчване на прав прът, натоварен с променяща се по квадратна парабола нормална

сила, Год. Том, XXV сб.111, 1972-1974 год.; Димитров Д. Г., Усукване на стоманени мостове с две главни греди, стоманобетонна плоча горе и прътова връзка долу, научна сесия, 1980 год.

Автор е на проектите на следните големи стоманобетонни и стоманени мостове у нас:

* стоманобетонен гредови мост при гара Бов на три отвора 20+24+20m в хоризонтална крива с радиус 300m и косота в средата 38 градуса.
* стоманобетонен пътен надлез (запъната рамка с отвори 16-24-16m) при гр. Горна Оряховица.
* стоманобетонен ж.п. мост над р. Искрецка, под главен коловоз-непрекъсната греда в крива с радиус 300m и отвори 16-20-16m при гара Своге.
* стоманен ж.п. мост над р.Искър под главен коловоз- непрекъсната греда с отвори 42-54-42 m, с кутиено напречно сечение, приложено за първи път у нас.
* мост над р. Искър ж.п. линия Курило-Световрачене (максимален отвор 54,60m).
* Мост на автомагистрала Хемус, км. 26 (с максимален отвор 72 m-виадукт 1); (Фиг. 2).

Съоръжението представлява непрекъсната шест отворна греда с обща дължина 410м. Отделните полета на моста са с дължини: 60+ 4x72,50 +60m(Фиг.2а). Конструкцията на моста е разположена в хоризонтална крива с радиус 850m, в напречен наклон за всяко пътно платно 4.5% и надлъжен наклон 4.61%. За всяко пътно платно главната конструкция се състои от две стоманени греди с височина 3,600m, разположени на разстояние 9,200m една от друга. Между главните греди са монтирани през 3,150m една от друга вертикално, напречни прътови връзки. По долния пояс между главните греди е развита "Х,, – образна противоветрова връзка. В първоначалния проект на моста, пътната плоча е проектирана като комбинирана стомано–стоманобетонна конструкция. По време на монтажа на 07.05.1984 год. в лявата част на моста изпълнявана с технологията чрез „Потактово избутване" на сглобената зад единия устой конструкция настъпва аварийна ситуация. Това довежда до падане на почти цялата конструкция на моста с дължина 350m, върху капиталните ригели. При падането цялата конструкция се измества с 150cm извън дъгата на трасето. Заключението на държавната комисия постанови, че аварията се дължи на комплекс от причини, включвайки като основни: неравномерно нагряване, акумулиране на енергия, центробежни сили и др. Авариралата конструкция е възстановена в проектното си положение през юни – юли 1985 год. Аварията на моста предизвиква преоценка на първоначалния вариант на проекта. При пътно платно от комбинирани стомано - стоманобетонни конструкции, се дава ново решение с ортотропна стоманена конструкция(Фиг.2б). Същата се реализира чрез създаване на пътна скара от напречни стоманени греди с конзоли, които стъпват върху главните греди на моста през 3,150m. Общата широчина на пътното платно заедно с тротоарните конзоли е 13,70m. Мостът е въведен в експлоатация през 1986 г.



Фиг. 2а. Мост на АМ Хемус,км 26, 884(с максимален отвор 72 m);



Фиг. 2б. Мост на АМ Хемус,км 26, 884(с максимален отвор 72 m);

* мост на автомагистрала Хемус, км. 27(с максимален отвор 162m)(Фиг. 3). Известен като Виадукт – 2, той представлява непрекъсната греда на 3 отвора с размери: 100 + 162 +100m. Той е разположен в трасе с надлъжен наклон 4.61% и в хоризонтална крива с радиус 1407,25 в оста на всяко пътно платно. Напречното сечение е с кутиеобразна форма и със средна височина 5,0m и широчина от 6,50m. Ортотропната пътна плоча, стеблата и дъното са оребрени с плоски ребра. Дебелината на горната плоча варира от 12 до 16mm, на стеблата- от 10 до 14mm и на дъното – от 10 до 30mm. Общата широчина на пътното платно е 13,70m (фиг. 3). Дълбочината на премостваното препятствие в най-дълбоката му част под моста е 60m. Стълбовете са с максимална височина 42,50m. Мостът е подпрян върху 4 опори. Две от от тях представляват устоите от страна Варна и София и два стоманобетонни стъба под моста, изпълнени с правоъгълно напречно сечение и плоско фундиране.

Връхната конструкция е изготвена от стомана St 52.3 по DIN. Всички секции, изготвени в завода са изцяло заварени конструкции. Монтажните снаждания по горната плоча и надлъжните монтажни снаждания по дъното се изпълняват със заваръчни шевове. Останалите монтажни снаждания са на високоякостни болтове. За осъществяване на проектната геометрия бе изпълнен пробен монтаж на конструкцията в обърнато положение т.е с горната плоча надолу. Проектът за връхната конструкция първоначално бе разработен с отчитане конзолен метод на монтажа. По късно, по предложение на монтажната организация СМП „Метални конструкции", ИПП „Транстпроект", разработи проект за монтаж чрез надлъжно придвижване на конструкцията чрез хидравлични цилиндри. Тъй като стеблата на главните греди на моста са с много малка дебелина а укрепителните ребра осигуряват устойчивостта му само в експлоатационно състояние, за осигуряване възможността за монтаж чрез надлъжно преместване бе необходимо да се усили стоманената връхна конструкция.

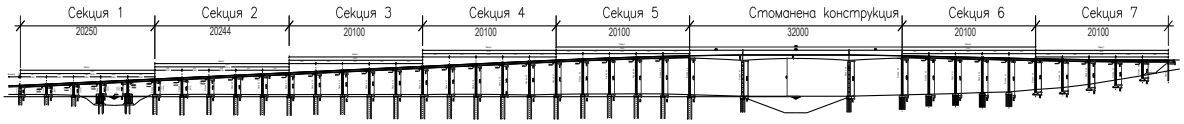
Основният проблем е осигуряването на местната устойчивост на стеблата. В резултат на обстойни изследвания бе възприет и осъществен вариант с допълнително надлъжно укрепване от външната страна на моста. Теглото на цялата метална конструкция, за всяко едно от двете пътни платна, изпълнени като ортотропни плочи възлиза на 2200 т. С изпълнение на нов вид балансьори, бе увеличена дължината на контакта с връхната стоманена конструкция по време на преместването и.Така се създадоха условия чрез крикове да бъде следена и корегирана големината на опорната реакция по време на осъществяване на определени монтажни операции. Конструкцията на моста е подпряна върху устоите чрез стоманени лагери, ролков тип „Кройц", а върху стълбовете еластомерни лагери на фирма „ GUMBA". Двата виадукта са мостови съоражения с най – големи премостени отвори по АМ „Хемус„ а Виадукт – 2 е с най – голям отвор в сравнение с всички подобни съоръжения у нас. Разходът на метал е: Виадукт–1: 400 kg/m^2; Виадукт-2: 420 kg/m^2 . По този показател те превъзхождат „Аспарухов мост„ във Варна и се подреждат сред най – добрите постижения в Европа. С въвеждането им в експлоатация бе завършен най – трудният участък от автомагистралния пръстен у нас. Мостът е въведен в експлоатация през 1986 г.



Фиг. 3. Мост на АМ Хемус, км 27, 500(с максимален отвор 162 m);

* Проф Д. Димитров, изпълнява функцията на ръководител на колектива и главен проектант на стоманената част на новия Аспарухов мост във Варна с отвори 80,50-160-80,50m, който е изпълнен с кутиясбразно напречно сечение, изпълнено за първи път у

нас и започнал да се изгражда през 1973 год. (фиг.4а). Мостът е с обща дължина 2050 м. Проектиран е като стоманобетонна конструкция, с 40 метрови предварително напрегнати стоманобетонни греди, с обща дължина 2050 м. , като частта от моста над канала е изпълнена от непрекъсната три отворна изцяло заварена **стоманена конструкция,** монтирана върху четири стоманобетонни стълба. Централният отвор над водата е с размер 160,00m, а двата странични отвора с размери по 80,50m(фиг.4б). Стоманената конструкция е изпълнена с кутияобразно напречно сечение с широчина 5,50m, и височина вариращa от 2,80m над крайните стълбове до 6.60m над централните. Пътната плоча на стоманеният мост е изпълнена като ортотропна конструкция с надлъжни ребра, от трапецовидно огъната листова стомана и напречни пълностенни греди. Устойчивостта на стеблата на стоманените главни греди е осигурена с надлъжни ребра, монтирани от вътрешната страна на гредите. К-образни стабилитетни връзки по дължината на моста осигуряват неговата усуквателна коравина. За производството на моста е използувана стомана марка St52 с граница на провлачане (f_y = 330 N/mm^2). Монтажът на моста е изпълнен по т.нар. полуконзолен метод, при който се използват по две временни подпори във всеки отвор. Издигането на частите на стоманената конструкция, с дължини между 12 и 18 метра,се осъществява с помоща на два кулокрана. Тротуарните конзоли са монтирани с помощта на автокранове, придвижвани по вече готовата основна конструкция. При монтажа на стоманената конструкция на моста са използвани 78 394 специални високоякостни болтове с различни размери. Мостът влиза в експлоатация през 1976 год.



Фиг. 4а. Големият Аспарухов Мост във Варна от предварително напрегнати стоманобетонни греди с дължини по 40m и стоманена част над канала; (80,50+160+80,50m)



Фиг. 4б. Големият Аспарухов Мост във Варна (стоманена част: 80,50+160+80,50m)

Както по отношение на възприетата носеща конструкция, така и по отношение на размерите на моста, беше главен проектант. За тази му дейност е награден с орден "Г. Димитров".

PROTECTION AND RENEWAL STRATEGY PROPOSAL FOR THE CITY OF PRIZREN'S OUTSTANDING VALUE

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Abstract - Basic principles for cultural heritage protection, governance and usage for the City of Prizren's outstanding value sites are discussed in the paper. Positive and negative characteristics of actual state of cultural heritage of Prizren are numbered. Starting points explaining the reasons why, how and which cultural assets have to be selected and stressed by the strategy are elaborated. Strategy concept, which apart the protection, define the managing process and cultural heritage usage models is exposed in brief. The study of quantitative and qualitative cultural heritage potential is presented, according to international, theoretical and practical, experience. The priorities, which have to be realized as soon as possible, or the precious cultural heritage of Prizren will be lost are pointed at the end.

Keywords **–** Prizren,built heritage, renewal strategy, economic potential, scenario solutions

1. Introduction: historical basis for protection of prizren's outstanding value sites

Kosovo is a small region in the Balkans, with a cultural heritage that is well known in the world because it holds a very significant place in the world's sacred histories. Often mentioned in the history of Byzantine Empire, medieval Serbian state and Ottoman Empire, the region arguably possesses some of the most important orthodox churches, monasteries and Islamic mosques and sites in the Europe.

One of the most important treasures in the region is the historic city of Prizren. Some researches believed that Prizren is one of the most beautiful towns on the Balkans, consisting of high esteemed historic, cultural, religious, artistic, built and intangible values. There the fragments of medieval Christian archaeological and architectural remains are organically incorporated in an urban tissue and oriental way of life. The importance of the city through its glorious past was reflected in the sacral and public buildings of outstanding value such as Fort Kaljaja, residential area of Podkaljaja (Nenkaljaja), old town centre, monastery of Holy Archangels and Fort Visegrad, Sinan-paşa mosque, Syzi Ćelebisë mosque, the complex of the Albanian League of Prizren, and many others. [1]

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They are all listed as cultural monuments of great or outstanding value. Only one, the church of Holy Virgin of Ljevisa is part of the Medieval monuments in Kosovo inscribed on the UNECSO World Heritage List. Unfortunately, the church is also on the UNESCO Danger List. It could be state that other cultural heritage do not attract the attention in the way it deserves.

Due to lack of maintenance, in few examples due to extensive inadequate conservation interventions, and mostly due to the recent ethnic conflicts, the cultural heritage of Prizren is under serious threat. Some monuments were destroyed by the fire and blasting since 1999 so their construction and aesthetic appearance were seriously disturbed, especially in their interiors, in some examples completely destroyed. [2] Due to the historical turbulences at the end of 20th and in the first decade of 21th century, the outstanding sites of Prizren now definitely suffers of complete degradation, and the need for new protection and renewal strategy is urgent [3].

2. Brief insight through Prizren's urban and cultural development

Prizren has a very long urban history as the archaeological remains testify the existence of a Roman settlement. During the period of medieval Serbia Prizren belongs to the type of the settlement which formal characteristics represented the result of a very long and multilayered process of settlement transformation which covered the period from the breakdown of the Roman Empire till the fall of Byzantium.

Unlike these forms, the settlement of oriental type, which had begun to appear after Turkish conquering the area of Kosovo, has a different developing course. Apart from the elementary religious structures, all other structures of the type were built according to the needs and number of inhabitants of the town. The oriental type cities mainly consist of two different urban structures, of different size; commercial and trade centre (çarsi) and housing quarters (mahalas). The concentration of commercial buildings is the essential symbol of the economic life of these cities.

The Turkish conquering of the Balkans imposed deep changes to the position, nature and role of city parts. The major changes could be felt up to 16th century. At that time the Balkans had become a part of unique state and economic area in which the settlements developed in the different ways.

Prizren is excellent example of the settlement developed by bridging or linking of two different not distanced city entities. The very old city of Prizren experienced a sudden development in the 14th century that led to the defining of tree zones of urban contents that also cover certain periods in the development. The oldest part of the city was formed by shaping a group around a non-fortified settlement with the Episcopal centre of Holy Virgin of Ljevisa church as a domineering structure. Then the fortification with the out skirting belt was formed on the top of the hill, while the marketplace was created in the third phase, on the side of the fortification. The creation of all these phases is related to some parts of the city directed towards the Bistrica River which runs at the spot and which had changed its course [4].

This was still a period during which the Turks, after conquering, were tolerable towards the religion of the inhabitants, so that the physical form and the social structure of the inhabitants was slowly changed. This can be testified by the stagnation through which Prizren had undergone from 1459 till the half of the 16th century (Fig.1) [5].



Fig.1: Church of the Holy Virgin of Ljeviša and Sinan pasha mosque *(photo archive*: *http://www.spomenici.heritage.gov.rs/lat/nkd/pregled/crkva_bogorodice_lje; http://dtk.rks-gov.net/tkk_objekti_en.aspx?id=1268)*

Today's Ćarsi, called Arasta, which only testifies about the age of its position, had been formed, however, in the peripheral part of the medieval settlement on the right Bistrica bank. Fortress Kaljaja is of structural, urban and historical importance, since it has preserved traces of Byzantine, Serbian, and Turkish fortification buildings methods. By using the inherited fortified castle, strictly for military purposes, Turks ménage to get this type of necessary structure without any greater efforts.

Several churches from the 14th century, located on the foothill of fortress, besides historical and architectural significance, they also have urban importance, proving the existence of settlement, s*uburbia* type, below the fortified city. From the Turkish period the most important heritage is represented by the elements of urban plan structure, bridges, river canal, sewerage system, springs with drinking water, and public bathhouses.

Among the most important profane structures are the housing buildings of Potkaljaja (Nenkaljaja). Even though they are the oldest examples of this type of structures from the end of 18th and the beginning of 19th century, they also exhibit the development of residential houses in Balkan cities during an earlier period, influenced by the living conditions, religious and class status of the Prizren citizens. This part of the city kept fully its character, scenery, ethnic structure, and presents a unique ambient of urban whole (Fig.2)..

The outskirts that also developed in the previous period, in Turkish period remained settled by the Christian inhabitants. This can be testified by the name of mahala – Varos, which is territorially identical with the former medieval out skirting.



Fig. 2: Dominant position of the fortress Kaljaja and remains of the houses of Potkaljaja (*photo archive:http://mirjanadetelic.com/leksikon/gradovi/sl/6266270_1.jpg; http://www.kosovo.net/burn.html)*

3. Key issuess of cultural heritage strategy

Identification and critical analysis of key issues of protection, arrangement and use of cultural heritage and Prizren city identity is carried out as an overview of available documents and personal observations which analyzed the condition of the area of immovable cultural goods. Some of the standpoints were based on the data from scientific publications, statistical reports, and analytical papers from various media.

The main areas of motion and development of cultural context do not coincide completely with municipal borders as they are defined today. According to natural and social conditions, the conclusion is that:

- The topology of Prizren is diverse and its history is complex, which results in rich and valuable cultural heritage;
- Historically, some parts of Prizren have been developed in different ways and in different time as integral parts during the ancient civilizations, Byzantine and Turkish culture, and during the time of socialism and transition;
- Prizren was during the history a pluralistic society; today predominant ethnic group consist of Albanians, but still there very small groups of Serbs, Turks, Roma, which all have their identity and cultural heritage;
- The recent history of Prizren is abounding with conflicts and problems in various areas – social development, economy, politics, culture, religion, social ethics, etc.
- There were a lot of different spatial and urban plans prepared for Prizren which were formed according to developmental priorities which differ in some segments with the real developmental potentials, which particularly apply to the field of cultural heritage. A careful analysis shows that in many former plans there were some highlighted policies which were promising a change in treatment of cultural heritage, but they lacked proper enforcement in practice. [6]

This conclusion serves as a starting point for the SWOT analysis. The analysis shows that the greatest strength is outstanding cultural and economic value of built heritage; the greatest weakness is the inability of local communities to take advantage of these resources. Possibilities

for the use of cultural heritage are numerous and achievable, but the threats are mainly presents in the form of a negative impact of the activities of people living in the city.

4. Approach to the preservation and renewal strategy proposal

The course of the development of formulating for the establishing the theoretical grounds of the study and preservation of cultural heritage is not the sole approach; there are also other reasons for the protection. Undoubtedly, one of the most important reasons is the national and political need. The other reason is the strong and fast expansion of the development of the society, especially in the last few decades, which resulted in the acceptance of new values. These values put a great pressure on the remnants of Prizren's cultural values, and even pushed them to the margins of social importance. Somehow, the study of built heritage is considering necessary and positive, whereas preservation is not always seen as necessary and indispensable. In the community of Prizren it is often seen as hindrance to the development of the environment and its degrading factor.

The preservation of heritage, especially built heritage, is not consider to be an important political issue, and therefore this field is rarely discussed seriously at the high levels of government. One of the answers lies in the fact that for them is very difficult to define clearly and comprehensibly why to take care of built heritage in the first place, while they have to make decisions on so many other problems. But, the relationship of a community with its heritage is primary emotional, and it is more often than not the matter of the heart and not the mind. There is no point in a single element of heritage unless the society it belongs to feels affection, devotion and love for it. However, in order for those emotions and memory messages to reach others, who do not share the same feelings, it is necessary to explain the values in written way. [7] Through the last two decades huge demographic changes in Prizren occurred and that have caused the present hostile attitude of the population towards the built heritage from the period before the Ottoman rule.

The physical aspect of cultural heritage contains a complex of values which correspond to various needs of the community, and is the result of accumulated human work at a certain time and space. These values cannot all be accepted at the same time, as they are sometimes confronted by each other. But, it is important to realize the need for defining the presence of cultural heritage, and to possess the ability of formulating the reasons for this. Therefore, it is necessary to establish a transdisciplinary method of strategy making, which can bridge the deficiencies of communication, which certainly contributed to the specific present day situation of Prizren's built heritage.

5. Preservation and renewal strategy proposal

The proposal is based on the statement that in the actual politics of the protection and renewal of cultural monument and sites all the possibilities that are available are not considered. Different approaches and methodological concepts of space and physical planning, conservation, maintenance and usage of Prizren's heritage offer possibilities for reconsidering and improvement of cultural heritage as a driver of local economic development. Different types of cultural heritage contribute to the valuable built surrounding and traces the directions of the cultural routes, basic focal points and image that characterize the borderland where overlapping medieval Byzantine and Serbian, and later Oriental, Islamic architecture and urban patterns. It

was perceived that it was not done enough to present the whole meaning and universal value of Prizren cultural heritage in local, regional, and international frameworks.

Renovation of Kaljaja (fortress) started in 2012 by the Archaeological Institute. Donations came mainly from the Ministry of Culture of Kosovo, but the largest investment, in the amount of two million dollars, was donated by the US Embassy in Kosovo. This restoration was carried out by archaeologists from the Archaeological Institute of Kosovo and was based more on restoration and excavation, while CHWB (Cultural heritage without border) dealt with the promotion of this valuable cultural monument (Fig.3).

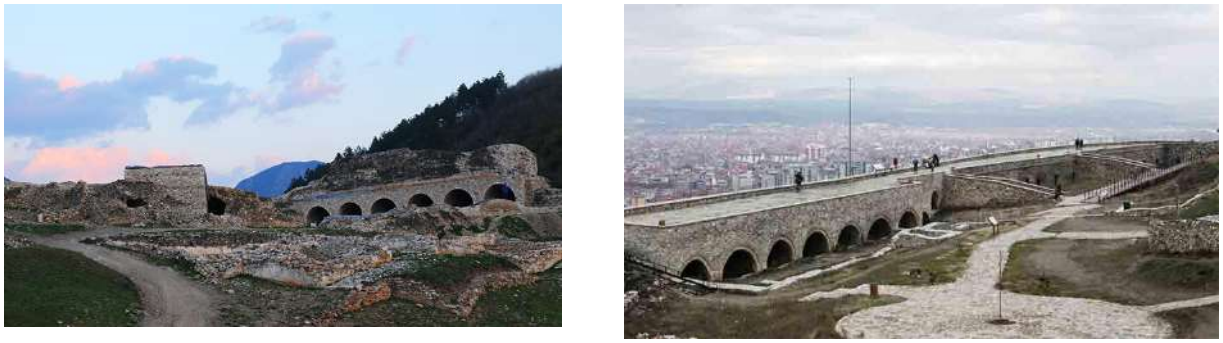


Fig. 3 Kaljaja (fortress) during archaeological research and after reconstruction *(https://www.info-ks.net/vijesti/lokalne/54204/prizren-na-kaljaji-pronadeno-50-topovskih-duladi; https://www.info-ks.net/vijesti/lokalne/86441/prizrenska-kaljaja-turisticki-i-historijski-biser-kosova-fotovideo)*

Among different strategies, the proposal suggests the more efficient mechanism of the protection in connection with the series of legal and economic instruments. [8] It implies the new formulation of purposes and aims of the protection – the transition of the processes of the integrative protection and extensive restoration of the great parts and complexes. In that process of the total protection and restoration, the only guarantee of success is the wise introducing of the cultural heritage into the economic processes. On the other hand, it demands the search for the balanced compromise between the doctrine of the protection, everyday life necessities and the lawfulness of the economic circulation. [9]

The reasonable protection of the cultural heritage includes the aspect of the creation of that which is recognized as the "cultural society"; it means the accepting the fact that the cultural sector have to be recognized as the economic one. By this approach new business, market and incomes can be created, but above all this will generate real protection of built environment harmonized with building innovations. That is why defining of the presumptions and aims of whole city are needed. It includes an overview to previously determined strategies and marking of the strategic directions of the future development.

Starting from the fact that the cultural and historical heritage of the city is one of the basic initial factors in the process of modern planning, the strategic directions in the domain of protection and improvement of built heritage can be formulated as:

- Adjustment of the development policy related to the protection of built, and all cultural heritage, with the complex development of the city in general;

- Adjustment of the program related to the domain of protection of built heritage with the programs relating to other domains which are active participants in planning in order to create optimal conditions for urban renewal of Prizren;
- Adjustment of the program in the domain of protection of built heritage with the requirements and progress of the modern development and the conditions of social and economic changes in the society in transition;
- Provision of an uniform realization of the program for protection of built heritage through the phases, in this way creating the conditions for the acting which is of general and permanent significance for the social life;
- Defining of the typo-morphology of Prizren for purpose of creating a permanent image about starting and general characteristics for planning of whole city;
- Determining a basic principles of evaluation for all forms of built heritage which exists in Prizren, as that is now a little bit elusive;
- Integral treatment of the historical center of the city with determining the principles for gradual evaluation of the spatial cultural and historical units inside the focused area and classification of minor areas within the historical city center;
- Evaluation and classification of particular spatial and historical sites outside the historical city center.[10]

In recent decades, partial renovations of the old parts of the city have begun, but also a drastic change in the general image of the city. Namely, in certain parts of the city, new residential districts are planned, which is necessary for the social and economic prosperity of the city [11]. With this, however, Prizren will significantly lose the compactness of the urban fabric, the recognizable urban matrix and exceptional sights (Fig. 4).



Fig. 4 Old photo of Prizren under the fortress (Potkaljaja) and New residential and commercial buildings design https://canprojem.com/en/kosova-prizren-2/

In the eighties of the twentieth century, Potkaljaja, as an extremely picturesque and densely populated part of the city, attracted the attention of many architects. One of the proposals designed by arch. L J. Folic, only in recent years has been accepted as a model that is copied in different variants and for different purposes (Fig. 5 and 6).



Fig. 5: Solution proposal for the houses of Potkaljaja transformed in touristic pensions *(arch. Lj. Folić)*





Fig. 6 Renovated residential houses and a hotel in Potkaljaja *(https://balkaninsight.com/2020/01/14/return-to-kosovo-the-serbs-who-re-embraced-their-hometown/; https://www.aa.com.tr/ba/balkan/prizrenska-kaljaja-turisti%C4%8Dki-i-historijski-biser-kosova/1044510)*

6. Concluding remarks

Out of the abundance of new attitudes regarding the protection of the built heritage available today in the world, we have selected only those which have been rarely applied in our circumstances, while the study has found out that they should be represented to a greater extent here. These are the changes in evaluation of the built and spatial heritage in relation to the period in which they were established, then selecting of the basic groups of elements during the work on the built heritage, and evaluation of typological features of the elements of the spatial cultural-historical areas. The special group of consideration is represented by finding out of new solutions in the domain of historical areas (urban archaeology, usage of the sub-terranean space in order to preserve surface terrain integrity of the area).

In the domain of methodological planning apparatus, two levels can be discerned clearly: its modernization in general sense and development of specific patterns for the needs of planning in the protected historical urban core. Following the principle that a problematic approach to

planning of protected city core should be applied, we also considered the planning within the context, as well as establishing of communication between the protected urban core and the wider parts of the city. As a result of a discussion, it was pointed out to the need for a higher degree of determination than it is usually done.

Summing up the knowledge about protected historical monuments and areas in optimal determination of the potentials for its development, and the development can be considered through studying and by proposing variant solutions which are existing in the practice of many countries and known under different terms (scenario, variants, options, etc.)

In the considerations of the quality of the physical space of the protected urban areas, a special attention has been paid to the transformation in the approach of formation of the urban area. The urban area, especially the historical one, performs the communicative, informative and knowledgeable function and as such it conditions the manner of our expert work. The most sensitive is knowledgeable function which deals with diversity of multi-sensual feeling of the space and possibilities of "making the space alive" through personal activities of the users through which the space quality called the richness of the space is achieved.

The following basic aims have been defined:

- Enlarging of the historical city core down the Bistrica river and reviving of the river banks;
- Putting an end to the further degradation of built heritage and historical urban areas;
- Reviving of the economic potential of historical areas;
- Differentiated preservation of built heritage;
- Upgrading the quality of the environment.

Having analyzed the series of phenomena and processes of all categories, it was possible to determine the potential of the focused areas which cover the spatial, natural, social, ecological and economic potential of the space.

By applying this knowledge and conclusions to the variants of scenario for strategically options, it was also possible to name the main factors which feature certain scenarios.

Scenario I: maximum protection of built heritage is in the first place, as well as restoring of the morphological structure of the old Prizren, specific culturological relationship towards the heritage and careful implantation of the new elements in the old context;

Scenario II: it is based on the modernized treatment of the protected historical city cores, on the preservation of the historical buildings without subordinated to them, on reviving of economic potentials of the historical areas, on activating of the passive and abandoned zones, on integration of the historical area into the modern city development trends and on considerable reviving of the touristic potentials;

Scenario III: it is based on contrasts, on the spirit of a modern city, on the fact that there are certain economic possibilities of protected areas, that the economic flow allow the development of a business and touristic zone of Prizren, with maximum touristic potentials.

It is almost impossible to discuss the protection and restoration of historic urban areas, and that the greatest attention is focused on a direct link between integrative conservation and sustainable development. This paper seeks to explain the elements that drive and create these links in the historic urban areas. Some basic conclusions on cultural policy are drown, which must be defined by characterization of urban landscapes and neighborhoods. The strategy proposal affirms the historical urban area and is in line with European views on the community neighborhood, city, region and country.

Despite the strong cultural diversity posed by migratory movements in the Prizren, and that has to satisfy everyone's desires and to be readily developed, protection and renewal of historic urban areas should rely on common values adopted by the people who coexist in a given space and partially or completely transform the social structure that has historically created Prizren. In this context, protection and renewal strategy proposal for outstanding Prizren sites encompass different solutions which vary in a wide but delicate range from satisfying basic consumer needs to the creation of high professional range.

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TOGETHERNESS OF SWISS AND VENEZUELAN PAVILION IN THE ARCHITECTURAL BIENNALE DI VENEZIA 2023

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Abstract: *The Swiss Pavilion and The Venezuelan Pavilion are neighborhood pavilions, where through main entrance of Giardini area, the first pavilion at right is the Swiss Pavilion, that was designed by Bruno Giacometti in 1952, and the second one is the Venezuelan Pavilion, that was designed by Carlo Scarpa in 1956. Both of them are nearly 70 years old. Their architectural design and structural design are unique as pavilion architecture, with steel mostly at roof level and with glass to get the daylight in.*
This year, the curators designed both of them as sister pavilions, by letting a walkway from Swiss Pavilion to Venezuelan Pavilion. They put a carpet over ground with a design of architectural drawing in scale 1/100, original, by B. Giacometti, including both pavilions in the plan.

Key words*: Venice Architectural Biennale 2023 in Giardini, Swiss Pavilion, Venezuelan Pavilion, structural steel*

1. Introduction

In the paper, the architectural design together with structural design of the Biennale Pavilions will be examined separately and then together [1] [2].

It will be very interesting to investigate that the buildings, built 70 years ago, are very suitable to be connected. This feature of the pavilions shows how an architectural design may be a universal design, free from time (Fig.1).

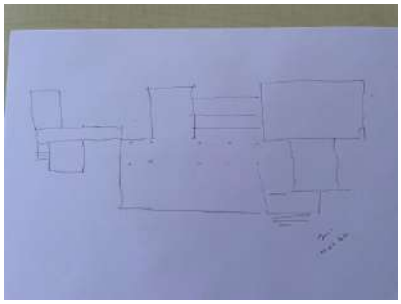


Fig.1. The architectural plan of both Pavilions (drawn by Y.K.Aktuglu)

2. The feature of the Pavilions

The Swiss Pavilion and the Venezuelan Pavilion are having a rectangular plan for spaces. Both of them are having windows and transparent facades at roof level, to get the daylight in, with a positive effect on the exhibited goods (Fig.2).



Fig.2. The frontal façade of both of them (photo taken by Y.K. Aktuglu, on 29.09.2023)

This time, the white and black carpet over the whole area of the larger space of Switzerland Pavilion, lets visitors feel a wider area, as they are at their own home, with the help of white color, on the floor, and over the walls, together with light coming from the top (Fig.3).



Fig.3. The inner view of Swiss Pavilion with carpet over the floor and with the structural steel trusses at the roof level, with V-shape roof (taken by Y.K. Aktuglu, on 30.09.2023)

Similar in Venezuelan Pavilion, the daylight comes directly from top parts of the walls and also from the roof level(Fig.2)(Fig.4).

The roof structure of the Switzerland pavilion, is having a V-shaped outlook to inner part for the larger space (Fig.3), and a conoid roof form for the smaller space, supported by structural steel both.



Fig.4. Inner view of Venezuelan Pavilion (photo taken by Y.K. Aktuglu, on 30.09.2023)

In the Venezuelan Pavilion, the design of the roof structure is more calm, with stripes (Fig.2) (Fig.4).

The most important part of these pavilions is the intersection part, where the visitors flow from one to another, automatically (Fig.5) (Fig.6).



Fig.5. The connection part of two pavilions, from Switzerland to Venezuelan (photo taken by Y.K. Aktuglu, on 30.09.2023)



Fig.6. Connected part, in Venezuelan Pavilion (photo taken by Y.K. Aktuglu, on 30.09.2023)

For all features, structural steel is the main structural material to make the design be built in reality.

3. Conclusion

After visiting two pavilions, Switzerland and Venezuelan, it could be easily written that, both pavilions are full with daylight, with their rooftop windows.

The structural steel trusses to span the Switzerland Pavilion’s roof level, tranfer the loads through cantilever structural steel triangular trusses to the perimeter walls, and then to the foundations.

In Venezuelan Pavilion, the upper walls of the major and minor spaces, are designed with transparent and solid sections, continue also at the roof level. The rest is constructed with solid perimeter walls.

The idea to connect the two pavilions, helps visitors to pass from another in an easy way, in a continuous flow.

The magical white carpet, with the architectural plan of the spaces, over the ground, lets the visitors feel themselves as over a flying carpet, as in the story of Aladdin and Magic Lamp.

As a conclusion, to support the idea as Neighbors:A Manifest (Fig.7) , the architectural planning and the structural solution of the Pavilions as exhibition pavilions are overlapping in a perfect way to highlight the pavilions are looking like statues, which is the sacred aim of architectural design and structural design.



Fig.7. Neigbors: A Manifest (photo taken by Y.K. Aktuglu, on 29.09.2023)

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HISTORY OF SHORT SPAN ORTHOTROPIC BRIDGES CONSTRUCTED IN CANADA AND THE USA

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Abstract: *This paper will describe innovative bridges built and in operation. Short span bridges with orthotropic steel decks in North America are very rare. Many believe welded orthotropic steel decks are too challenging to design and offer little benefit to the owners of short span bridges. The advantages will be summarized. Featured bridges of North America are the Ulatis Creek I-80 Test Bridge, California (1966); the Battle Creek Test Bridge, Salem, Oregon (1967); Indian River Bascule Bridge in Port Carling Ontario (1975); Cordova Alaska Ferry Terminal Articulating Ramp (1969); USA Navy Floating Pontoon Bridge with Bi-serrated Ribs (1968); the "multi-cell steel slab bridge system" is manufactured in Redding, California (2000); the Sagtikos Parkway of New York (1991); and the Braille Trail Pedestrian Bridge of Santa Rosa, California (1976). Bridges discussed in detail were selected to demonstrate the complete range of all short types. A comprehensive reference list will be provided to assist in obtaining more detailed information about these bridges and other short spans bridges can be time consuming. It is well beyond the scope of any paper to discuss.*

Key words: *short span, orthotropic bridge, steel decks*

1. Introduction

1.1. Ulatis Creek Bridge Vacaville, California, USA

Construction was completed in 1966. It is only the outside eastbound third lane a 5-span bridge with 6-bents for eastbound Interstate I-80. All spans are 8-M long and an orthotropic deck with "Split-T" ribs supports eastbound traffic. The engineers wanted to test the durability of five types of wearing surfaces. San Francisco Bay Toll Crossing engineers supervise this testing because the best material was selected for the wearing surface of the Hayward San Mateo Orthotropic Bridge. Ulatis Creek was repainted and a single replaced wearing surface was installed [the best of the 5]. The Ulatis Creek experimental bridge lane is still in service and the current owner is Caltrans, plus the original wearing surface for Hayward San Mateo Bridge is still in place after over 34 years [1].

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1.2 Battle Creek Test Bridge Salem, Oregon, USA

In 1966, the State of Oregon Highway Department designed the Battle Creek experimental bridge. Construction was completed in 1967. It is a 3-span bridge with 4-bents. The main span is 9.144-M long and the side spans are 6.858-M long. The bridge is 14.021 m wide. An orthotropic deck with trapezoidal ribs supports the northbound traffic, while an orthotropic deck with flat plate ribs supports the southbound traffic. The engineers wanted to test the durability of both open and closed ribs (Fig. 1). The bridge is still in service and the current owner is the City of Salem.

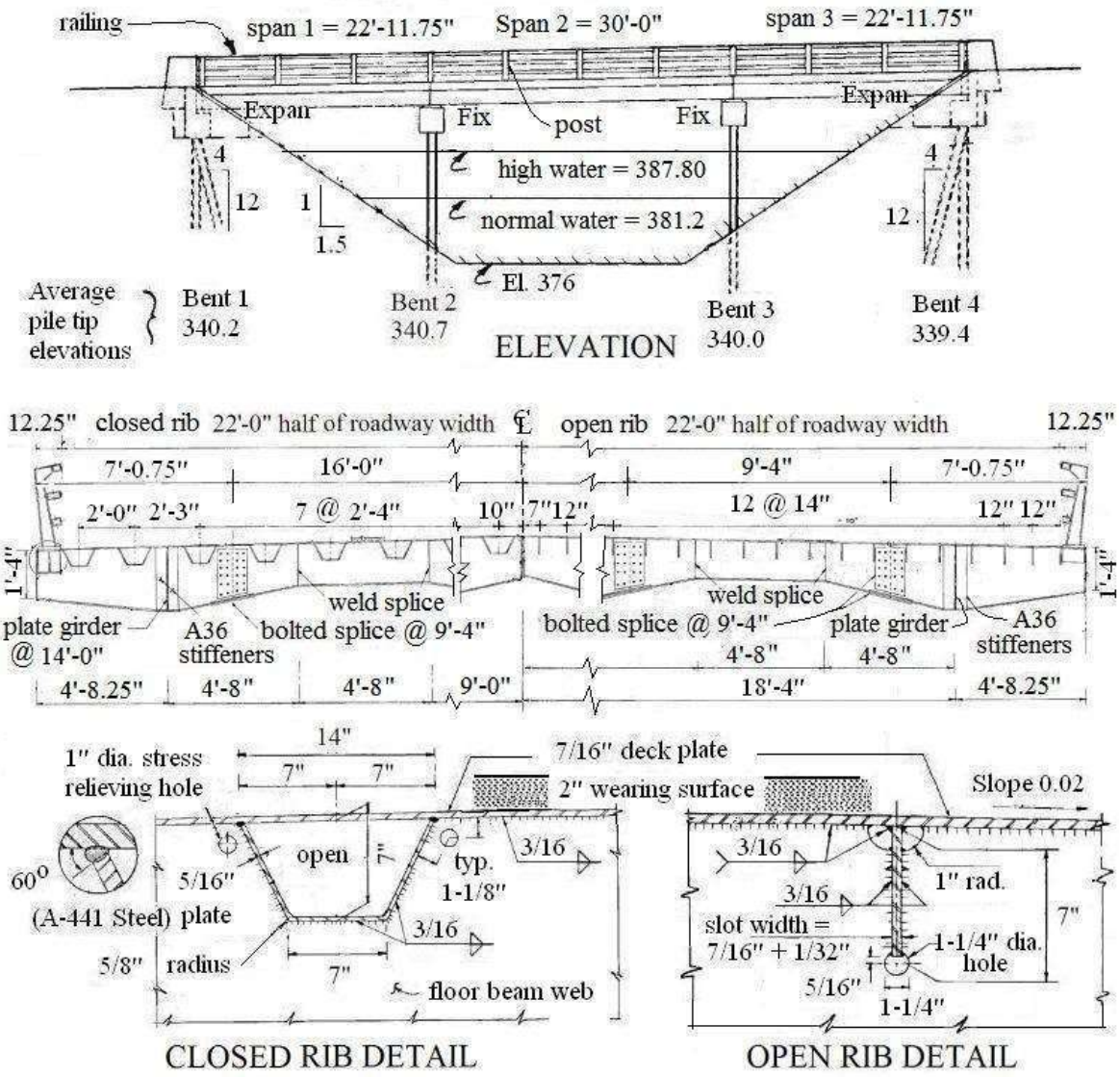


Fig.1. Battle Creek Bridge

1.3. Port Carling Bridge across the Indian River, Ontario, Canada

The Indian River Orthotropic bascule bridge at Port Carling, Ontario built by O.J. Gaffney Ltd., Stratford, was designed and administered by the consulting firm from Toronto. The first time the span was raised better than 50% of the asphalt concrete surface slid off. At the request of the Ministry, Bemac Protective Coatings was engaged to design and construct a new mastic surface for this bridge in 1975. The solution was 13-mm x 25-

mm steel strap plates that were welded at 61-cm centers in zigzag pattern, plus 13-mm diameter x 25-mm steel headed studs at 46-cm centers midway between these strap plates. Bemaphalt hot mastic was 50% washed concrete sand; 40% crushed limestone; 7% (85/100 Pen.) asphalt, and 3% Trinidad asphalt. This wearing surface has worked successfully.

2. Ferry Terminal Ramp Bridges Canada, Alaska, and Washington USA

In 1968, an articulating ferry terminal was designed for the Alaska Ferry system located in Cordova, Alaska. Campbell & Associates of Douglas, Alaska designed the 4.877-M wide by 36.127-M long all steel ramp. One end of the ramp hinges on the end of the City of Cordova's cargo dock. Two counter-weight towers are used to adjust the ramp to the deck level at the stern of the ferry. The elevation varies based on the tide and freeboard of that particular end loading ferry. The key dimensions of the American rib are a structural height of 241-mm, a base distance between the outside face of the trapezoidal legs of 330-mm, bottom width of 165-mm and a plate thickness of 6-mm or width. The rib and its structural section properties are shown in a table of American ribs in Reference [1]. Also, the spacing of the ribs was selected from this design aid. The steel deck is 10-mm thick, which also becomes the top flange of the two 1.178-M deep plate girders. Epoxy grit was used for a thin wearing surface of about 6-mm. The ramp was removed in 2000, when the Ferry Terminal was changed. The owner was the Alaska DOT/PF.

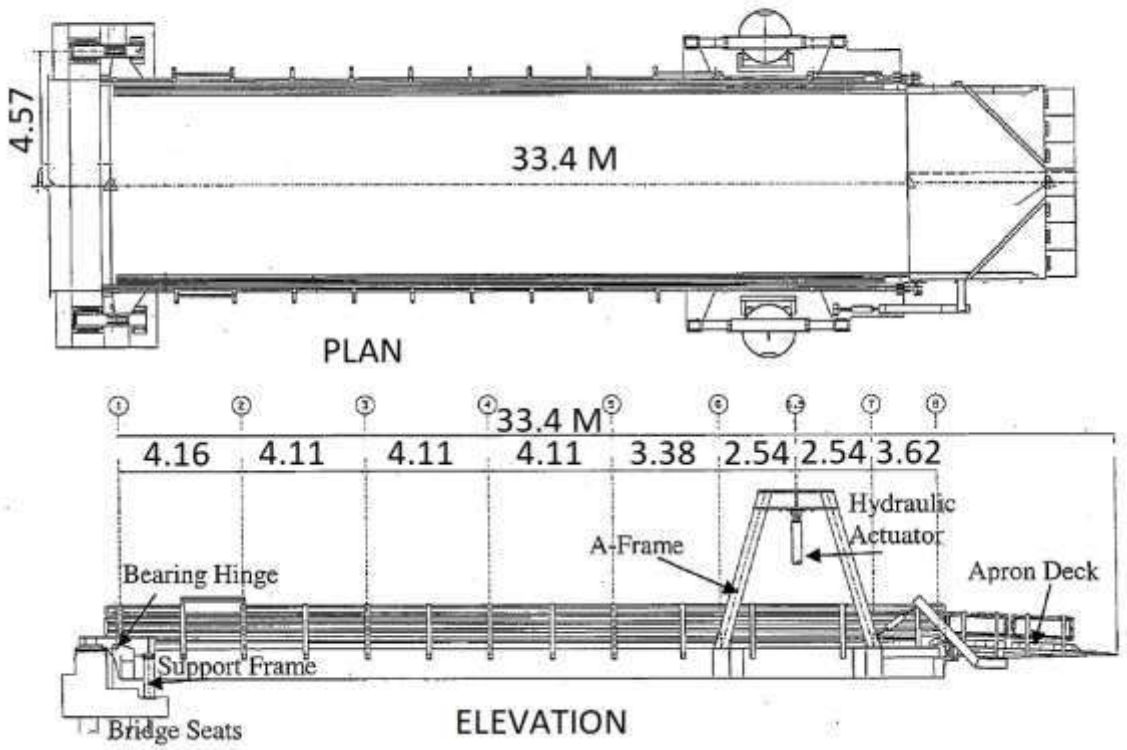


Fig. 2. Elevation + Plan State of design of three State of Washington Ferry Terminals in 2004

Similar ramps were built by the Province of British Columba Ferry System. Berger / ABAM [today WSP] designed similar ramps owned by the Washington State Ferrys [WSF].

3. Floating Pontoon Bridge U.S. Navy, USA

The U.S. Navy has experimented with the "Ammi" system or biserrated trapezoidal ribs to reduce the total dead weight of the pontoon (Fig. 3 & 4). The welded-steel pontoon bridge was conceived and developed to be mobile, lightweight, strong and durable. The first 80 prototype modules were built in 1966 and 1968. Its main component is a welded steel pontoon 9.754-M long, 1.829-M wide and 1.219-M deep. It is fitted with two detachable, rubber- tired wheels for towing. During towing it carries two 457-mm pipe piles and 20 orthotropic girders with web openings for lightness. The bridge pontoons may float or be elevated on the piles they carry. The deck beams link the pontoons to form 6.069-M modules. In combination, the Ammi bridge components can create a 6.069-M roadway capable of carrying a 54.4- metric ton army tank. The watertight body is made of 6-mm thick plates, stiffened of a novel shape, which the Navy's engineers designated as "biserrated-orthotropic." In the top and bottom these ribs, spanning transversely 609-mm on centers, are shaped as trapezoid ribs about 152-mm deep and have serrated upper edges. In the sides, they are of triangular shape and are placed vertical 304-mm on centers. The pontoon is also featured by 508-mm diameter cylindrical wells at each end for one mooring spud or support pile. Weighing only 37.23kg per linear meter, the section possesses great flexural and torsional strength. It is made of two parts: a serrated-edge trapezoid rib and a top closure or flange plate. The channel, in turn, is obtained by bending a plate to that form after the strip is cut along two parallel serration lines. The "cut-out" or interconnections. Secondly, it increases the depth of the section, and hence its strength by a height equal to the depth of the serration without an increase in the weight. In more permanent installations, welded connections can be provided by means of seat and ring segments placed at bottom and top of the pontoon spud wells. serration, which occurs in staggered pattern in the two faces, serves two main purposes: first it makes possible the use of full-penetration welds for the flange-to-web interconnections. Secondly, it increases the depth of the section, and hence its strength by a height equal to the depth of the serration without an increase in the weight. In more permanent installations, welded connections can be provided by means of seat and ring segments placed at bottom and top of the pontoon spud wells.

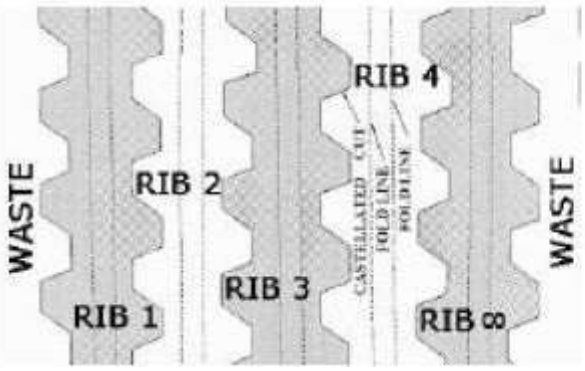


Fig. 3. Bi-serrated Ribs' Cutting Pattern

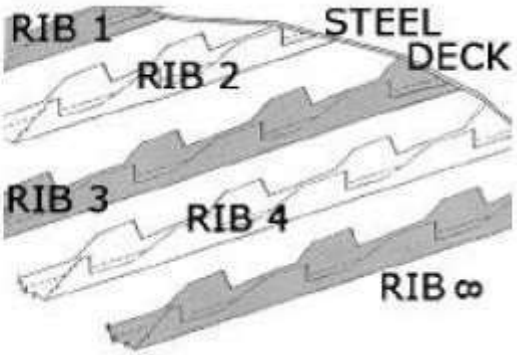
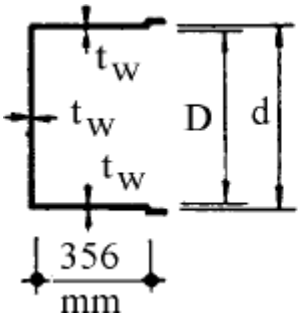


Fig. 4. Bi-serrated Ribs

4. **Slab Bridge for Mass Manufacturing Redding, California, USA**

The modular bridge system was developed by Advanced Bridge Systems Inc., Redding, California and was engineered by Jiri Strasky, Consulting Engineer, Mill Valley, California, for bridges with spans from 6-M to 30-M. The bridge deck is assembled of steel modules and a composite concrete deck slab (Fig. 9 & 10).



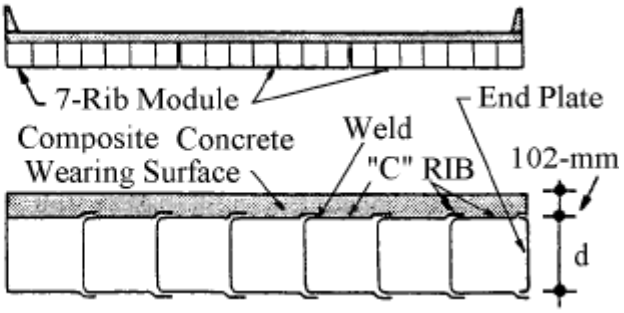


Fig. 5. "C" Rib Details

Fig. 6. Modular Slab Bridge with "C" Ribs

The depth *d* (Fig. 5) and number of ribs depend on the span length and loading. The number of modules depends on the total width of the bridge. The width of module shown is 2.540- M (Fig. 6). A seven-rib box module forms spans from 6-M to 17-M; and spans from 18-M to 23-M are formed by six-rib module. Shear studs guarantee a composite action of steel modules, with a concrete deck. Steel stiffeners and concrete diaphragms have been designed at the end of the deck. Static and fatigue loading tests were done at the University of California, San Diego. The "C" shaped orthotropic ribs are formed quickly from 8-mm coils of flat steel up to 1.829-M wide. These "C" shaped beams are then easily nested and welded together with just 5-mm fillet welds. All welds are quickly and easily made externally. Welding by a Patents Pending method(s) reduce up to 75% of the effort needed. All exterior welds mean faster production and simpler inspection. Reduced weight saves 10% minimum materials over other systems. Ideal for emergency bridge replacement, since orders can be shipped in seven days. The modules fit on standard trucks and are easily deliverable on all North American highways and most rural roads [8].

5. Temporary Bridge for Construction Detours Long Island, NY, USA

The bridge owner was New York State DOT; designed by Roman Wolchuk Consulting Engineers; built by general contractor Hendrikson Brothers, and the A-36 steel fabrication was by Quickway Metal Fabricators (Fig. 7). The length of the temporary two-span bridge with overhanging end supports was 64-M. The width was 12.192-M,

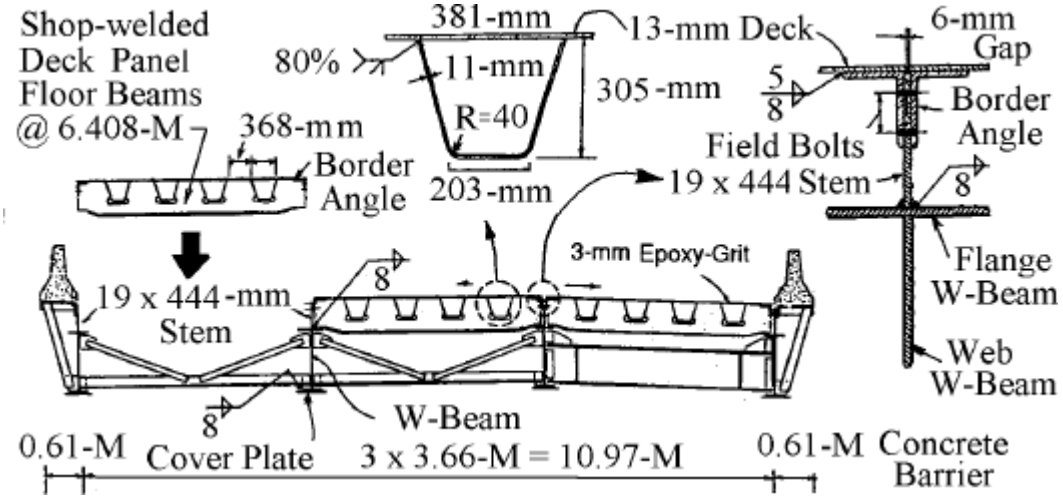


Fig. 7. Sagtikos Parkway "Temporary" Bridge Panelized Deck Details

including three 3-658-M. roadways and the precast concrete parapets mounted on the cantilevered brackets. The superstructure consists of a total of 15 deck units and 12 girder segments. This temporary bridge over the six-lane Long Island Expressway was put in place in just one hour and twenty minutes. The reasons for selection were: speed of erection and the eventual removal of this bridge; cost savings on traffic protection; potential reusability of the modular structure on similar projects in the future; and low depth and light weight of the structure. The superstructure is composed of two types of identical prefabricated components: the girder segments and the deck units. A 21.336-M long girder segment consists of a W-beam 914mm X 61kg section with a welded 25-mm X 406-mm bottom cover plate and a vertical 19-mm X 444-mm stem welded to the top flange. A typical deck unit is 12.80-M long and 3.658-M wide and consists of a 13-mm thick deck plate stiffened with four trapezoidal ribs and two 203-mm X 152-mm angles at the longitudinal edges. The ribs are supported on cross beams spaced at 6.40-M. and welded to the vertical legs of the border angles. The deck and girder components are

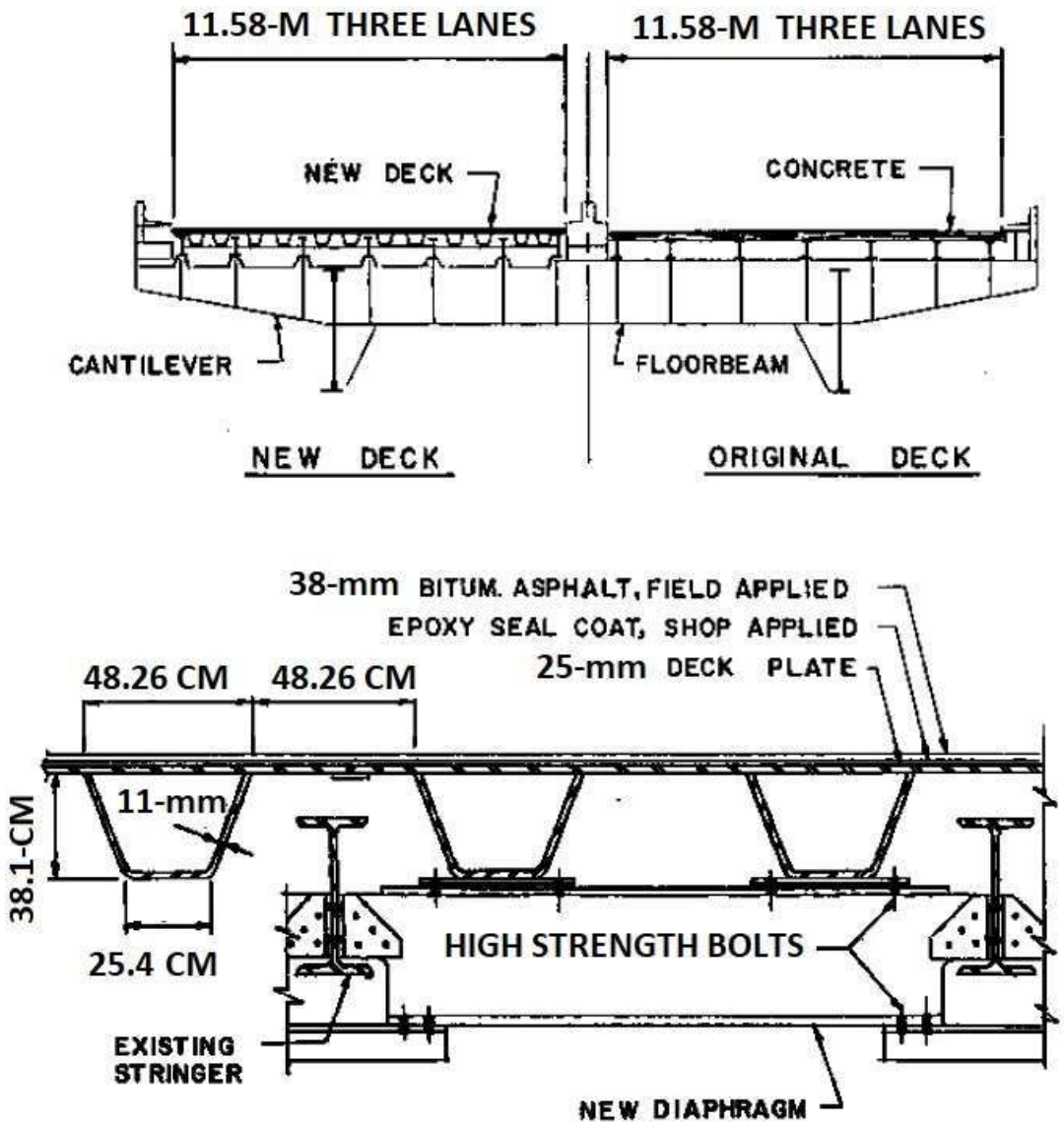


Fig. 8. Approach Spans for Throgs Neck Panelized Deck Details

joined in the field by bolting through modular holes spaced at 101-mm in the vertical legs of the border angles, and in the 19-mm thick girder stems. The 3-mm thick layer of shop-applied epoxy-grit surfacing required the use of countersunk bolts for the deck plate splices. The deck units are designed to carry the HS 20-44 wheel loads and to act as top flanges of the girders. Launching noses [4.877-M-long] were bolted to the front ends of the girders. A pivoted yoke was mounted at the rear to assure centric application of the launching push by one bulldozer [9].

6. The Bronx and Queens Approach Spans to Throgs Neck Bridge, NY, USA

The bridge owner was redecked with orthotropic deck panels and was designed by Ammann & Whitney Consulting Engineers (Fig. 8). The length of the thirty five approach spans varies between 42.7 meters and 57.9 meters [10]. This total deck area was about 45,700 square meters.

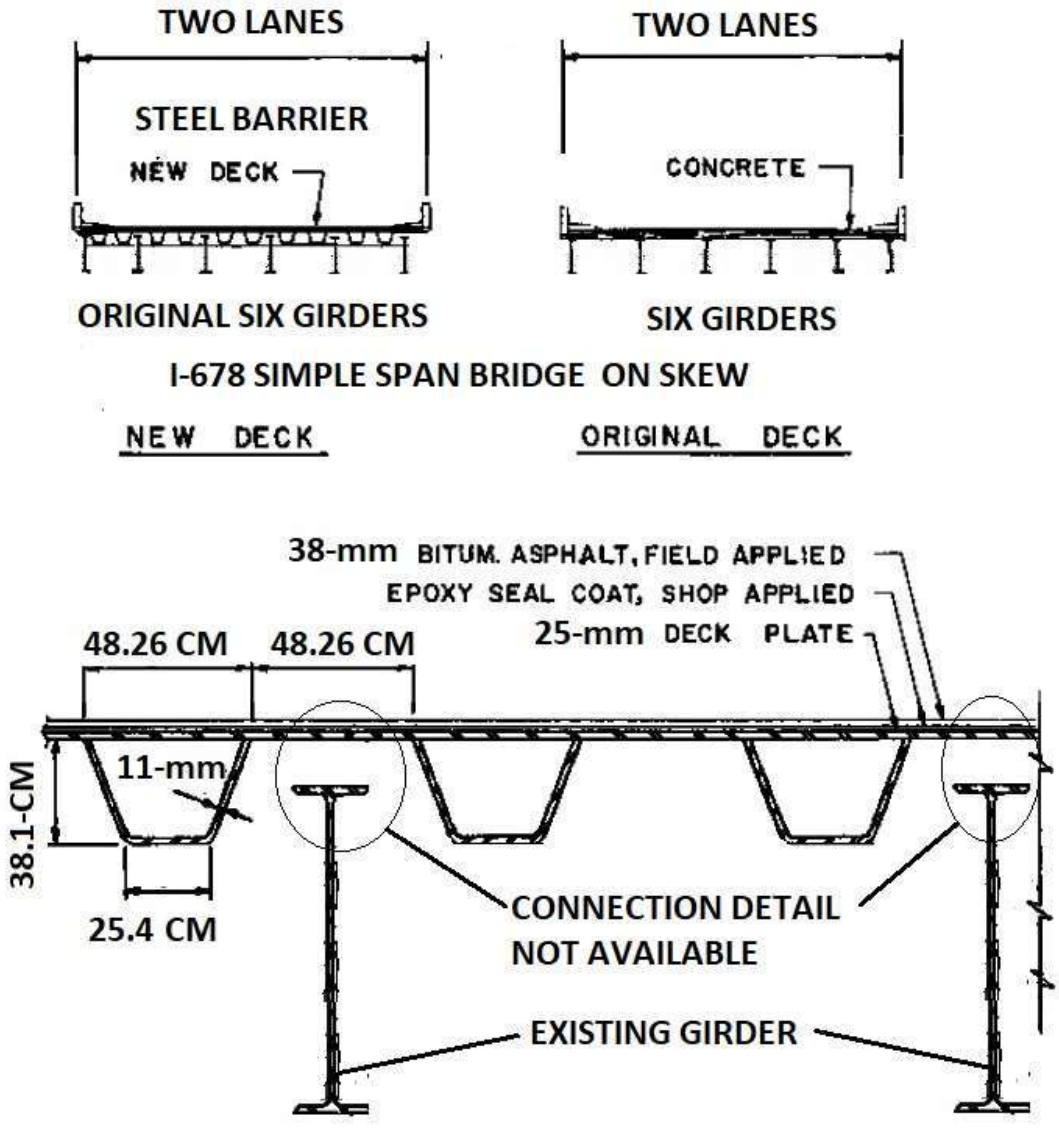


Fig. 9. Simple Span I-678 Panelized Deck Details

7. The I-678 Simple Span Bridge, NY, USA

This deck was used the I-678 expressway top ramp of a 3-level interchange for 21.3 bridge at the south end of Bronx Whitestone Bridge in Queens Borough in NYC. Replacement of the concrete deck occurred from 9pm Friday night to 6am Monday morning in 1984 [10]. See Figure 10.

8. Orthotropic Pedestrian Bridges

The orthotropic bridge across Santa Rosa Creek is an integral part of the Braille Trail, built to help the visually impaired and those in wheelchairs to enjoy Spring Lake Park of Santa Rosa, California (Fig.11 & 12). This bridge is capable of being periodically submerged by floodwater and aesthetically pleasing as part of a beautiful park. A timber bridge would float and poor soil support discouraged heavy concrete span; therefore, an orthotropic steel plate bridge was the economical choice. Engineers with the Sonoma County Water Agency selected an all welded steel orthotropic bridge to create a minimum weight structure with clean lines for a pleasant appearance and ease of maintenance. A dark brown acrylic epoxy compliments the banks of the channel near the bridge and contrasts well with the green park. A sand-wearing surface, bonded to the bridge's deck provides a non-skid surface for wheel chairs and pedestrians. The superstructure was completely prefabrication at a plant 240-km from the park. The 1.829-M wide by 17.678-M long span was trucked to the site and lifted into place by a crane in 1976 [11].

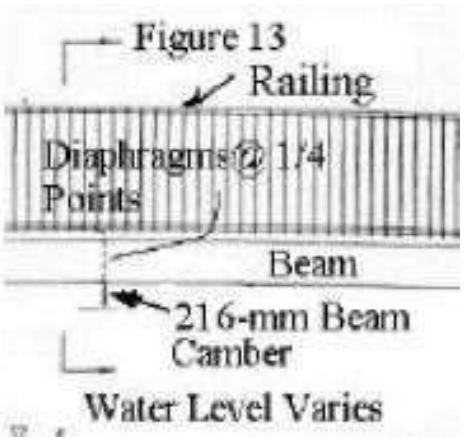


Fig. 11. Bridge Elevation

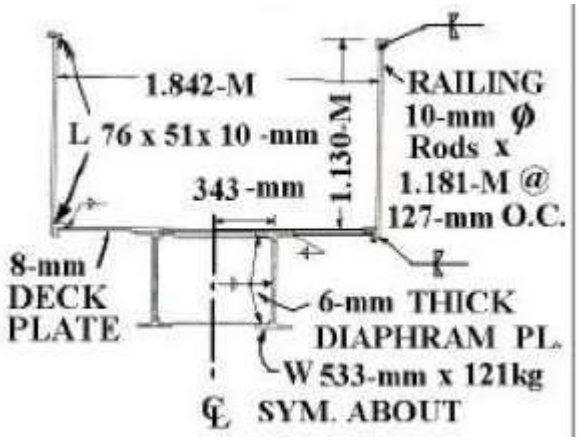


Fig. 13. Braille Trail Pedestrian Bridge

9. Conclusion – Why use short span bridges with orthotropic steel decks?

Short span orthotropic steel deck bridges have proven to be durable. Engineers are conservative and prefer to use a proven durable system. The other reasons used for selecting orthotropic steel deck bridges may be also applicable to short spans. Short span orthotropic steel deck bridges can be economically built in cold regions during any time of the year. Concrete must be at or above 4.44 degrees Celsius to properly cure. Although it is

physically possible to encapsulate plus heat the concrete construction process, it is usually more expensive. The orthotropic steel deck integrates the driving surface as part of the superstructure, and has the lowest total mass of any practicable system. Around the world floating bridges and movable bridges are built with orthotropic steel decks. A lower mass requires less energy to move the operable spans of a movable bridge. The mass of every movable span eventually reaches a practical limit due to the size of motor needed to move the bridge. Thus, even short movable spans built of heavier reinforced concrete are much more expensive for these bridge types. The lower mass results in lower seismic forces.

Hence, large orthotropic bridges in high seismic zones such as Japan and the western coast of North America. The examples in this paper provide a matrix of rib types, superstructure types and various bridge types. Other short span orthotropic bridges of North America include the North 680 to West Connector Test Bridge of Danville, California (1966); the Sacramento River at Colusa, California (1970); Walpole Island swing bridge of Ontario, Canada (1970); the Miller-Sweeney bascule bridge on Alameda Island, California (1974); and the city of Valdez, Alaska floating dock articulating transfer bridge (1981). The Maxi-Park Company of Vancouver manufactures orthotropic decks for parking garages floors. The Rabbit Creek, Willow Crest, Lathrop, and Egan Drive Orthotropic Pedestrian Bridges are located in urban areas of Alaska, and owned by Alaska DOT/PF.

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PROTECTION OF IMMOVABLE CULTURAL HERITAGE, CURRENT STATE AND CHALLENGES

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***Abstract**: The paper analyzes problems in protecting architectural heritage at the beginning of the 21st century with reference to the area of southern Serbia. In the paper, first of all, a comparative analysis of the legal regulations concerning the protection of architectural heritage in Serbia and the world is carried out to determine possible shortcomings or omissions of Serbian legislation in this field. After that, the treatment of cultural heritage in Serbia is presented based on positive and (mostly) negative cases. Then, the condition of the architectural heritage buildings is analyzed, representing the basis for later successful interventions. The causes that led to the poor condition of the buildings are listed, as well as the problems faced by the owners of the protected buildings. Finally, after reviewing the general situation, suggestions are made on how the problems can be alleviated and overcome over time.*

***Key words**: building heritage, preservation, (re)reintegration, protection*

1. Introduction

Architectural heritage represents one of the most significant categories of the country's spatial and cultural identity. It is a significant part of a country's identity and includes a wide range of buildings and structures created over the centuries, from ancient to today's achievements in architecture. By the term "built heritage," we mean historic cities, parts of settlements, old buildings, archaeological sites, and areas with historical, cultural, or artistic value. This heritage has different categories, such as cultural monuments, spatial cultural-historical entities, archaeological sites, and famous places. Those categories help better understand and classify various aspects of architectural heritage crucial for preserving and emphasizing cultural heritage.

The protection of architectural heritage is a very complex procedure with various challenges. Even during the construction process, each building is exposed to specific influences. They can later threaten the original appearance and characteristics of the building, both aesthetic and constructive. The causes of degradation are numerous and are

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reflected in the complete or partial demolition of the building. However, this also includes upgrades, extensions, and changes of purpose. They can be divided into long-term and immediate [1].

Apart from damage to existing buildings and attempts to preserve them in their original form, a big challenge today is the coexistence of historical buildings with newly built structures in the urban fabric of cities. The coexistence of old and new buildings has always been challenging in Serbia and the world. For coexistence to be possible, multidisciplinary and institutional cooperation is necessary, especially in the phase of previous research, preparation of strategic documents, and spatial planning to fulfill numerous conditions to ensure heritage preservation.

The paper analyzes exactly these problems when designing new buildings in protected areas. The problems observed during the application of legal regulations in practice were analyzed. Also, the bad condition of buildings with monumental properties was pointed out, with the reasons that led to them, to suggest how to improve the current state and overcome the problems that have arisen.

2. Comparative analysis of legal protection of cultural heritage in the world and Serbia

Laws protecting cultural heritage were created at different times in different regions worldwide. Even though in the ancient times of Ancient Greece and Rome, archaeological remains were recognized as heritage, and their protection was part of various edicts passed by rulers [2], the roots of modern heritage regulations can be dated back to the European 17th century.

The Royal Placard of 1666, issued by the governing council under the minority of King Karl KSI of Sweden, is often cited as the first example of succession law in modern times [2]. However, the 19th century saw the emergence of what we recognize as modern heritage legislation. The Law on the Protection of Ancient Monuments from 1882, which the British Empire passed, should be mentioned as one of the most influential. France was among the first countries to address this issue. 1837, it established the Higher Commission for the Inventory of Cultural Monuments, which created the first list of protected monuments in 1840 [3]. In the same year, the Commission hired the architect Viola le Dik to restore the great church in Vesle. France passed the first Law on protecting its cultural heritage in 1887. Since then, the legislation has been continuously changed and improved to achieve today's complete protection system, which serves as a model for many countries in the world [3].

International regulations related to heritage protection can also be traced back to this period. The Hague Convention of 1899 and the Athens Charter of 1907 on the Restoration of Historic Buildings are significant documents for protecting and preserving cultural heritage at the world level. Since then, until today, many charters, conventions, recommendations, resolutions, and declarations, which in various ways relate to the preservation of architectural heritage, have been signed and ratified.

During the 20th century, international legal regulations continued to develop and improve. However, in the middle of the 20th century, architectural and urban practice implemented the modernization of cities, which predicted the massive demolition of inherited buildings and entire blocks under the pretext of wear and tear of materials and the necessity of total reconstruction. Some countries, including France, previously established a protection system that managed to preserve a rich cultural heritage. Back in 1946, France prescribed a protection perimeter of a 500 m radius around all protected cultural monuments, and in 1962, the Law on Protected Sectors, which saved city neighborhoods and even entire cities from destruction [3].

The history of protection in Serbia does not lag behind documents of this type brought in Europe in terms of time or content. The first legal document in Serbia that protects heritage is the Decree on the Prohibition of Demolition of Old Towns and Their Ruins. It was adopted in 1844 and represents one of the first modern general monument-legal acts in 19th-century Europe. Two and a half decades later, in 1870, the first official list of protected monuments was compiled and published - 152 localities, including old towns, cemeteries, and old places of churches [4]. The first cultural monuments in Serbia were placed under state protection immediately after the Second World War when the field of cultural heritage protection was legally regulated and institutionalized. At that time, the Law on the Protection of Cultural Monuments and Natural Rarities from 1946 was in force, according to which all immovable and movable cultural-historical and other monuments are placed under the protection of the state, and the relevant protection institutes decide on this [5].

Today, heritage protection is a Constitutional right and obligation of every citizen of Serbia - Article 89 of the Constitution of the Republic of Serbia [6]. states, "Everyone is obliged to preserve natural rarities and scientific, cultural and historical heritage, as goods of general interest, in accordance with the law." The Republic of Serbia, autonomous provinces, and local self-government units are specially responsible for heritage preservation."

Apart from the Constitution, heritage preservation is woven into several laws, strategies, and planning documents. Laws dedicated to this area are the Law on Cultural Property [7] (1994), the Law on Cultural Heritage [8] (2021), and the Law on Culture [9] (2009). The Law on Planning and Construction [10] (2009) has a significant influence on the treatment of cultural heritage in modern conditions, while the National Architectural Strategy and the Sustainable Urban Development Strategy determine the strategic direction of action in order to develop and improve the quality of architecture and the built environment and preserve the architectural heritage.

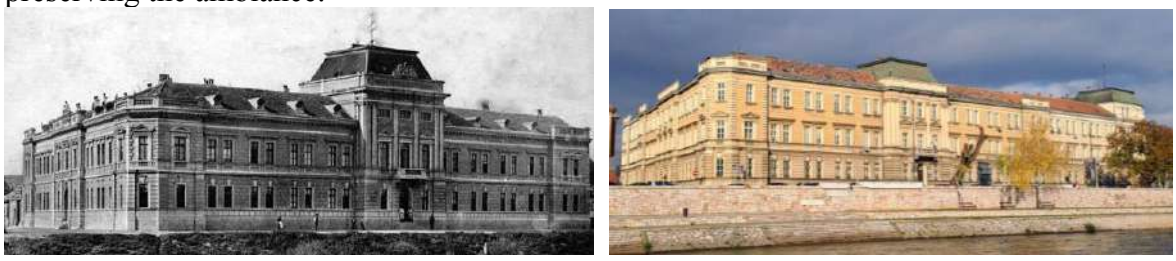
Regardless of everything, architectural heritage does not enjoy the treatment guaranteed by the law. Planning in the context of preservation and affirmation of heritage has not become a practice, and the mechanisms for practical action have not been sufficiently developed. The Law on Planning and Construction stipulates that planning, arrangement, and use of space are based on the principles of sustainable development, whereby non-renewable resources should be used rationally and sustainably by encouraging urban and rural renewal and protection. However, the practice is usually far from that. The problem of inadequate protection treatment in the last thirty years has been recognized through recent strategic documents - the Strategy of Sustainable Urban Development until 2030 [11] and the National Architectural Strategy 2023-2035—year [12], which increased the focus on architectural heritage. The strategy of sustainable urban development identifies that cultural heritage is not sufficiently represented in the practice of spatial and urban planning and that architectural heritage and other elements of the urban structure, in addition to immovable cultural assets, represent valuable factors of the character and identity of the settlement. However, there are often forms of construction that are not aligned with the principles of sustainable development and preservation of urban identity. The mismatch of dimensions of old and new buildings and the generally disturbed balance of design elements of architectural and urban solutions lead to the devastation of the urban identity of entire cities. This problem is discussed in depth in the National Architectural Strategy, which states that "sustainable use and restoration of architectural heritage is not only a cultural resource but also an economic potential that significantly affects spatial development and the preservation of spatial identity." The strategy proposes a review of all forms of heritage and mandates that "the modernization and improvement of such (recognized) entities must be based on the study of the

characteristics of each of the observed forms of heritage, with their full respect and the design of new contents that will not violate the basic values of the found heritage, i.e. spatial cultural -historical entities."

According to these documents, before defining the urban parameters, it is necessary to carry out extensive analyses and define the typological characteristics of the settlement or part of the settlement to reduce the generalization of the parameters.

3. Treatment of immovable cultural property

Generally, the treatment of protection and intervention on immovable cultural assets, their protected environment, and architectural heritage can differ. Numerous factors influence the different treatment: the degree of protection, the immediate environment and the context in which it is located, the time distance between the construction of the original buildings and the later additions, the scope of the implemented interventions, the currently current conservation practice, but also the styles and trends in architecture. In order to preserve the harmony of the environment, there is a tendency to harmonize with the architectural elements of the existing building (height and treatment of cornices, stucco, rhythm, and proportion of openings, respect for vertical/horizontal) or a complete contrast of the new building compared to the old one is sought. Regardless of the approach to creating an architectural solution, harmonizing with the dimensions and other architectural elements, designing in the context of the already built environment is a priority for preserving the ambiance.



a) b)

Figure 1. The Moravska Banovina building in Niš a) appearance of the building from 1886 [13] and b) the appearance of the building today, after additions from 1925 and 1935, photo by the author

When reintegrating new buildings into old city cores today, one should consider the context, the value of the preserved originals, the integrity of the design, and evidence of current style, forms, dimensions, and contrast [14]. In other words, the following should be taken into account:

- the state of the building and preservation of documentation about the building;
- how much the building was built in context, that is, what is its relationship with the architectural environment;
- the basic principles of respect for the authenticity of immovable cultural heritage, which imply that only the necessary minimum technical interventions are carried out on the original to return it to its original state, that is, the need for protection, preservation, and use;
- integrity - does the new building bear witness to the time in which it was created, and does it have value as a representative of the current style;

If possible, and if historical buildings are preserved at a high level, restoration and revitalization of existing buildings are carried out with minimal interventions.

Contemporary world practices for an extended period do not deal with the protection of individual cultural monuments. However, with broader strokes of spatial cultural-historical entities and cultural landscapes, and in the last ten years, the approach of looking at the entire city with its immediate surroundings, with a particular emphasis on aerial views, has been relevant. One of the approaches in the comprehensive overview of cities is

the approach of the historic urban landscape (Historic Urban Landsape), where it insists on multidisciplinarity, the cooperation of all institutions and professions, but also the permanent involvement of the local community and other interested parties in the process of planning, creating and using cities.



Figure 2. Strasbourg, a preserved city complex, photo by the authors

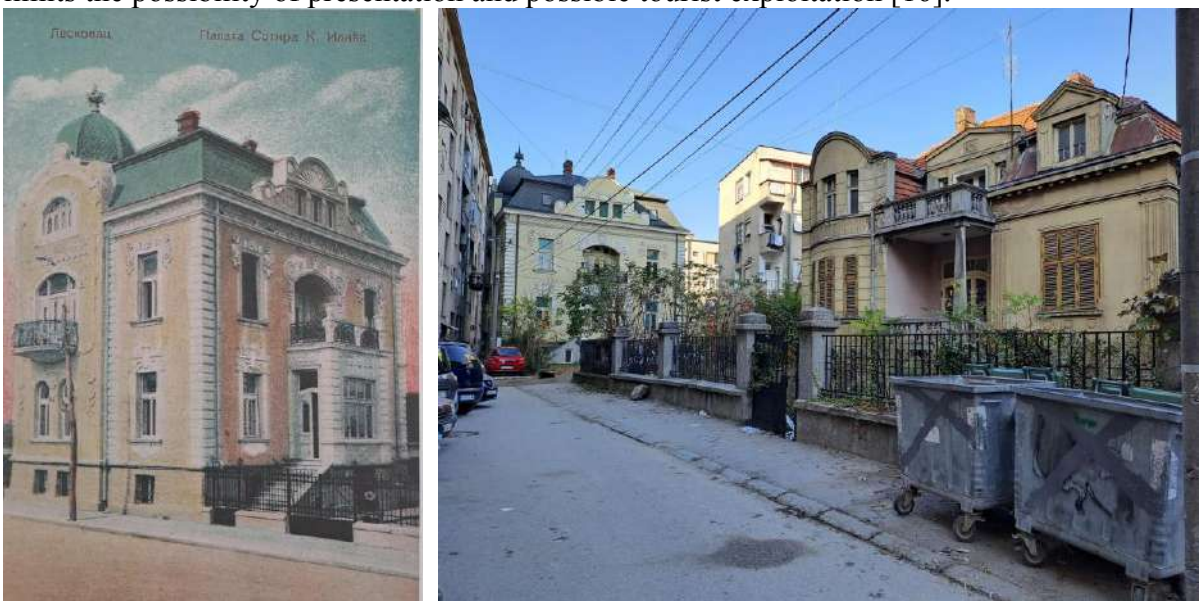
The picture in Serbia, specifically in its southern small towns, is unfortunately far from satisfactory in preserving architectural heritage. Historical buildings in cities, as well as the units themselves, rarely look representative. Mainly, there is an excessive number of subsequent, temporary, or permanent interventions on the buildings, which greatly disturb the ambiance of wholes or the general appearance of an individual building.



Figure 3. Interventions in "Kazandžijsko sokače", one of the central streets of Niš [15]

In the example of the "Kazandžijsko sokače" in Niš, one can see the type and degree of interventions that occur on historical buildings. Whether it is the demolition of parts or entire buildings, the extension of parts or the construction of complete buildings, or only partial interventions of a smaller scale (adding temporary or permanent canopies, external air conditioning units, advertising, removing plastic plaster, replacing original windows, etc. .), the

ambient whole is degraded. It loses its aesthetics and visual harmony, which further causes a negative impression on the building or the entire neighborhood, reduces its historical value, and limits the possibility of presentation and possible tourist exploitation [16].



a) b)

Figure 4. The urbanistic context of the immovable cultural property - the House of Sotir Ilić in Leskovac, a) after construction in 1911, b) appearance in 2022 [15]

The state of cultural assets and the challenges in preserving them can be illustrated by the house of Sotir Ilić, son of Kosta Ilić - *mumdžija*, a tallow candle maker, one of the first industrialists in Leskovac, a very respectable person of his time. The house was built in 1911, according to the project of the architect Svetozar Jovanovic the Elder. Immediately after the Second World War and the change of the political structure in the country, by the state-party political doctrine after the Second World War, new urban plans were drawn up in such a way that the newly built buildings deliberately hid and overshadowed and often replaced the preserved architectural heritage—pre-war period. As early as 1948, a new planning document for Leskovac was created with the idea of total reconstruction created by prominent architects of the time, Momčilo Belobrk and Ratomir Bogojević. Cultivating the idea of total reconstruction of the city center, shaped at the end of the fifth decade of the 20th century, Leskovac was transformed into a city with a completely mismatched urban-architectural structure, degraded and marginalized immovable cultural heritage. The following General Regulation Plan from 1956, drawn up by the bureau - Studio for Architecture and Urbanism Belgrade, architect Vido Vrbanić, continued the idea from the previous planning framework and envisaged the construction of large residential blocks in the central city core, completely ignoring the existing urban matrix. Based on this Plan, the construction of the Northern Block was started in 1957, which completely obscured the Ilić Palace and other representative buildings of Leskovac's pre-war industrialists.

4. Causes of the poor state of architectural heritage

If we look for the reasons that led to the poor state of architectural heritage in Serbia, we must recognize historical events and the political, social, and economic situation in which our country found itself during the nineties of the 20th century. The Law on Cultural Property, which is still valid today, was passed in 1994, at the time of the collapse of Yugoslavia and the survival of one of the most extended periods of hyperinflation in world economic history. Not only did this law not accept the novelties of the time in the

protection and definition of architectural heritage, its importance, and the role it has in the planning process, but the legal solutions set in this way led to a gradual weakening of the legal protection of immovable cultural assets and the collapse of the capacity of the entire protection service.

After the end of the isolation and the re-accession of Serbia to the international community, it was not possible to implement reforms in the field of cultural heritage in order to stop further devastation. Although international legal documents in this area were formally retroactively ratified, their implementation in the legal framework was absent. This situation caused the gradual degradation of protection, marginalization of the value of architectural heritage, insufficient number of professionals, and inadequate treatment through spatial planning documentation, leading to the massive loss of significant architectural heritage, ambient units, and the identity of entire cities.

Apart from this enormous problem, the low level of preservation of the construction stock should also be considered. Poor heritage status occurs due to the following causes:

- Unsettled property-legal relations - some buildings with monumental properties have had no heirs for a very long period, have a more significant number of them, or are in the process of restitution;
- multi-owner buildings - in the case of multi-owner buildings, there must be the interest of all owners to adequately take care of the buildings, which is often not the case;
- There is a lack of interest on the part of the owners of buildings with monumental values to adequately take care of their;
- lack of affirmative incentive measures for owners of buildings with monumental properties - there are no financial incentives in the sense of tax exemptions, subsidies for carrying out conservation-restoration works on immovable cultural heritage in private ownership, for procurement of materials, replacement of carpentry and the like;
- "complicated" procedure for carrying out work on such facilities (creating project documentation, obtaining conditions for undertaking technical protection measures, obtaining approval for project documentation...).
- difficulties in finding qualified artisans to perform craft works (absence of a database of qualified artisans who specialize in work on immovable cultural heritage);

high cost of works - replacing carpentry with wood while respecting the shape of the profile costs more than replacing it with ordinary PVC carpentry of standard dimensions (although in the long term it can pay off); the impossibility of installing thermal insulation on the outside of buildings, due to the existing plaster plastic, implies the use of more expensive thermal plaster systems [17]; restoration of plastered plastic - profiled wreaths, cast plastic, etc. increases the total cost of renovation of facade planes.

Due to those above "procedural" and financial difficulties, owners often take matters into their own hands and independently, without consulting the protection services or architectural experts, carry out interventions on their buildings. Such procedures result in permanent damage to the visual appearance of the building, as well as the entire ambient environment.



Figure 5. Non-professional interventions at the facility in Vlasotince [15]

All of this, unfortunately, very often results in a complete lack of interest in building maintenance, which ultimately leads to the collapse of individual parts or the entire building.



Figure 6. The same building in Nis a) in 2014 [18] and b) in 2023, photo by the authors

5. Suggestions for overcoming the problem

Based on all of the above, it can be concluded that most of the problems in the protection of the architectural heritage arise for two reasons: the inconsistency of the legal regulations concerning the protection of the heritage with the planning documents that are adopted, as well as the poor general condition of buildings with monumental properties and lack of interest or financial impossibility owners to take better care of such buildings. The problem can only be overcome by the joint action of all parties involved in this procedure, from owners of buildings, investors, designers of protection services, and planning institutions. The problem can be explained as follows:

- Respecting the adopted international charters, resolutions, and principles of integrative protection of cultural and natural resources;
- Anticipating mandatory analyses of the built stock, the typology of parts of the settlement, and especially the architectural heritage, i.e., urban areas with a greater concentration of architectural heritage;
- Defining the urban planning parameters after the analysis, defining the typological features of a part of the settlement;
- By working on improving the planning methodology with the mandatory preparation of studies and previous analyses;
- By implementing protection studies into planning documents and harmonizing planning documents with study documentation;

- Harmonization of planning documents with protection measures after the declaration of immovable cultural assets;
- By increasing investment in various forms of protection;
- By introducing incentive measures for building owners;

By creating a database with information of interest to facility owners.

6. Conclusion

We must be aware that cities develop and change to the needs of their inhabitants. With the increase in the number of people in cities, there is a need to increase the housing stock, so it is necessary to determine the locations we want to develop by planning and doing the necessary studies for them. Properties that enjoy prior protection can be, and most often are, positioned in desirable locations for new construction, so they should be respected.

As already determined by the strategic documents, it is necessary to carry out preliminary research on architectural heritage by studying the characteristics of each of the observed forms of heritage. Based on the previous analyses, with full respect for the preserved heritage, new contents can be planned to enable the development of locations but will not damage the fundamental values of the existing heritage, spatial cultural-historical entities, or environments. This approach is not only "protective" and "conservative" but also developmental in a long-term and sustainable sense. Numerous European studies [19] have confirmed that heritage is the essential resource that makes a rural area, city, or region attractive, which further attracts investments, leads to the development of the private sector, and the creation of new jobs. Preserving cultural heritage is part of the solution for reducing the impact of climate change, reducing the overheating of cities and the creation of thermal islands, as well as reducing pollution, primarily waste construction material. The fact that heritage preservation affects the quality of life of residents and makes a place suitable for living, working, and visiting is in favor of heritage preservation.

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STRENGTHENING OF OLD EXISTING REINFORCED CONCRETE STRUCTURES, THROUGH THE APPLICTION OF STEEL FRAMES

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Abstract: *In the report, the authors make an attempt to acquaint our and foreign colleagues with a modern method imposed in practice for strengthening existing reinforced concrete structures, through the application of steel frames, which has found its rightful place in the last decades in the Bulgarian construction practice.*

Key words*: existing reinforced concrete structure, steel frame, method for strengthening*

1. Introduction

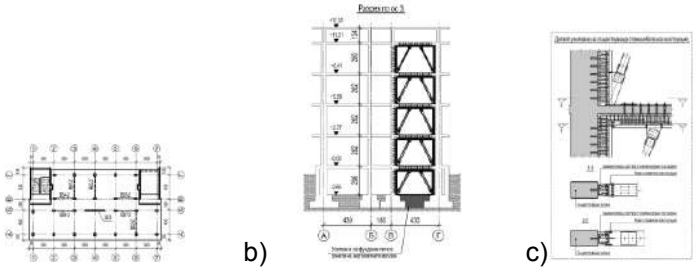
Reinforced concrete skeletal structures, infilled with masonry walls built in seismic zones in Bulgaria before 1960, have been designed to resist mainly to gravity loads and wind. At present, when such types of structures are subject to structural and architectural evaluation, in most cases they need to be strengthening according to the current seismic design code. One method of seismic strengthening of existing reinforced concrete frames with removed masonry walls uses additional earthquake resistant system such as various types of steel frame. One way of seismic strengthening of existing R.C. Frames is realized by improving strength, stiffness and ductility of r.c. elements, using various techniques such as added high-strength mortars, jacketing with steel elements or fibre plastics. An alternative approach for seismic strengthening of existing r.c. frames can by accomplished by adding an additional earthquake –resistent system, such as various typologies of eccentrically braced steel frames (EBSF)(Fig.19). Strengthening of reinforced concrete frames is usually performed for the purpose of increasing the strength and stiffness against earthquake induced forces. Steel frame strengthening system, in combination with moment resistant frame may cause an increase in the stiffness and strength of structure. Through the steel frame, earthquake induced loads transferring from top stories to the lower stories and foundations.

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2. Analysis on five practical Examples from the Bulgarian Cnstruction Practice

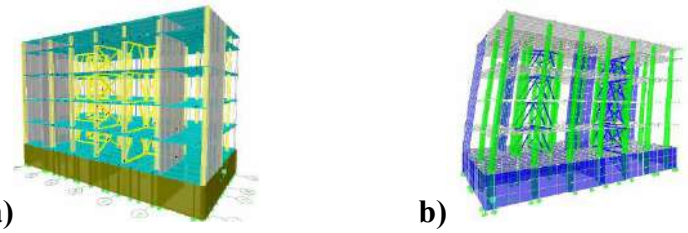
2.1. Case Study 1- Strengthening by embedding steel frames in reinforced concrete structures [1].

The ways to enhance reinforced concrete skeletal structures are commented on with the help of built -in steel frames in the light of modern Japanese practice and the application and in the specific Bulgarian conditions according to [2]. By investing steel frames in reinforced concrete frame structures, a significant increase is achieved both at their stiffness and their bearing capacity [3,4]. The main characteristic of the Japanese construction practice is the dispersion transmission of the efforts of the steel connection of the existing reinforced concrete structure, which is achieved through a specific detail, which is shown in Figure 1a, b,c. A key component of this detail is the curing-compensated grout that is placed between the steel connection and the existing reinforced concrete structure. This solution is additionally reinforced with a reinforcing skeleton, which has the function of limiting its transverse deformations, thus increasing the ductility of the contact joint between the old and the new element. The specific application of this method of seismic strengthening in Bulgarian conditions is given in Figure 2. The implementation is for the seismic strengthening of the building of ViK Pernik, damaged in the earthquake of 22.05.2012. Details regarding the calculation of this method of strengthening can be found in [3] and [4]. The construction of the building is provided in accordance with the norms [2] according to the computational model shown in (Fig.2).



a) b) c)

Fig. 1a, b, c. Reinforcement of the supporting structure of the Pernik Waterworks by means of embedded steel frames in an existing reinforced concrete structure.



a) **b)**

Fig. 2a,b. Seismic investigation of the Pernik Waterworks building in accordance with [2]- a) spatial model and b) deformed scheme from the seismic impact

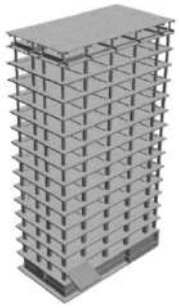
2.2. Case Study 2- Seismic retrofitting of high rise buildings with external steel structure[5]

An example of seismic strengthening of the fourteen-story reinforced concrete structure of the Faculty of Dental Medicine at the Medical University of Sofia was considered. The building was designed and completed in the late 1960s. As a result of the analysis and examination of the

supporting structure, it was established that the structural system of columns and flat beams has insufficient stiffness and bearing capacity for the seismic security of the building. The implemented solution for reinforcement with an external steel structure ensures the preservation of the architectural-functional solution and does not require stopping the operation of the building during the execution of the seismic reinforcement. The reason for the study is the serious damage that the building suffered during the earthquake of 22.05.2012, which was weak for the Sofia region (see [3] and [4]). Then the building registered significant damage in non-structural elements in the upper floors, and especially above the tenth. This necessitated a structural survey of the building in order to draw up a technical passport, according to which seismic strengthening of the supporting structure of the building is required. The aim of the current development is to present a new method for the Bulgarian construction practice for seismic strengthening of tall buildings, which has not been applied in our country, although it is widely used in countries with a degree of seismic risk comparable to that in Bulgaria. The external steel structure seismic strengthening method is very suitable for relatively tall buildings, for example 10-20 stories. It turned out that many such buildings in Bulgaria, built before 1987, have major seismic problems.

2.2.1. Seismic evaluation of the existing building

The building was designed and completed in the late 1960s. It is relatively high - fourteen floors, with a reinforced concrete supporting structure. As a result of the analyzes and examination of the supporting structure, it was established that the structural system, for example, of block A consists of columns and reverse flat beams plus an elevator core located at the end and a wall of the stairwell (Fig. 1 and Fig.2). The core and wall significantly worsen the seismic behavior of the building, making the structure very sensitive to torsion. Most likely, the building was investigated for an earthquake according to the Bulgarian seismic norms of 1964. For the purpose of developing a project for seismic reinforcement of the building, its existing supporting structure was seismically evaluated. The studies were carried out in accordance with the requirements of Eurocode 8, parts 1 and 3. Due to relatively insufficient data on the reinforcement and construction of the elements absorbing the seismic impact, a method with a behavior coefficient q = 1.5 was used in the seismic evaluation. as required in [2]. For this purpose, a spatial model of the building was developed and studied. The relative inter-story displacements along the short side of the building, in the damage containment check, are shown in Fig. 3, and the periods of free oscillations for the first few shapes are as follows: T1 = 2.62 sec. (predominantly torsional), T2 = 2.21 sec. (mostly translational in the transverse direction), T3 = 1.85 sec. (mostly twisting). Evaluation of the P-Δ effect shows that it is also a significant contributor. Additionally, the bearing capacity checks in a seismic design situation of a significant portion of the seismic reinforced concrete elements are not satisfied. The main conclusion from the analysis of the existing structure of the building is that its supporting structure has insufficient stiffness and load-bearing capacity for its seismic protection. In addition, the time when the building was built (the 1960s) gives reason to think that its structure has almost no ductility, because the Bulgarian seismic norms of 1964 did not require ensuring stable inelastic behavior of the buildings.



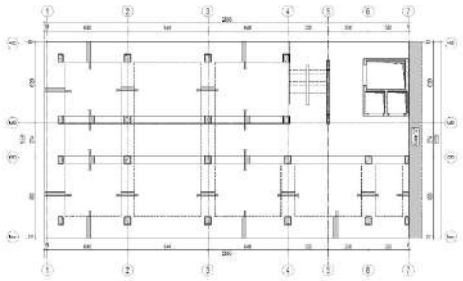


Fig.3. Spatial model of the existing structure **Fig. 4.** Formwork plan of type floor of block A

2.2.2. Choosing a strategy for the seismic strengthening of the building

A more detailed analysis of different seismic amplification strategies is given in [3] and [4]. The fact that the building has significant problems, both with stiffness and bearing capacity and ductility under seismic impact, necessitates the choice of a seismic strengthening strategy that excludes the strengthening of individual structural elements, i.e. the methods with externally glued polymer reinforcement, which are very modern in Bulgarian construction practice, are excluded as inappropriate. A strengthening option was sought to improve both the load-bearing capacity, the stiffness of the building, as well as its ductility as much as possible. Additional problems to find a solution are the requirements for a minimal suspension of the building from operation. I.e. in practice, the building must function almost continuously. Variants with only the installation of steel connections and reinforced concrete walls inside the building were investigated, but after their investigation they were rejected for the following reasons: - Impossibility to ensure the load-bearing capacity of the embedded new elements - steel connections or walls. The application of such methods necessitates stopping the functioning of the building for significant periods of time, which is not permissible in the specific case; - It is necessary to seriously strengthen the existing foundation, which is an expensive and labor-intensive operation. For these reasons, a new method for seismic strengthening was chosen for the Bulgarian construction practice: - application of an external steel structure. This method of seismic strengthening has been applied for years in countries with a seismic risk comparable to that of Bulgaria. The method is even considered in some seismic amplification standards [2]). The advantages of the method are as follows: - The suspension of the functioning of the building is for a relatively short period of time, according to the imposed requirements; - There is no significant change in the existing architectural and functional solution; - The external steel structure is performed on its own foundation, which is usually only connected to the existing one, and significantly heavy and expensive interventions in the existing foundation structure are eliminated. The significant intervention in the existing facade of the building is considered a disadvantage of the method in Bulgaria. In other countries, such as Japan, for example, this is not considered a disadvantage, but is even classified as an advantage, because it gives a safe, modern and technological look to the building. It should be borne in mind that in many cases the method of seismic strengthening with an external structure is only possible for high-rise buildings, if high-tech and expensive methods with vibration damping devices, seismic isolation, etc. are excluded.

2.2.3. Implementation of the selected amplification strategy

A project has been developed for the chosen amplification strategy. A scheme of the applied construction is given in Fig. 5. Due to the presence of an adjacent joint body (body A), it is possible to apply external steel construction only on three sides of the building. This also necessitated the installation of two additional reinforced concrete walls – one actually reinforcing

an existing wall from the stairwell, and the other located in an existing frame structure along the wall next to the joint. The availability of the foundation structure of the building, which is a rigid box type, leads to the minimization of interventions in the foundation because of the applied two additional walls. For the reinforced building, it is assumed that the existing reinforced concrete elements - columns, core, etc. are secondary elements, according to the classification of [2]. These secondary elements are strengthening locally. The significantly improved seismic behavior of the building is evident from the relative inter-story displacements presented in Fig. 6 and Fig. 7. Additionally, the periods of free oscillation for the first few shapes are as follows: T1 = 2.35 sec. (mostly translational in the transverse direction), T2 = 2.27 sec. (mostly translational in the longitudinal direction), T3 = 1.39 sec. (mostly twisting). The assessment of the P-Δ effect shows that it complies with the requirements of [2]. A major problem in the seismic design of buildings with an external steel structure is ensuring a connection between the existing old reinforced concrete building and the new structure, including the foundation. For this purpose, the recommendations given in [3] were used, i.e. the Japanese experience. A detail of the connection between the old and the new construction is given in Fig. 8a. The expected visual result of seismic amplification is shown in Fig. 8b. According to the authors, it is impressive and gives a new, modern look to the building.

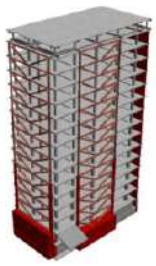
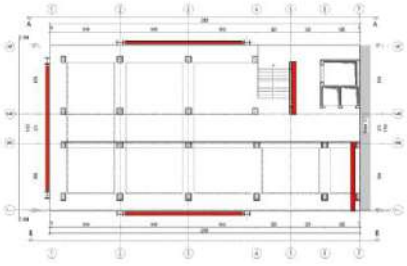
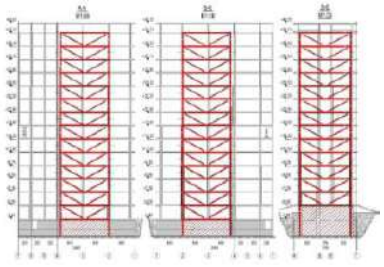
a)  b)  c) 

Fig.5a,b,c. -a) Model of the structure after seismic strengthening with external steel structure; -b) Formwork plan of a type floor after strengthening the structure; -c) Vertical views of the structure after reinforcement

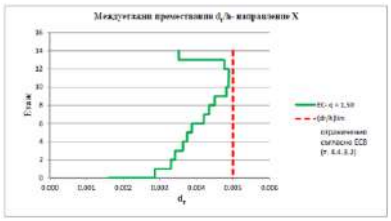


Fig. 6. Relative inter-floor displacements in the "X" direction (longitudinal)

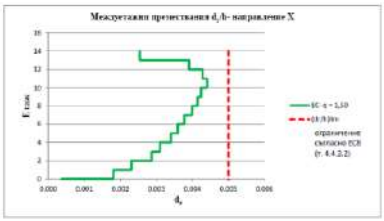
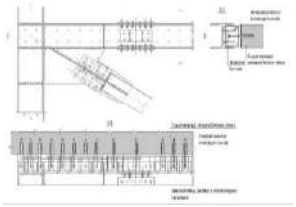


Fig. 7. Relative inter-floor displacements in the "X" direction (longitudinal), after reinforcement

a) 

b) 

Fig. 8a, b.-a) Details of connection between the existing and reinforcing structure; -b) Visualization of the design solution on the facade of the building

2.2.4. Conclusions

The conducted researches, briefly presented in the present work, enable the following main conclusions to be made: - The examined method for seismic strengthening with an external steel structure enables effective strengthening of relatively tall buildings in Bulgaria; - When implementing the method, special attention should be paid to the connection between the old and the new construction, including the level foundation; - Eurocode does not give sufficiently specific guidelines for the application of the method, but fully sufficient information on the problem can be found in the Japanese Standard for Seismic Reinforcement and in Japanese practice.

2.3. Case Study 3-Application of viscous wall dampers for the extension of „Radisson" hotel in Sofia[6]

2.3.1. General Data

This report presents the upgrade of a two-story existing reinforced concrete structure with seven floors. A steel structure with vertical connections in both directions is applied to the superstructure. The existing structure was completed in the 1960s and was designed for vertical loads only. This necessitates its seismic securing before the superstructure is implemented. In order to prevent contact between the superstructure and existing adjacent buildings in case of seismic impact, a passive seismic control system has been implemented. It consists of 21 vibration damping devices (Viscous Wall Damper), significantly reducing the absolute and inter-floor displacements, as well as the forces transmitted to the existing structure. Introduction a real project for upgrading an existing reinforced concrete building using a steel structure is presented. The investigated approach to improve the seismic behavior of the structure is the incorporation of a passive seismic system in the form of vibration damping devices installed in the steel superstructure. The main purpose of such type of systems is to reduce the seismic forces that are transmitted from the superstructure to the existing reinforced concrete building by increasing the damping. In addition, the heat-damping system is an effective method to significantly reduce the horizontal displacements of the superstructure under seismic conditions. Thus, hitting adjacent existing buildings during strong earthquakes can be avoided. The main purpose of the report is to evaluate the effectiveness of the system.

2.3.2. Description of the building

The presented report is focused on a real project that is in progress. The existing building is a two-story hotel with a reinforced concrete frame structure designed to withstand vertical loads only. The building was built in the 1960s and is being upgraded with a seven-story steel structure that has vertical seismic connections in its two main directions. Additionally, they are provided twenty one vibration damping devices to be installed in the metal structure. In order to be resistant to seismic impacts, the existing structure is strengthened by means of vertical steel ties that are embedded in the reinforced concrete frames. Reinforced concrete casings of the columns are implemented only to absorb gravity loads. The building's single foundations are united by a reinforced concrete foundation slab. It is also installed a system for monitoring the stresses in the main plane of the foundation construction and the settlements in the depth below it. A view of the building is presented in Fig. 9.



Fig. 9. View of the building under consideration

2.3.3. Reinforcement of the existing building

2.3.3.1. Installation of vertical steel frames

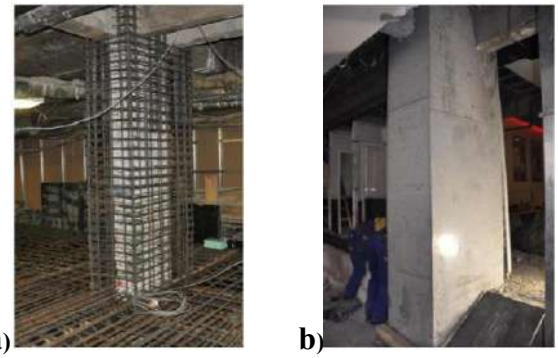
Part of the strengthening of the two-story part of the hotel was done by incorporating steel ties. They are embedded in the existing reinforced concrete frames of the building. These frames were originally designed to take vertical loads only. The computational procedures used in this strengthening approach are based on the Japanese Standard in accordance with Eurocode 8 and are detailed in [3] and [4]. Example embedded connections are presented in Fig. 10a, b.



Fig. 10a, b. Typical built-in vertical steel frame

2.3.3.2. Reinforcement of columns

The main columns are reinforced by reinforced concrete casings to be able to absorb the additional vertical load resulting from the superstructure. Some details of their reinforcement and a view of the reinforced concrete casings are presented in fig. 11a,b. Self-compacting concrete was used for the execution of the casings.



a) **b)**

Fig. 11a,b. Reinforcement by reinforced concrete casings

2.3.3.3. Strengthening of foundations and monitoring

The existing single foundations of the building are united by a common foundation plate, and a monitoring system has been implemented to monitor stresses and deformations. The main purpose of the monitoring system is to compare the calculated stresses in the base plane and the soil settlements at depth below it with the actual ones during and after the upgrade. It is important to check whether the redistribution of stresses in the main plane of the foundations after their union is as expected and to determine whether excessive settlements have been generated (Fig. 12)

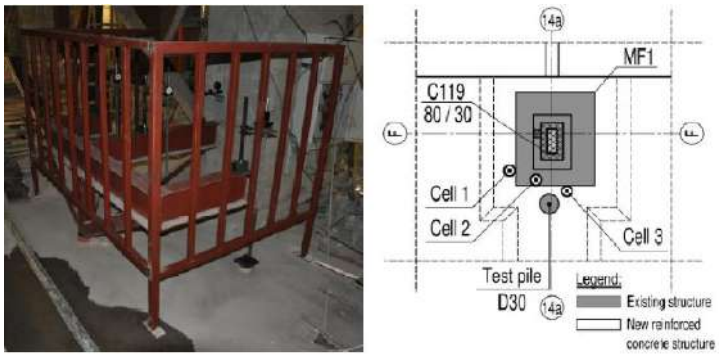


Fig. 12. System for monitoring the foundation structure

2.3.4. Analysis of the application of vibration damping devices

2.3.4.1. General description of vibration dampening devices

The existing part of the hotel is located next to other neighboring buildings (Fig. 9). In addition, the possible locations for embedding vertical steel connections were limited due to the functional requirements of the building. A passive seismic system has been added in some of the superstructure's vertical steel connections in the form of energy dissipating devices. The goal is to be achieved two main goals - reduction of seismic displacements, and hence avoidance of hitting neighboring buildings and reduction of the effect of seismic impact on the existing reinforced concrete frame construction. The vibration dampening devices are shown in fig. 13. A total of twenty-one vibration damping devices were installed - fourteen in the longitudinal (X-axis) and seven in the transverse (Y-axis) direction.

a)  b) 

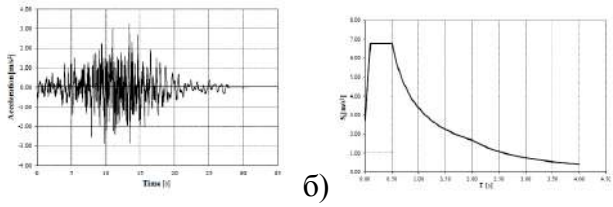
Fig. 13a,b. Testing of vibration dampening devices-a) and their storage on the construction site-b)

The results show that the structure can be significantly lightened by reducing some important parameters (such as acceleration, shear forces in foundations, etc.) through the use of

vibration damping devices. The main advantages of vibration damping devices are: • Well-known, reliable, simple and proven technology for all possible types of constructions all over the world; • The direction of applicability is horizontal; • Reduction of interference forces; • Economically expedient solution without the need for maintenance.

2.3.4.2. Computational model for analysis with accelerograms (timehistory)

In accordance with Eurocode 8 [2], the Bulgarian elastic response spectrum for the territory of the city of Sofia has been applied (Fig. 6). The analysis with accelerograms (time history) was performed using artificially generated ones that fit the response spectrum used in the calculations (Fig. 14a.b).



а) б)

Fig. 14a,b. Artificially generated accelerogram and elastic response spectrum for the city of Sofia (soil type C)

The dynamic analysis of the building was performed using the software product SAP2000, version 15.1.0. More information on the analysis approach is available in [2] and [3,4]. A spatial model was analyzed using the finite element method. The idealization of the building is done using elements from the SAP2000 library: frame elements (for columns and beams), link elements (for vibration damping devices) and shell elements (for walls and slabs) - Fig. 15.

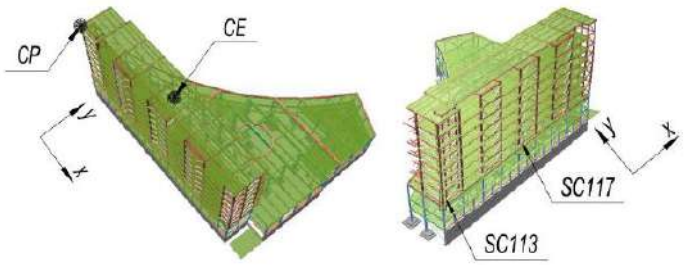


Fig. 15. Model showing points and elements in which the changes of displacements and efforts, respectively, are traced

2.3.4.3. Results of dynamic analysis with accelerograms (timehistory)

The analysis with accelerograms (time-history) was performed to establish the behavior of the structure under seismic conditions. Two models of the building were studied - with and without vibration damping devices. The first two oscillation modes for both models are shown in Fig. 16.

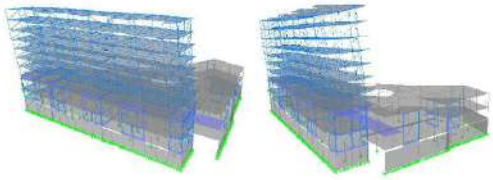


Fig. 16. First and second form of vibration (translational)

The periods of the first two forms of oscillation for both models are as follows: • without vibration damping devices - T1=1.075 s; T2=0.968 s; • with vibration damping devices – T1=1.406 s; T2=1.091 s. Some parameters of the seismic behavior of the construction are given quantitatively in Fig. 17.

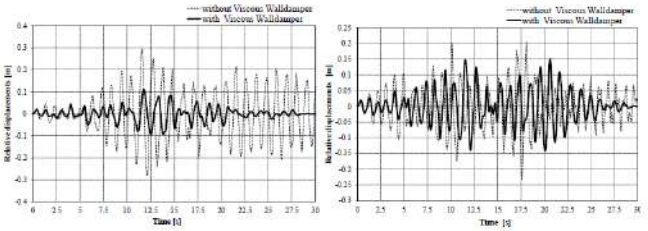


Fig. 17. Movements over time of point CE at the roof plate level in the two main directions X and Y

Some typical results are shown in Table 1. A comparison is also made between the results of a system with and without vibration dampers.

Table 1, Comparative analysis in direction x

Max. absolute value	Without Viscous Wall-damper	With Viscous Wall-damper	Difference[%]
Displacement of the point CP in the level of roof plate	293,13	110,75	37,8
Displacement of the point CE in the level of roof plate	273,02	104,83	38,4
Acceleration of the point CE[g]	3,0	0,63	20,9
Acceleration of the point CP[g]	2,72	0,58	21,3
Shear Force in Column SC113 [kN]	317	195	61,5
Shear Force in Column SC117 [kN]	292	201	68,8
Shear Force in fundamen[kN]	60969	29754	48,8
Overturning moment	1751673	1454059	83,0

6. Conclusions

The presented research allows the following conclusions to be made: • The passive seismic system with vibration damping devices is a very efficient method for improving the seismic behavior when upgrading existing reinforced concrete structures with metal structures; • Vibration damping devices significantly reduce inertial forces in both structures - the existing and the superstructure (by 35÷65%). In this way, the requirements for strengthening the existing part are eased; • Vibration damping devices significantly reduce displacements (by 30÷60%) compared to the same construction without vibration damping devices. This helps avoid hitting neighboring buildings in strong earthquakes.

2.4. Case Study 4- Project for the conversion of an electrical and radio repair shop into a shop and showroom for heating and solar installations, located in a housing cooperative in the city of Sofia, Oborishte district, Gen. Danail Nikolaev Blvd.; No. 8; bl. 3, (seismic insurance)[7].

2.4.1. Reinforcement of the construction of an apartment in a housing cooperative in Sofia

The site is a seven-story residential cooperative, the ground floor of which has been remodeled to create a store and a showroom for heating and solar installations. During this reconstruction, a total of four vertical brick walls were removed, playing the role of earthquake diaphragms, ensuring the earthquake resistance of the building. In place of the removed brick walls, this project provides a structural solution to seismically secure the building's structure to an earthquake of the VIIIth degree, by implementing a total of four steel reinforcing frames, which are installed in the places of the removed brick walls.

2.4.2. History of the construction of the residential building

According to the residents of the building, it was built in 1966 as a massive brick-reinforced concrete structure. It was executed monolithically and consists of reinforced concrete slabs, beams and columns, as well as brick walls made of solid single bricks with a thickness 12.5 and 25 cm. The perimeter basement (basement) walls are made of monolithic concrete with a thickness of 60 cm. The building is 7 floors with a floor height of 2.90 m. Ground floor with H - 2.9 m, and basement with H - 2.1 m ., with a flat roof - cold, with the second roof plate being inclined, with waterproofing. For this reason, the analyzes that will be prepared to investigate these structures for seismicity will not treat the bearing capacity of the block as a whole in an earthquake, but only the forces that will be obtained at the locations of the removed brick walls. From the on-site inspection of the existing building and from the analysis of the static calculations, the main conclusion emerges that the assumption of the vertical operational loads and their transfer to the foundations of the building is realized through a skeleton-beam system consisting of slabs, columns and beams. The spatial stability of the building for seismic impacts is ensured by the reinforced concrete skeleton and the brick walls made of solid bricks in the building. After a thorough study of the presented architectural projects, the following facts were established: An architectural project was prepared with the client Emil Jartov. It provides for the demolition of four existing brick walls with dimensions: 25/247 cm.; 25/314 cm.; 12.5/400 cm. and 25/240 cm., as well as the opening of a new door in an existing wall with dimensions 80/200 After the conversion of the ground floor, the floor slab will have to take the same calculated useful load as it was before the conversion, according to the "Regulations for Loads and Effects on Buildings and Facilities" 1989, and due to the non-change of the purpose of the site. The need to reinforce the openings in place of the removed internal brickwork is correctly reflected.

On the basis of the extensive study of the architectural documentation of the completed reconstruction, the following conclusions were drawn: The realization of openings in the brick walls involved in the absorption of horizontal impacts is allowed to be carried out only on the condition that the disturbed net stiffness and load-bearing capacity of the structure is restored the building being reduced by removing brick walls. This is accomplished by framing openings with steel frames firmly connected to the brick wall elements or columns and purlin around the opening. The calculation of each reinforcing frame must be preceded by the study of a well-prepared dynamic model of the building through software products: "SUPERSTAT" ; "SEIZM" ; "SAP2000" etc. The resulting seismic forces and displacements in this part of the building must be satisfied in terms of strength and deformation by the construction of the reinforcing frame. In the implementation of the architectural project, which proposes the removal of existing brick walls between reinforced concrete columns in order to unite adjacent spaces, it is necessary to implement steel reinforcing frames in the openings. They will guarantee the same seismic resistance of the building as it was before the proposed reconstruction. The earthquake building was studied with software products established in construction design, and the results will serve as a basis for the implementation of the strengthening frames. Static calculations of the

reinforced concrete structure of the VIIIth century earthquake building were performed with the program of "CESSI" - "SEIZM" (right and left sections). After that, static calculations were performed with the "SUPERSTAT" program of the steel frames and their corresponding sizing was done. In order to obtain the forces with which to calculate and size the steel reinforcing frames placed in the places of the removed brick washers, the building was calculated to the VIIIth degree for the Sofia city area according to several calculation models / described in Appendix I - "Seismic analysis" - see folder "KMD"/. On the basis of the comparative analysis, it was established that the proposed mechano-mathematical model of the supporting structure, adopted in the "SEIZM" program of "CESSI" gives reliable results that can be used for the study of the reinforcing steel frames. In this program (unlike many others, which work with a cantilever, with a stiffness equal to the sum of the stiffnesses of all the diaphragms), all the washers are worked at the same time, described with their geometrical characteristics, number of floors covered in them and modulus of elasticity and angular deformations. In addition, it also covers rotational effects, resp. twisting moments resulting from the mismatch of the center of mass with the center of rigidity of the building. The resulting limit forces at the locations of the removed brick walls: horizontal earthquake force, vertical earthquake moment and vertical earthquake load, are used to size the steel reinforcing frames. In the computer printout from the "SEIZM" program attached to the static calculations, all framed and unframed brick diaphragms with vertical reinforced concrete columns are listed, their geometric parameters, number of floors they reach, reduced modulus of elasticity and angular deformations, according to the joint behavior of the reinforced concrete columns and brickwork under horizontal earthquake loads. Also determined is the type, location and direction of placement of B.H.K. in the building. The location and characteristics of the applied weights along the height of the building have been determined, as well as the necessary coefficients set according to the 1987 NPSSZR. to determine the storey seismic forces. As a result of these calculations, the limit values of displacements and shear forces in B.H.K., on which we dimension the reinforcing steel frames, were obtained.

2.4.3. Static calculations of reinforcing steel frames

The reinforcing steel frames were tested at three simultaneous earthquake forces obtained at the locations of the removed brick walls of the VIII degree earthquake. For the calculation of the steel frames of the simultaneous action of horizontal earthquake force, vertical earthquake moment and vertical earthquake load, the program product "SUPERSTAT", developed on the basis of the finite element method, was used. Each one of the steel frames is descretized into a certain number of end elements with different geometric characteristics, on which, as well as on their nodes, the corresponding effects were placed - horizontal and vertical forces in the nodes and an extra-nodal uniformly distributed load. The resulting sectional forces: bending moments, transverse and normal forces are plotted in "M", "Q" and "N" diagrams, where the nodal equilibrium of the structure is checked and a final static check is made according to the laws of construction mechanics.

2.4.4. Dimensioning of structural elements of steel reinforcing frames

For the resulting shear forces, the steel frames are dimensioned and secured, according to "Standards for Design of Steel Structures" - NPSK 1987. of strength, resistance and stiffness. The anchor bolts at the steps are sized for shear, tension and bending forces resulting from the earthquake actions at the column toes. The anchor bolts that connect the steel structural elements of the reinforcing frames to the reinforced concrete parts of the building, columns and slabs/ are also sized for shear and bending forces resulting from the impact of the horizontal seismic force. The corresponding dimensions of the applied connecting means /welded and bolted joints/

according to NPSK 1987 have also been carried out. Inspections of the horizontal displacements of the frames due to the seismic load were carried out, and it was observed that they satisfy the requirements for inter-floor displacements according to the 1987 NPSSZR. The column base plates are dimensioned as an elastic plate loaded with the reactive stresses produced beneath it in the reinforced concrete slab by the vertical axial forces in the columns.

The current work project under the "Construction-constructive" part was developed on the basis of a written assignment by the investor, to carry out strengthening measures in the ground floor converted into a shop at "Gen. Danail Nikolaev Blvd. No. 8, block 3" with the aim of achieving of such seismic resistance of the building as it possessed before the reconstruction in the ground floor. For this purpose, a seismic analysis of the existing load-bearing structure of the residential building was carried out in order to find the corresponding floor seismic forces and displacements in those parts of the building where the vertical brick walls were removed. On the basis of the mechano-mathematical models known in practice for determining the calculated seismic load on the supporting structure of the building and with the help of the calculation program of CESSI - "SEIZM", the story seismic forces and displacements in those parts of the building in which they were removed were determined in the remodeling the brick walls, and in place of which we will design replacement steel frames. The calculations were carried out under the assumption that the brick walls and reinforced concrete columns work together on seismic effects, which is accounted for by the adoption of a bent module of the masonry. Analyzing the results of the research /in the prepared "Seismic analysis" - Appendix No. I/, it was found that the building according to the model of Prof. H. Nissimov, Prof. L. Tsenov, as well as the "Eurocode 8" designs, does not give satisfactory results in the studies of the VIII degree earthquake. For this reason, on the basis of the obtained seismic forces in the places of the removed brick walls, the relevant reinforcing steel frames were investigated, the constructive solution of which will be the subject of the written material presented below in the text.

2.4.5. Description of the structural solutions of the steel reinforcing frames. Load and impacts. Static calculations.

2.4.5.1. Steel frame "P1"

The frame is located in letter axis "B" and numerical axes "3" and "4". The frame is completed in place of a removed internal partition wall, adopted in the calculations for the earthquake washer "Ш1 in the right section.". The thickness of the brick wall was 25 cm. In its place, a portal opening was left, formed by two reinforced concrete columns "K2" and "K5" along axes "3" and "4" and a reinforced concrete slab along axis "B". The steel frame is located under a reinforced concrete slab. The presence of two reinforced concrete columns and a slab determined the nominal axial distance between the columns to be 2468 mm. /trait No. 1/. The frame is a single-ship, full-wall, single-story, double-jointed structure with external axial dimensions: 2468x2780 mm. It is made of (GVP) HEB 200 steel profiles with full-wall I-cross sections. The steel frame is U-shaped, composed of two vertical columns "K1 and K2" and three horizontal beams "G1, G2, G3". Vertical columns and beams are connected by welding joints. At their lower end, the columns end with support plates with dimensions: - 30x400x400 mm. connected to them by welding seams and two ribs each – 14x250x400 mm., placed on the sides of the column and welded to it and the slab. The anchoring of each of the columns to the reinforced reinforced concrete slab is carried out by means of seven "HILTI" type anchor bolts with a diameter of 20 mm. and length 190 mm. On the sides, the steel columns are connected to the reinforced concrete columns by means of steel plates, anchored in turn with "HИLTI" M 16 adhesive anchors to them. Steel plates "FP1" with dimensions 20x200x220 mm. fix during installation, the exact distance between the outer sides of the columns, which in this case is 2468 mm. This distance is fundamental to the geometry of the frame and must be respected when fixing the location of the columns. This means that the installation of these fixing plates must be

checked with great responsibility and in the presence of the designer. Two holes with φ18 mm are made in the plates. with a countersink large enough to allow the stud nut to be sunk into it. This allows after the adhesive anchor is installed, the protruding part of the stud can be cut off and the nut left submerged, then welded into the hole. At their upper end, the columns are connected to the steel beam "G1" by means of nodal plates anchored to the ceiling, which unite the two elements on one side. The planks are 20 mm thick, 400 mm long. and width – 220 mm. and are anchored to the ceiling slab. In addition, the columns and beam "G1" are united by nodal plates, reinforcing the frame node. Beams "G2 and G3" are made of HEB200. In the places where the beams are connected to the columns, local ribbing of the stem of the columns with the dimensions of the plates: - 10x170x85 mm is carried out. The connection is realized by welding joints. The connection of the steel bar "G1" with the reinforced concrete slab is carried out by anchored with Hilti to the reinforced concrete slab - 20x200x220.

2.4.5.2. Steel Frame for strengthening of building apartment

The photos focuses on the strengthening of non –ductile RC building, designed in 1966. The case study present an example of a seven –stories buildings with removed masonry walls, replaced by steel frame. The –infill r.c. frame is designed in accordance with regulations of 1964,s. The building is strengthening with steel frame in the first floor to comply with the current national seismic code. In fig. 18 are shown: Construction detail of the steel frame, between column and beam of frame. Steel frame for strengthening of non –ductile RC frame after removed the masonry infill. Two one span steel frames in the first floor of the building during strengthening.

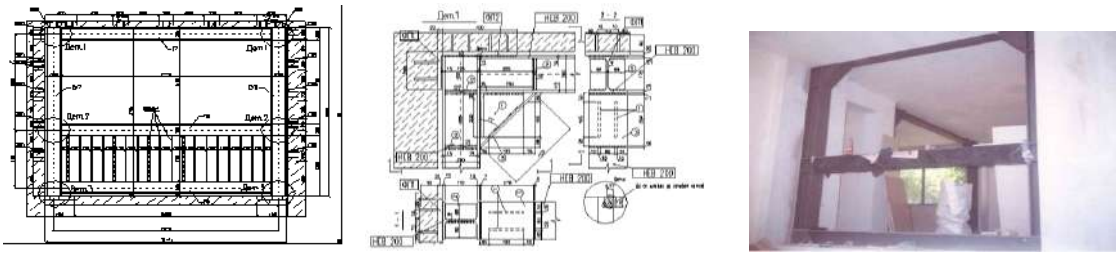


Fig.18. Two one span steel frames in the first floor of the building during strengthening, Designers: D.N.P and M.S.P. 2006

2.5. Case Study 5- Redevelopment of apartment No. 2 in two stores for industrial goods in a housing cooperative, located in quarter 439, m. Centera, property No. 11 according to the plan of the city of Sofia, district "Sredets" SO, str. Lyuben Karavelov" No. 24, (seismic insurance)[8]

2.5.1. General data

The current work project under the "Construction-constructive" part was developed on the basis of a written assignment by the investor, to carry out strengthening measures in the residential apartment converted into shops at 24 "Lyuben Karavelov" street in order to achieve such seismic resistance of the building , as she owned it before the remodeling on the ground floor. For this purpose, a seismic analysis of the existing load-bearing structure of the residential building was carried out in order to find the corresponding floor seismic forces and displacements in those parts of the building where the vertical brick walls were removed. On the basis of the mechano-mathematical models known in practice for determining the calculated

seismic load on the supporting structure of the building and with the help of the calculation program of CESSI - "SEIZM", the storey seismic forces and displacements in those parts of the building, in which they are removed at rebuilding the brick walls, and in place of which we will design replacement steel frames. Calculations were carried out under the assumption that the brick walls and reinforced concrete columns work together on seismic actions, which is accounted for by adopting a translated modulus of the masonry. Analyzing the results of the research /in the prepared "Seismic analysis" - Appendix No. I/, it was found that the building according to the model of Prof. H. Nissimov, Prof. L. Tsenov, as well as the "Eurocode 8" designs, gives satisfactory results at the studies of the VIII degree earthquake. On the basis of the obtained seismic forces in the locations of the removed brick walls, the relevant reinforcing steel frames were investigated, the description of which will be the subject of the written material presented below in the text. One way of seismic strengthening of existing R.C. Frames is realized by improving strength, stiffness and ductility of r.c. elements, using various techniques such as added high-strength mortars, jacketing with steel elements or fibre plastics. An alternative approach for seismic strengthening of existing r.c. frames can by accomplished by adding an additional earthquake –resistent system, such as various typologies of eccentrically braced steel frames (EBSF)(Fig.19).

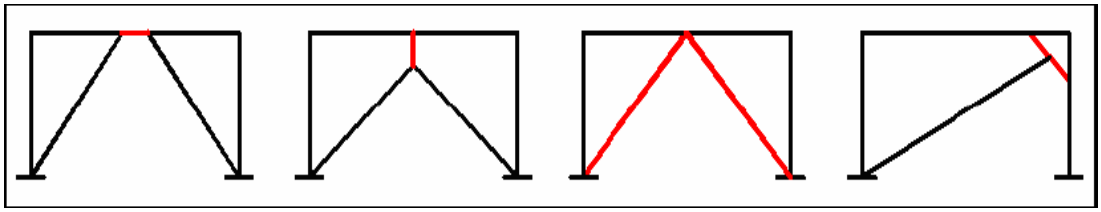
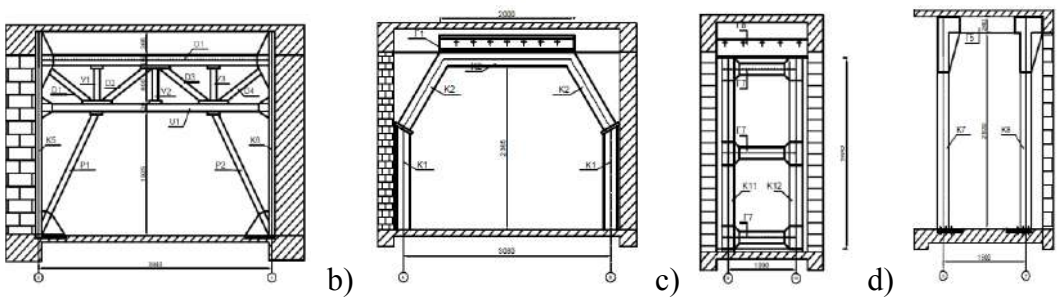


Fig.19. Different typologies of Dissipative Frame: a) inverted V eccentrically braced frames, b) inverted Y eccentrically braced frames, c) concentrically braced frames using buckling – restrained braces, d) knee-braced frames

The new structural system can be easy designed as a dissipative system. In order to limit the damage in the existing r.c. structure and to concentrate plastic in the dissipative steel bracing systems, the latter should possess a higher stiffness than the former. This objective is accomplished naturally in case of braced steel frames. Reinforced concrete skeletal structures, infilled with masonry walls built in seismic zones in Bulgaria before 1960, have been designed to resist mainly to gravity loads and wind. At present, when such types of structures are subject to structural and architectural evaluation, in most cases they need to be strengthening according to the current seismic design code. One method of seismic strengthening of existing reinforced concrete frames with removed masonry walls uses additional earthquake resistant system such as various types of steel frame(Fig.20).



a) b) c) d)

Fig.20a,b,c,d. Different typologies of Steel Frame uses as additional earthquake resistant system: a) Truss steel frame, b) Polygonal steel frame, c) Steel frame consists from two closed counter, d) П-shape steel frame

The new structural system can be easy inserting inside the R/C Frame. This system have several advantages: Steel members can be connected directly to the existing R/C elements; Connection between the new steel frame and existing R/C frame can realized over a longer interface, leading to smaller concentration of forces due to earth-quake loading.

2.5.2. Application to an building apartment

1.5.2.1. Steel Frame Strengthening Method

This retrofit method has been used in a total of 10 projects consisting of nine apartment in residential public buildings and one railway station.



Fig. 21. Application to an building apartment



Fig.22. Application to an building apartment

They was developed in Japan, USA, New Zealand. Internal or external steel Frames are installed to the existing R/C structural frame with the aim to enhance its seismic performance**.**



Fig.23. Steel Frame in the first floor of the building

An alternative solution consists in inserting a full steel frame inside the r.c. frame(Fig.23). This system has several advantages. The choice of frame typologies is larger in this case. Connection between the new steel frame and existing r.c. frame can realized over a longer interface, leading to smaller concentration forces due to earth-quake loading. The configuration of the opening, where we must install the new steel frame and " Proposal for the strengthening" In this seismic retrofit method for strengthening of existing reinforced concrete frames, steel frame are imply to the existing R/C columns and girders of the retrofit structures. This retrofit method has been used in a total of 10 projects consisting of nine apartment in residential public buildings and one railway station. The steel frames are usually placed in vertical aligned spans. This system allows to obtain a great increase of stiffness with minimal added weight. It is very effective for existing structure for which the poor lateral stiffness is the main problem. The steel

frames increase the lateral stiffness of the concrete frame , thus increasing the natural frequency and also usually decreasing the lateral drift. However increase in the stiffness may attract a larger inertia force due to earthquake. Then arise the question: *How to transfer the internal forces in the mixed structures?* Only through the new steel frame or through the composite steel-concrete construction. A special configuration is possible when the new steel frame are inserting inside the R/C Frames(Fig.24).

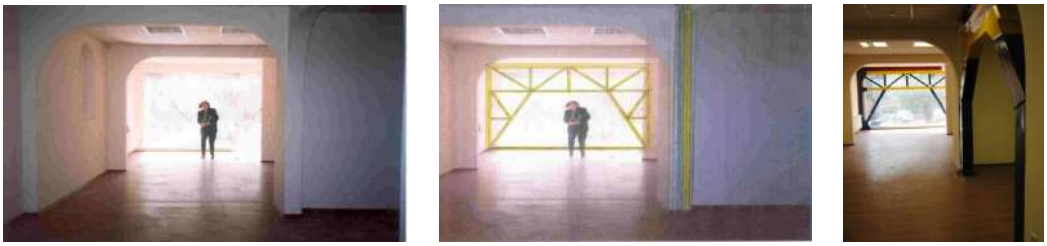


Fig.24. Example of application of steel truss frame and polygonal steel frame which are inserting inside to the R.C. Frame. Front of view elevation with Steel Truss Frames and Polygonal steel frame

2.5.3. Seismic analysis of the existing structure

Distribution of horizontal seismic load between masonry walls (linear strut model). Floor horizontal seismic forces from acting on the building shall be allocated to the respective direction-along X and Y direction proportional to the stiffness of individual vertical diaphragms in the present direction. In order to obtain maximum horizontal seismic forces in places removed masonry walls, which must be substitute with steel frames, stiffness of individual diaphragms was adopted without taking into account the presence of openings for windows. and doors directions. The idea of vertical polygonal steel frame between r. c. columns and girders came up as a suitable solution. For this design solution, detailed static and seismic analysis was carried out to verify the stability of the structure and the bearing capacities of all cross sections and the reconstructed structure as a whole as well as the limit state of usability. View of steel frame of the polygonal steel frame is shown in Fig.25.

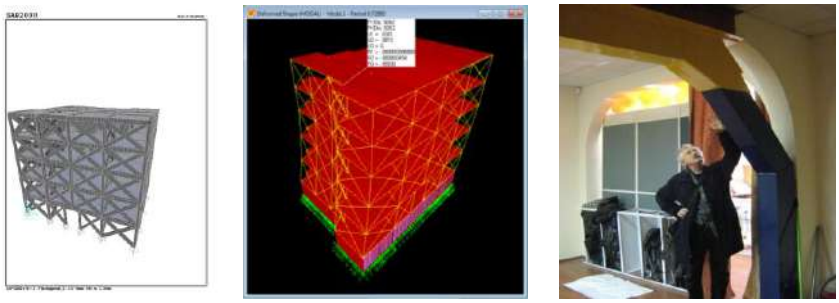


Fig.25. Pushover Analysis of Steel Frame used for strengthening of Building

2.5.4.Conclusion

The application of linear models of the infill is useful in everyday design, but not when assessment of existing buildings is performed. Such simplified models should be used mostly for preliminary analysis and modal dynamic analysis. As is well known , masonry infill bring to structures extremely inelastic behavior, due to the brittle nature of the composing materials. The proper assessment of the behavior of infilled R/C frames is may possible when nonlinear modeling and analysis is applied. Application of the SEISMOSTRUCT multi - strut nonlinear

infill model in combination with programs "Adaptive Nonlinear Pushover Procedure" is presented in first two figures. A part of the mentioned above structure(front facade of the building) is modeled and analyzed with SEISMOSTRUCT software package. The main purpose was assessment of the effect of 1-st storey masonry infill removal, which has happened during a reconstruction. The results shown , that if the infill was not removed the ultimate total base shear force would have been 1150 kN, but after the removal it is only 900 kN, or more than 20 % decrease in the load carrying capacity of the building even though all elements were left intact. (The nonlinear analysis was provided from Ci.Eng. V.Petkov, one of the author of the paper in the frame of his Ph study).

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MASS TIMBER IN TALL BUILDINGS AND IN OLD STRUCTURES

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Abstract: *Information about the development of the tallest buildings made of mass timber. The height of record-tall timber buildings has tripled in recent decades. The tall buildings can be: a) all-timber, b) timber-concrete hybrids, c) steel-timber hybrids, d) timber-concrete-steel hybrids. Load-bearing elements: columns, beams and walls can be made of grown wood or of certain types of composite timber. We distinguish the following types of structural composite lumber for load-bearing purposes (SLS): a) CLT, b) DLT, c) NLT, d) MPP, e) GLULAM, f) LVL, g) PSL, h) LSL. The contribution represents a part of the power-point lecture, which is used in the pedagogical process by Pavla Balážová , for Slovak and foreign students (e.g. from Albania, Belarus, Brazil, Egypt, Georgia, Greece, Hungary, Iran, Kazakhstan, Kenya, Kuwait, Lithuania, Mali, Nigeria, Palestine, Poland, Russia, Serbia, Spain, Syria, Great Britain and Ukraine). The CTBUH materials and diagrams from Damian Koh's website are used in the contribution. Old timber structures.*

Key words*: tall buildings, mass timber, timber-concrete-steel hybrids, education of structural engineers, old timber structures*

1. Introduction

Today there are 139 mass timber buildings around the world of eight stories or higher, either complete, under construction, or proposed. CTBUH reviews a decade of progress in the worldwide mass-timber movement, with analysis by building height, location, function, structural material, and several other parameters.
In the CTBUH study performed till February 2022 it is written [1]:

- there are 66 completed mass timber buildings globally, 8 stories and higher (54 in Europe – 71 %; 6 in North America; 5 in Australia and 1 in Asia);
- 64 % of all mass timber buildings are residential;
- the key advantages of mass timber: a) they are strong, the mass timber can be used to supplement or replace steel or concrete as a structural solution; b) they are sustainable. The forests are sustainably managed so we can rely on timber supply

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for generations; c) they are safe, the mass timber building systems are proven to meet or exceed code requirement;

- it took only 77 minutes for Austrian forests to grow back the timber used in HOHO building, Vienna. HoHo is 84 m high building with 24 floors and structural type timber-concrete hybrid;
- it took 49 weeks to complete Stadhaus (29 m high, 9 floors, all-timber), London, UK, in 2009 in mass timber, compared to 72 weeks for equivalent concrete building;
- 900 000 kilograms of CO_2 were saved through building Origine, Québec City, Canda, in timber rather than convential materials. Origine consists of 12-floor solid wood structure on a concrete podium and measure 40,9 m in height (Figure 12).



Fig. 1. Some species of mass timber used for beams, columns and walls in timber mass buildings (Fig.3).

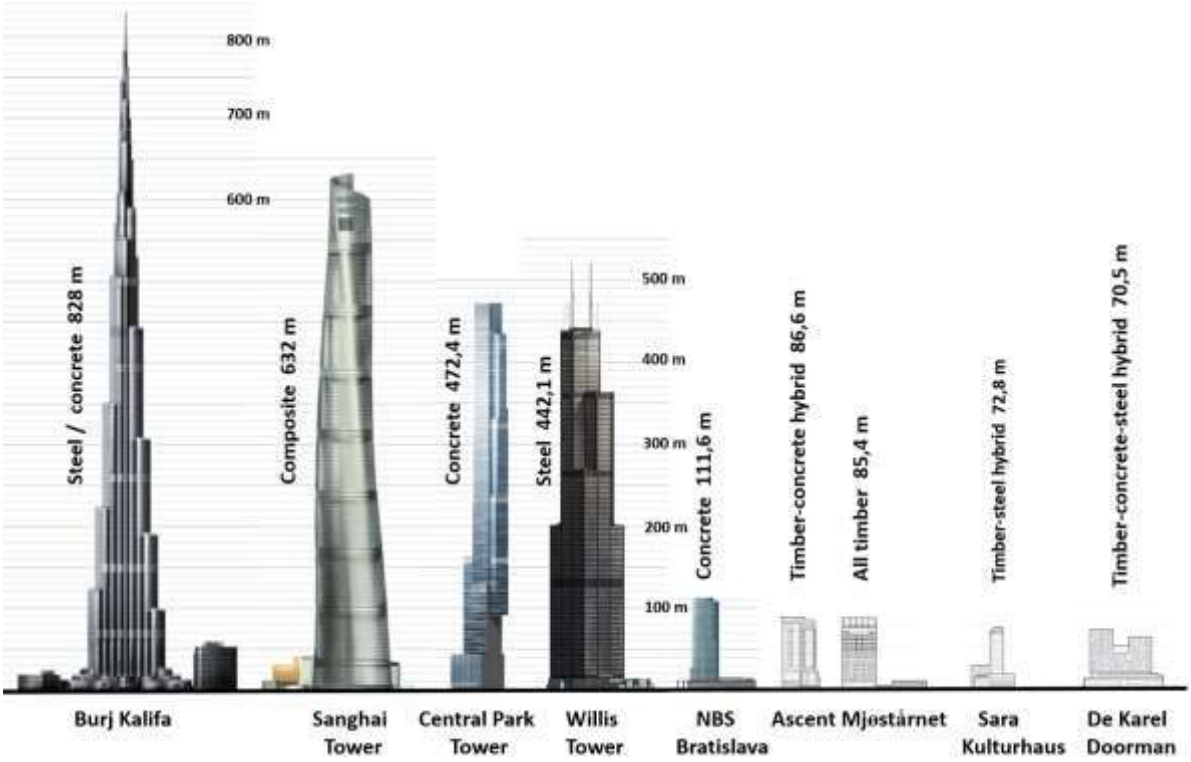


Fig. 2. Comparison of the tallest buildings in the world from different structural materials (state in February 2022) [1], [2].

2. Application of mass timber in tall buildings

According Total number of mass world-wide buildings, 8 stories and higher, grouped by project stage (completed; under construction; proposed), as February 2022 is 139. Completed total 66 buildings (47 %); under construction total 18 buildings (13 %); proposed total 55 buildings (40 %). Residential total 54 buildings (64 %); office total 16 buildings (19 %); mixed-used total 12 buildings (14 %). Europe total 60 buildings (71 %); North America total 18 buildings (18 %); Australia total 8 buildings (10 %); Asia (Singapore) total 1 building (1 %).

Table 1. The tallest buildings in the world from different structural materials (state in February 2022) [1].

Name	City, State	Completion	Height [m]	Floors	Material	Function
Burj Khalifa	Dubai, United Arab Emirates	2010	828	163	steel/concrete	office, hotel, residential
Shanghai Tower	Shanghai, China	2015	632	128	composite	hotel, office
Central Park Tower	New York City, USA	2020	472,4	98	concrete	residential
Willis Tower	Chicago, USA	1974	442,1	108	steel	office
Ascent	Milwaukee, USA	2022	86,6	25	timber-concrete hybrid	residential
Mjøstårnet	Brumunddal, Norway	2019	85,4	18	all-timber	mixed-used
Sara Kulturhaus	Skellefteå, Sweden	2021	72,8	19	steel-timber hybrid	mixed-used
De Karel Doorman	Rotterdam, Netherlands	2012	70,5	22	timber-concrete-steel hybrid	mixed-used
300 Main	Winnipeg, Canada	2022	142	42	Tallest DELTABEAM structure in the world. Peikko.	

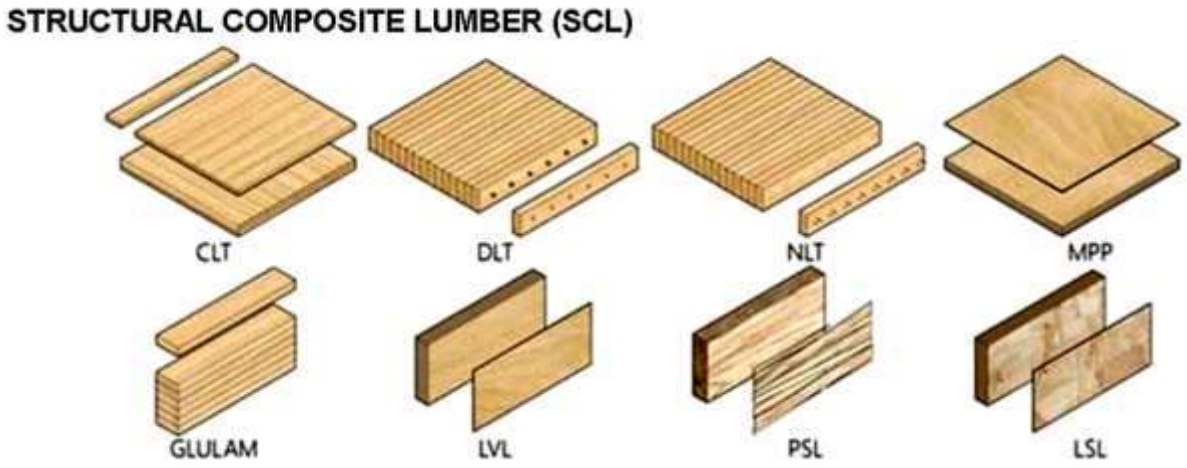


Fig. 3. Main species of mass timber: Cross Laminated Timber (CLT), Dowel Laminated Timber (DLT), Nail Laminated Timber (NLT), Mass Plywood Panel (MPP), Glued Laminated Timber (GLULAM), Structural Laminated Veneer Lumber (LVL), Parallel Strand Lumber (PSL), Laminated Strand Lumber (LSL).

There are 60 buildings in Europe (54 completed, 6 under construction): Sweden 12, France 11, Norway 11, United Kingdom 7, Finland 5, Germany 4, Netherlands 3, Austria 2, Italy 2, Switzerland 2, Spain 1.
There are 15 buildings in North America (6 completed, 9 under construction): Canada 10, USA 5.

There are 9 buildings in North America (6 completed, 3 under construction).
There is 1 completed building in Asia, in Singapore.

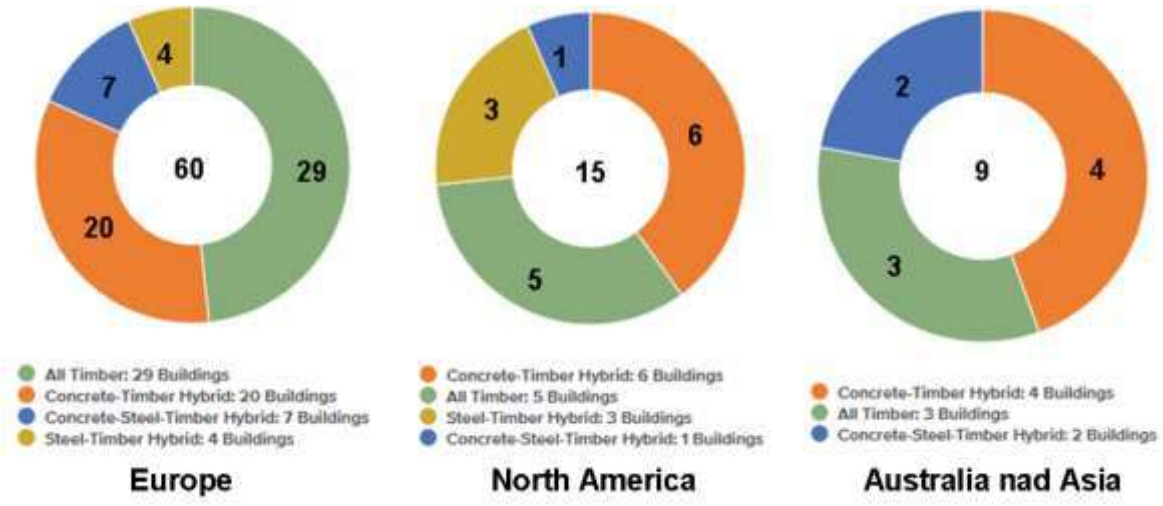


Fig. 4. Number of tallest mass timber buildings grouped according to the structural type [1].

3. Examples of mass timber in tall buildings



Fig. 5. Parameters see in Table 1 [1].

Hybrid construction systems integrate different structural materials to carry specific design loads. Connection and joint details are crucial for hybrid structures.

More details about mass timber structures may be found in [4]. The study analyses 350 contemporary multi-storey timber building projects between 2000 and 2021. It consists of 300 built projects, 12 projects currently in construction, and 38 design proposals.

Some advantages of the timber-steel hybrids [3]: a) greater spanning strength and ductility than timber alone; b) better suitability for lateral restraint systems especially for taller buildings; c) more flexibility with layouts and later renovations or change of use; d) ease of assembly and lower weight than concrete-timber hybrids; e) greater dimensional accuracy, steel akin to mass timber.



Fig. 6. Lighthouse Joensuu, Finland [1]. Timber-steel hybrid, 48 m, 14 floors, 2019, residential.



Fig. 7. HoHo building in Vienna, Austria [1]. Timber-concrete hybrid, 84 m, 24 floors, 2020, mixed-use.

Table 2. The tallest all-timber buildings (state in November 2023) [5].

Name	City, State	Comple-tion	Height [m]	Floors	Function
Mjøstårnet	Burmunddal, Norway	2019	85,4	18	residential, hotel, office
Limberlost Place	Toronto, Canada	2024	52,5	10	education
503 on Tenth	Portland, USA	2024	50	10	office
Treet	Bergen, Norway	2015	49	14	residential
Butler Square	Minneapolis,USA	1908	44,5	9	industrial
Obayashi Training Facility	Yokohama, Japan	2022	44	11	office
Hoas Tuuliniitty	Espoo, Finland	2021	?	13	residential
T3 Bayside	Toronto, Canada	2023	42	10	office
Blindern Studenthus	Oslo, Norway	2019	?	12	residential
T3 Sterling Road Building 5A	Toronto, Canada	2023	39,8	8	office
Auckland City Mission – Homegound	Auckland, New Zealand	?	?	10	office/residential

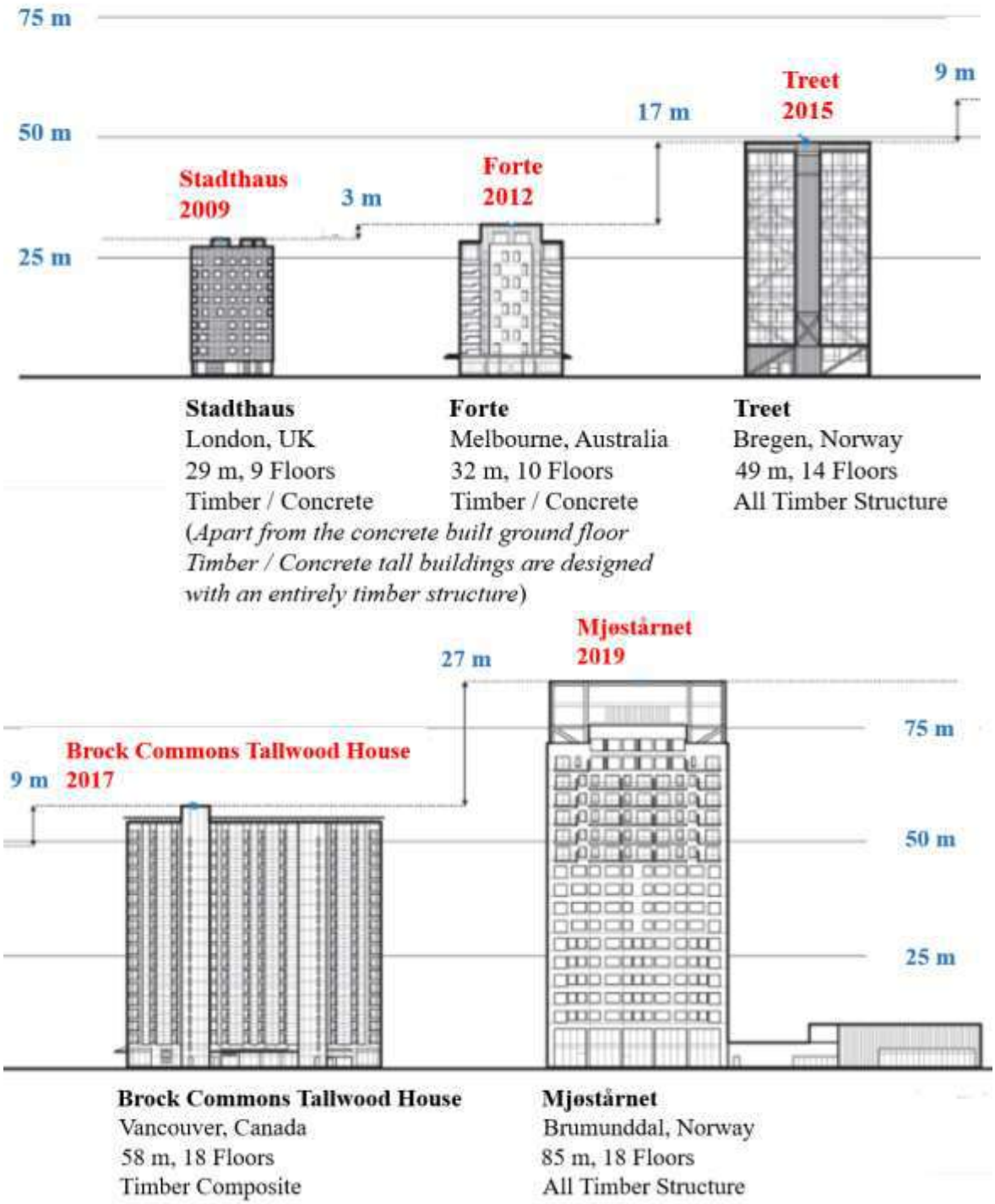


Fig. 8. The progression of the World's Tallest Timber Building has been swift, from 29 meters in 2009 to 85 meters in 2019; an almost 300% increase [6]. In November 2023 the tallest timber building is the Concrete-Timber Composite Over Concrete residential building Ascent in Milwaukee, USA, from 2022 with 86,6m and 25 floors (Figs. 2 and 5).

CTBUH reviewed till September 2020 more than 75 complete, under construction or proposed tall buildings around the world, 7 stories or 25 meters in height, that utilize mass timber in their structure, and determined their distribution by function, region and structural material use.

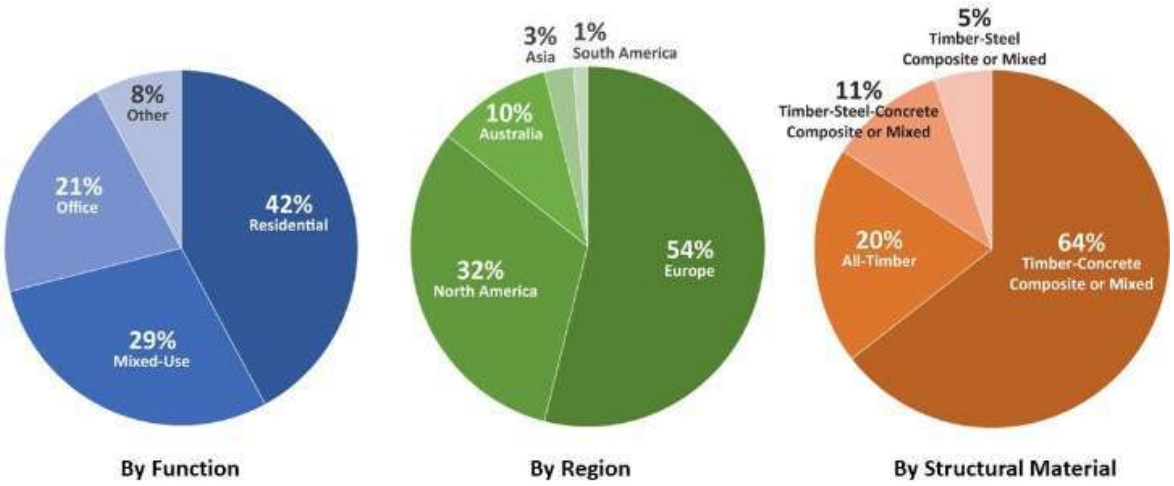


Fig. 9. Distribution of mass timber buildings [6].

4. Old timber buildings and churches

The Nanchan Temple, also located in Shanxi, is the world's oldest wooden structure The Great Buddha Hall, which is 11.75 meters long and 9.9 meters wide in plan, was constructed in 782 (see Fig. 10). During the past 1200 years, it has endured eight 5.0-plus-intensity earthquakes without sustaining damage [10].



Fig. 10. Nanchan Temple, Shanxi Province – the Great Buddha Hall, China [7].
Zeus 1234, CC BY-SA 3.0.

The Yingxian Wooden Tower (see Fig. 11) has been certified as resistant to a 7.0-magnitude quake, and the Nanchan Temple and The Hall of Supreme Harmony are rated to

withstand an 8.0-magnitude event. All these timber structures are still standing after suffering several strong earthquakes in the past hundreds of years, and have clearly proven the excellent seismic performance of timber structures, largely on account of the mortiseand-tenon connections used in all three structures. The deformation and frictional sliding between tenons and mortises, combined with the light weight, good ductility, and strong energy dissipation capacity of the system, allowed the buildings to balance, absorb and ultimately dissipate the seismic energy they experienced, and preserved their integrity [10, 11, 12].

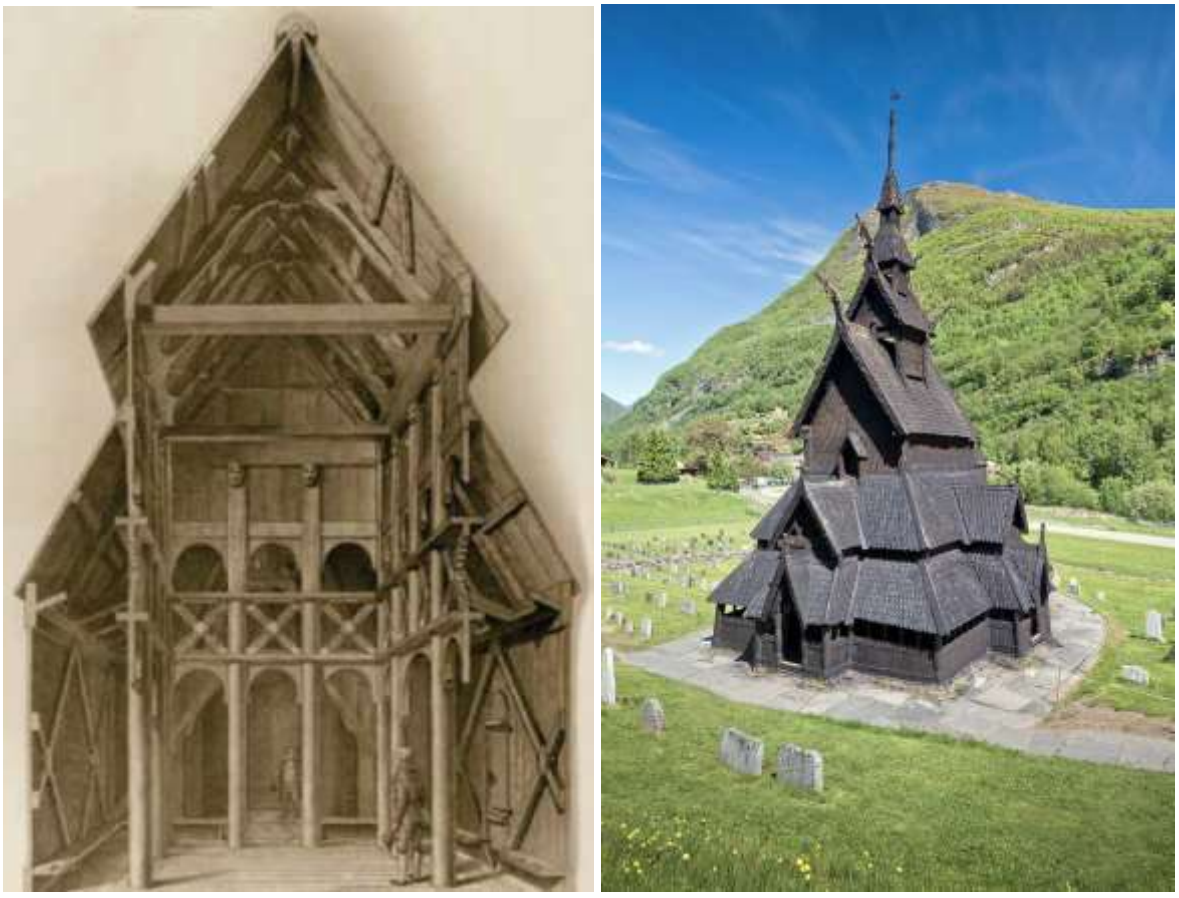


Fig. 11. Yingxian Wooden Tower, Shanxi province, China **[7].**
Charlie fong, CC BY-SA 4.0.



Fig. 12. The Hall of Supreme Harmony, Forbidden City, Beijing, China [7].
Rabs003, CC BY-SA 3.0.

The Hall of Supreme Harmony inside the Forbidden City (Palace Museum) of Beijing was first completed in 1420, and then reconstructed after suffering several fires (see Fig. 12). The current structure wasconstructed in 1695. It is 26.9 meters high, 64 meters long, and 37 meters wide. After reconstruction, it survived seven strong earthquakes with a seismic intensity of 6.0 or higher, and whose epicenter was less than 90 kilometers away. During an 8.0-magnitude quake in 1679, no damage occurred, even though the epicenter was only 45 kilometers away [11].



a) **b)**

Fig. 13. Borgund Stave Church, Norway, circa 1200 AD, still stands today; a) Drawing by G.A. Bull. (Public domain) [8]; b) Ximonic (Simo Räsänen), CC BY-SA 3.0.

Wood has been an integral part of Scandinavian architecture for centuries, deeply rooted in the region`s culture and history. The abundance of forests and the skillful craftsmanship of local communities have fostered a deep connection to wood as a building material,. From traditional stave churches to iconic timber houses, Scandinavia`s architectual heritage ambraces the use of wood in various structural systems. The drawing of the Borgund Stave Church in Fig. 13, for instance, shows this unique system of raised roofs, with which the tall internal posts (staver) are interconected with brackets (buaknær). Many of these practices have been carried forward onto contemporary high-rise construction. For instnace, the inclined timber frame crowning Mjøstårnet, in Brumunddal, Norway (Figs. 5, 8), resembles a conventional single-family timber frame home with a pitched roof [8].

Nordic countries have a rich history of using timber as a primary building material, with traditional vernacular architecture showcasing the beauty and functionality of wood construction. Wood appears not only in single family houses, but also in relatively high-

rise structures, known as Stave churches, for instance, Heddal Stave Church (1250 CE) in Notodden, Norway, (Fig. 14) which is the largest stave church in Norway.

The solid timber has many advantages over modern conventional structural materials, for instances it requires less energy than concrete or steel for its production. As a renewable resource with a negative carbon footprint and low embodied energy, timber is central in the discourse on carbon-neutral, energy- and resource-efficient construction [9].

The discourse of architectural scale plays a crucial role in envisioning timber high-rises. In early-modernism, timber has been associated with low-rise and midrise structures due to perceived limitations in its load-bearing capabilities. However, advancements in engineering, computational design, and manufacturing technologies have challenged this notion. Through the integration of engineered wood products such as cross-laminated timber (CLT), glulam, and laminated veneer lumber (LVL), architects and engineers are pushing the boundaries of what is possible in terms of height and scale. One maginificent effort is the Mjøstårnet building in Brumunddal, Norway (Figs. 5, 8), known as the Wood Hotel, which is officially the world's tallest wooden building, and it contains a hotel, apartments, offices, restaurant, common areas and a swimming facility [9].



Fig. 14. Heddal Stave Church, Notodden, the largest stave church in Norway Source: Micha L. Rieser; in the Norwegian Directorate for Cultural Heritage database [9].

There are unique wooden churches in Slovakia, the peculiarity of which was the construction technology - all the material had to be made of wood and nails were not allowed to be used during its construction. According to historical records, there were

more than 300 wooden churches in Slovakia, in which elements of Western, especially Roman and Eastern Byzantine culture intermingled stylistically. Currently, we still have up to 50 of these sacral monuments from the 16th - 18th centuries.

The oldest preserved Roman Catholic wooden churches are influenced by the Gothic style: Trnové (1500) - Žilina district, Rudno - relocated to the Slovak Village Museum in Martin, Tvrdošín (15th century) and Zábrež (1647), which is transferred to the Orava Village Museum in Zuberec - Orava. The only Roman Catholic church from the set of national cultural monuments is located in the village of Hervartov in the district of Bardejov.

Church of Saint-Francis of Assisi (Figs. 15, 16) is a Roman Catholic church situated in the village of Hervartov, Prešov region, The Slovak Republic. The church was constructed in wood at the end of 15th century.The wall paintings date between 1655 and 1805. On July 7, 2008, the church along with seven other monuments was declared UNESCO world heritage site under the name "Wooden Churches of the Slovak part of the Carpathian Mountain Area"

Roman Catholic Church of St. Francis of Assisi in a Slovak village of German origin, with its two-part floor plan, high roof and other features in the exterior and 16. and has a purely Gothic character. This wooden church from the end of the 15th century represents the oldest type in Slovakia. It is made of red spruce. The building consists of a polygonal sanctuary, a nave, a sacristy and a porch.. The interior underwent several changes in decoration, mainly during the Reformation period, when rare Reformation monuments were added, such as wall paintings from 1665. In the 18th century, part of the Baroque decoration and a late Baroque painting by J. Mirejovský from 1803 were added. The main altar, the Altar of the Virgin Mary, St. Catherine of Alexandria and St. Barbory from 1460-1470 was restored to its original form in 1985-1990 [13].



Fig. 15. Roman Catholic Church of St. Francis of Assisi in the village of Hervartov in the district of Bardejov, Prešov region, The Slovak Republic. Source: Muzeum.sk [13].



Fig. 16. Roman Catholic Church of St. Francis of Assisi in the village of Hervartov in the district of Bardejov. Prešov region, The Slovak Republic. Source: Muzeum.sk [13].



Fig. 17. Butler Square, Minneapolis, USA. It has industrial function. CC BY-SA McGhiever via CTBUH. See Table 2 [5].

Timber roof of Notre-Dam Cathedral in Paris [14, 15].

On April 15, 2019, a devastating fire occurred at the infamous Notre Dame Cathedral in Paris, France. The fire caused significant damage to the historic structure, which is a landmark in the city and a symbol of French Gothic architecture. The cathedral has been undergoing restoration works and is on course to reopen to the public in 2024.

A significant portion of Notre-Dame Cathedral's roof was completely destroyed. Before the fire, the wooden lattice structure underneath the cathedral's lead roof was known as the "forest". It was made up of centuries old oak trees that were added to the cathedral between 1220 and 1240. Its impressive dimensions measured more than 300 feet long, 42 feet wide and the transept was over 30 feet tall. Before it collapsed in the 2019 fire, it was one of the oldest frameworks in Paris. To reconstruct the roof, workers will install 26 timber frames (known as "bents" or "trusses"). This work is expected to take place from 2022 to 2024 [14].

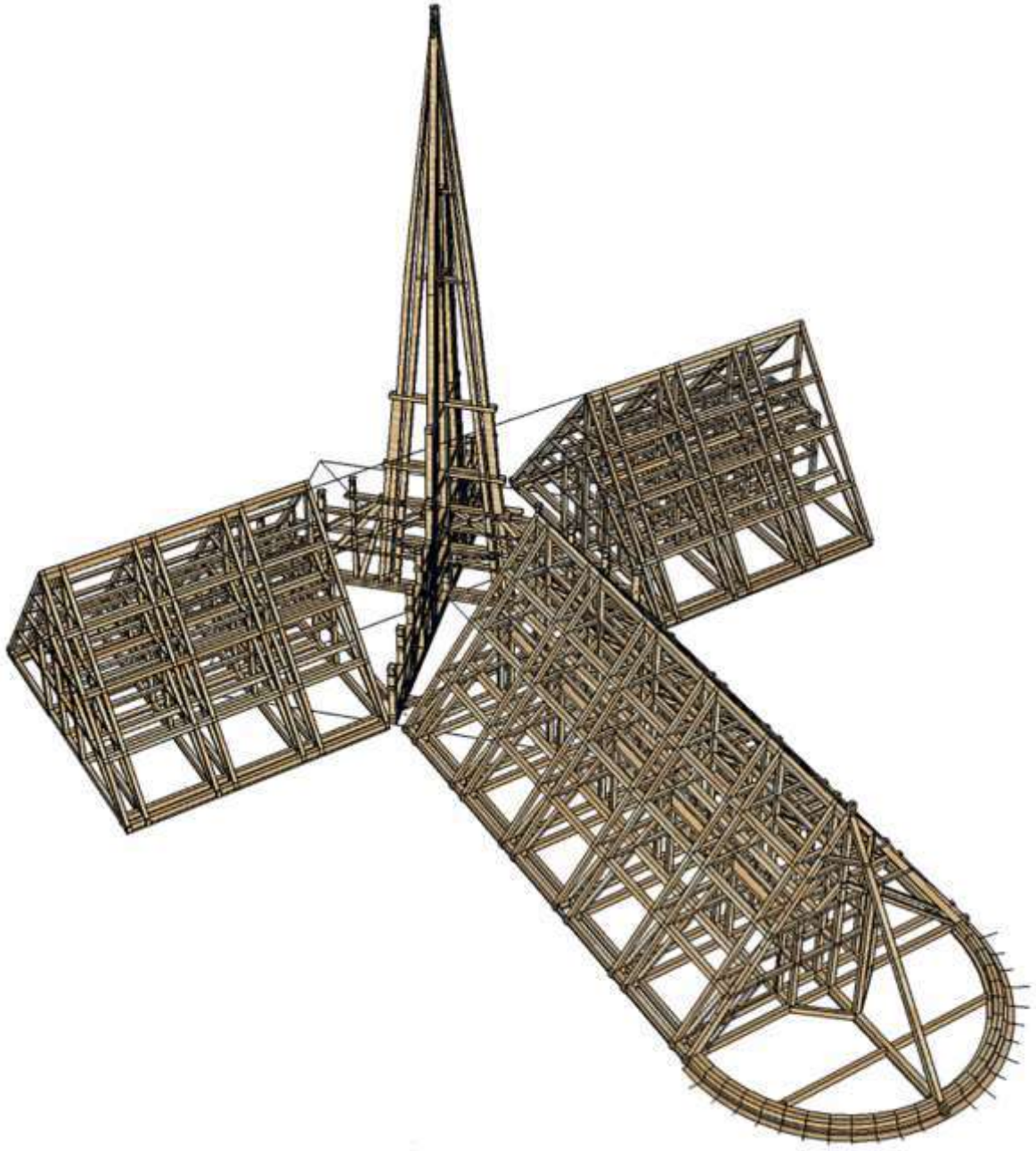


Fig. 18. Timber roof of Notre-Dam Cathedral in Paris [15]

In March 2020, 1,000 oak trees were cut from roughly 200 French forests to make the frame for Notre-Dame Cathedral's roof and spire. These trees were selected through a painstaking process in January and February 2020, as they need to fit the precise requirements defined by the chief architects in charge of Notre-Dame's restoration.

The oaks selected are roughly 150- to 200 years old, with some standing over 60 feet tall. Felled at the end of March, before their sap rose to prevent humidity in the wood, the wood will be dried for 12 to 18 months to make sure it does not shrink or move once in place.

In response to criticism over the felling of the oaks – some of which stood on plantations ordered by former kings to build ships and ensure the grandeur of the French navy – officials say that the number of trees felled represented a very small percentage of the oaks harvested each year in France, and that these trees would have been felled anyways as part of routine forestry maintenance.

In June 2020, the first four trees cut from the Bercé forest near Le Mans, France, were sent to a sawmill about 60 miles away. Transporting the four trees, with the longest measuring 85 feet long, required two cranes and a special trailer [14].

Think Wood about fire and seismic safety of timber structures [16].

In response to concerns about fire and seismic safety as it relates to wood construction, industry-driven group Think Wood continues to build resources in its Think Wood Research Library supporting the application of softwood lumber in residential and nonresidential building projects. Since its 2017 launch, the library has amassed nearly 900 pieces of research to aid architects, engineers, and other industry professionals in the design and building of safe, high-performing wood structures. Among the information collected by the association is research demonstrating fire can be safely and effectively controlled in wood buildings when proper protections are in place [16].



Fig. 19. Arup`s fire test report demonstrates how in a fire, char forms on mass timber at a predicable rate, slowing combustion while retaining strength. (Photo courtesy of Arup/Think Wood) [16].

Conclusion

The Slovak and foreign students are informed by members of the Department of Languages at Faculty of Civil Engineering, Slovak University of Technology in Bratislava about current problems related to the architecture and civil engineering. The topic related to the possible use of timber in the modern timber high-rise buildings and in the old timber buildings and churches as well is very interesting for the students. The paper should help the students better understand terminology used in [14]. Given information may be interesting also for designer in practice.

Acknowledgement

This paper did not receive any specific grant. Purpose: just to help students and to inform designers in practice.

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ДОКТОРСКИЯТ ПАМЕТНИК В СОФИЯ. ТИПОЛОГИЯ И АНАЛОГИИ. ПРОЕКТИРАНЕ И ПРОЕКТИ

THE DOCTOR'S MONUMENT IN SOFIA. TYPOLOGY AND ANALOGIES. DESIGN AND PROJECTS

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***Резюме**: Изследването запознава с малко известното в България име на Антонин Йозеф Томишка – архитекта-проектант на монумента, издигнат в чест на загиналите в руско-турската освободителна война през 1877-1878 военно-медицински служители – лекари, фелдшери, ветеринари, и фармацевти. Типологията и причините за построяването на паметника са разгледани в сравнителен аспект във връзка с подобни – от времето на Кримската война и жертвите по това време в Англия. Във връзка с типологията паметникът е разгледан в сравнение с други руски военни паметници в Обединеното кралство. За монумента в София са разгледани проектите, проектирането и изпълнението, описани в документи от Руския държавен военно-исторически архив в Москва. Неговата типология отвежда към монументалните съоръжения, характерни за обелиска или пирамидата. Поставянето на саркофаг върху нея засилва монументалното въздействие на съоръжението.*

***Key words**: Докторски паметник в София, проекти, архитект Антонин Йозеф Томишка*

***Abstract:** The research introduces the name of Antonin Josef Tomishka, little known in Bulgaria – the architect-designer of the monument, erected in honor of the military medical personnel – doctors, veterinarian, paramedics and pharmacists - who died in the Russian-Turkish war of liberation in 1877-1878. The typology and reasons for the construction of the monument are examined in a comparative aspect in relation to similar ones - from the time of the Crimean War and the victims of that time in England. In relation to typology, the monument is examined in comparison with other Russian war memorials in the UK. The composition of the monument in Sofia, its projects, design and implementation, described in documents preserved in the Russian State Military Historical Archive in Moscow, were examined. Its typology leads to the monumental structures characteristic of the obelisk or the*

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pyramid. Placing a sarcophagus on it enhances the monumental impact of the facility.

Key words: *Doctor's monument in Sofia, projects, architect Antonin Josef Tomishka\Tomiška*

1. Увод. Типология. Поджанрови разновидности на поменалните монументи от XIX в.

Докторският паметник в София е едно от редките по характера си съоръжения в почит на загинали във война медици – хората, които са призвани да спасяват живота на полето на смъртта и да бъдат до последния момент до воюващите. Тези специалисти също намират своята Голгота по различни поводи – някои от тях на територията на България, други след завръщането си в Русия – от страдания и увреждания. В този смисъл става въпрос за осъществяване на една благородна идея в памет на 531 души, чиито имена са изписани на елементите от фасадата на монумента.

Малко по-ранен е мемориалът, издигнат във военната болница „Нейно величество кралица Виктория“ в Нетли, Саутхемптън, в памет на медиците, загинали по време на Кримската война [1]. Известен е по публикация на гравюра от албума с фотографии на принца на Уелс (по-късно крал Едуард VII) [2] и от малкото запазени фотографии. Паметникът по план е октагон, подобен на готически донжон, базата му представлява стъпала. В основата на представляващата три нива кулообразната структура, отдалечено напомняща и построената в стил неоготика Спаска кула на Кремъл, се виждат фини неоготически трисъставни в очертанието си арки. Те са изключително изящни, повтарят се на всяко ниво, а в третата под арките са вписани скулптури. Най-високата част е увенчана с кръст. Монументът е изграден от портландски камък и е открит през 1864 г. Рисунката на кулата е вписана в широка островърха арка, около която са разположени растителни орнаменти, а в горната част е изписан надпис: “Memorial to the medical officers of the/Armi who died in the Crimean war (година с буквени означения, не се чете, поради неправилното изписване на повечето букви в целия надпис)“. В албума, авторът е посочен като неизвестен, въпреки наличието на подпис, макар и трудночетлив [3]. Разчитането и сравнителното изследване ориентира към автор с име Томас Хайтер Луиз (Thomas Hayter Lewis, (1818-1894), архитект, почетен професор, който всъщност е и архитект на съоръжението [4].

На седем от паната са записани имената на служителите, в чиято памет е издигнат паметникът. Те са на заместник главния инспектор, хирурзите, техните асистенти, с изписани данни за техните военни части, главните фармацевти, анестезиолозите, отговорниците за медицинското облекло, цивилните хирурзи и анестезиолози. На паметника не са включени онези членове на отдела, които са били инвалидизирани и впоследствие починали. [5] На осмото е следното описание: „В памет на онези офицери от армейския медицински отдел, които загинаха, докато служат в армията на Изток, по време на войната с Русия, през 1854, 1855 и 1856 г. Този паметник е издигнат от техни братя офицери и приятели“. Теренът за мястото е предоставен от лорд де Грей, държавния секретар по военните въпроси, и подготвен за основите под ръководството на майор Рейвънхил Р.Е. Разходите за изграждането му възлизат на 8921 лири.



Фиг. 1. *Мемориал, издигнат във военната болница „Нейно величество кралица Виктория“ в Нетли, Саутхемптън, в памет на медиците, загинали по време на Кримската война, 1864 г.*

Посочен по: https://www.netley-military-cemetery.co.uk/hospital-buildings/the-crimean-war-memorial-and-the-officer-s-mess/

През 1963 г. болницата е засегната от пожар и е съборена през 1966 г., а паметникът – през 1977, поради тежкото му състояние. До увреждането му довежда близостта със солената вода и липсата на юридическа защита, тъй като се оказва, че той няма статут на недвижимо културно наследство. При направените постъпки да бъде преместен, разходите през 1969 г. са оценени на 25 000 британски лири. След разрушаването му посветителната плоча е преместена в сградата на щаба на казармата в Keogh. Когато Центърът за обучение на медицинските служби на отбраната освобождава казармата през 2014 г., плочата с надписа панела е преместена в музея на военната медицина. Аналогията с Докторския паметник е, че този монумент е издигнат в близост до болница, така, както е имало идеи и за паметника в София.

Типологията на паметника в Нетли отвежда към друг, подобен по силует и художествени особености, какъвто е Руския паметник в църковния двор на храма Сейнт Джон суб Кастро (Russian Memorial In The Churchyard Of St John Sub Castro), издигнат от мрамор и камък към 1877 г. в град Луиз в Източен Съсекс, Англия в памет на руски войници, загинали като пленници. Издигнат е по заповед на руския император Александър II.



алица Виктория" в Нетли, Саутхемптън, в памет на медиците, загинали по време на Кримската война, 1864 г.

Посочен по: Netly Abbey matters! (Supporting Alzheimer's Research UK), 2024 https://www.netleyabbeymatters.co.uk/the-royal-victoria-military-hospital-pre-1914

От горните посочени примери следва да бъде направен извод, че паметниците с военен характер в Обединеното кралство, издигнати през XIX в. носят почитта към загиналите, чрез монумента и неговата форма, посредством текстовете, както и местоположението – за паметника на медиците – в близост до кралска военна болница или по нареждане на руския император. Тези детайли ще послужат при последващите коментари по отношение на Докторския паметник в София.

Паметникът на загиналите медици по времето на Руско-турската война в София остава единствен, съществуващ в момента, след унищожаването на този в град Нетли.

Още един вид сравнителен анализ посочва, че в Британското кралство като един от участниците в Кримската война, има установени различни типове военни паметници, които можем да причислим към типологията на военните паметници от XIX в. Сред тях са обелиски, колони и октагони-кули. [6] Тази типология е характерна за поменалните паметници през XIX в. Докторският паметник в София е четиристенна пресечена пирамида, увенчана със саркофаг с четири колони по ъглите.

През XX в. формите варират, като пример е паметникът на загиналите по времето на първата световна война медици в Букурещ от 1932 г. [7] Той представлява пирамидална композиция с пиедестал на четири ниски колони, увенчан с бронзова фигурална композиция.



Фиг. 3. *Паметник на медицинските герои в Букурещ, 1932 г.*
Посочено по: https://depositphotos.com/photo/medical-heroes-monument-bucharest-romania-135464140.html?ref=26681138

В по-късно време почитта към лекарите се превръща в традиция, но паметникът в София е вторият, след този в Англия. Първият е разрушен, поради скъпоструващото му преместване и липсата на юридическа защита като културно наследство. По-късните паметници са свързани най-вече със загиналите медици при лечението на Ковид-19. В случая става въпрос за различни жанрови подразделения на паметници, издигнати във връзка с войни, за военни или цивилни медици, загинали при военни действия, за медици загинали без военни действия и пр.

2. Докторският паметник в София

Паметникът в София е обявен за паметник на културата с титула „Паметник на загиналите медицински чинове в руско-турската освободителна война" (в. „Известия на Президиума на Народното събрание", бр. 73/ 1955 г.) и е деклариран за исторически, мемориален и архитектурен паметник на културата с категория „национално значение" (писмо на НИПК №2637/ 07.06.1988 г.; №4349/04.12.1992 г.). Вписан е в издание за военните паметници в област София. [8]

За разлика от паметника в Нетли в Обединеното кралство, който както беше разгледано, е издигнат по решение на кралицата в близост до военна болница, а също така, имайки пред вид и различната историческа обстановка, е създаден по решение на командването на руските войски. Средствата за него са събрани от доброволни дарения като те са били пропорционално разпределени между почитта и към загиналите военно-медицински чинове в град Карс, издигнат през 1910 г. [9]



Фиг. 4. *Паметнкът в Карс, 1910 г.*
Посочен по: https://noev-kovcheg.ru/mag/2007-10/910.html

Архитектът на паметника в София, който, подобно на паметника от Нетли, също става популярен, благодарение на репродукция на гравюра, е Антонин Йозеф Томишка, чех по народност и архитект в Руската империя. Той завършва Художествената академия в Санкт Петербург и преминава през задължителното за времето европейско стипендиантство с командировки в университетите във Виена и Рим за усъвършенстване на квалификацията. След завръщането си в Санкт Петербург става проектант на едни от най-значимите и сложни съоръжения. Сред тях са затворите (80-90-те г. на XIX в.), синагогата в Санкт Петербург (1883), приюти за немощни и бедни хора (1880-1881), училище за слепи деца (1887-1889), преустройството на Гробницата на великите князе в Петропавловската крепост (1900). Неговият проект (1882) за църквата „Възкресение Христово на мястото на смъртоносното раняване на починалия император Александър II“ [10] е първи в проектната надпревара при първия етап за издигането на този толкова съществен за руската държава храм. Антонин Томишка е професор в Академията (1888), ръководи ателието по архитектура (1894-1900), архитект в Министерството на вътрешните работи, действителен статски съветник – една от висшите длъжности в Руската империя, със специални привилегии в обществото.



Фиг. 5. *Едно от най-ранните изображения на Докторския паметник, публикувано през 1885 г. в западния печат. (Неизвестно издание)*
Посочено по: http://stara-sofia.com/shipka.html

Антонин Томишка, чието име се произнася неправилно в България като Томишко или Томишек, е роден през 1851 г. в град Хержманув мниестец, надалеч от Пардубице, тогава в Австро-унгарската империя. За да има възможност да работи успешно в Руската империя той приема православие и става поданик на руската корона. По този начин пътят на талантливия студент е отворен към проектирането в областта на сигурността, каквито са затворите и благотворителните заведения за приютяване на престарели, немощни хора, инвалиди, сакати и оздравяващи по това време, или така наречените богаделни. Най-впечатляващата сред сградите с подобно предназначение са Волковската богаделня на търговците (1880-1881) и затворът „Кръстове“ (1884-1889/1890), определени в руските издания „промишлен модерн“ (неоруски стил).

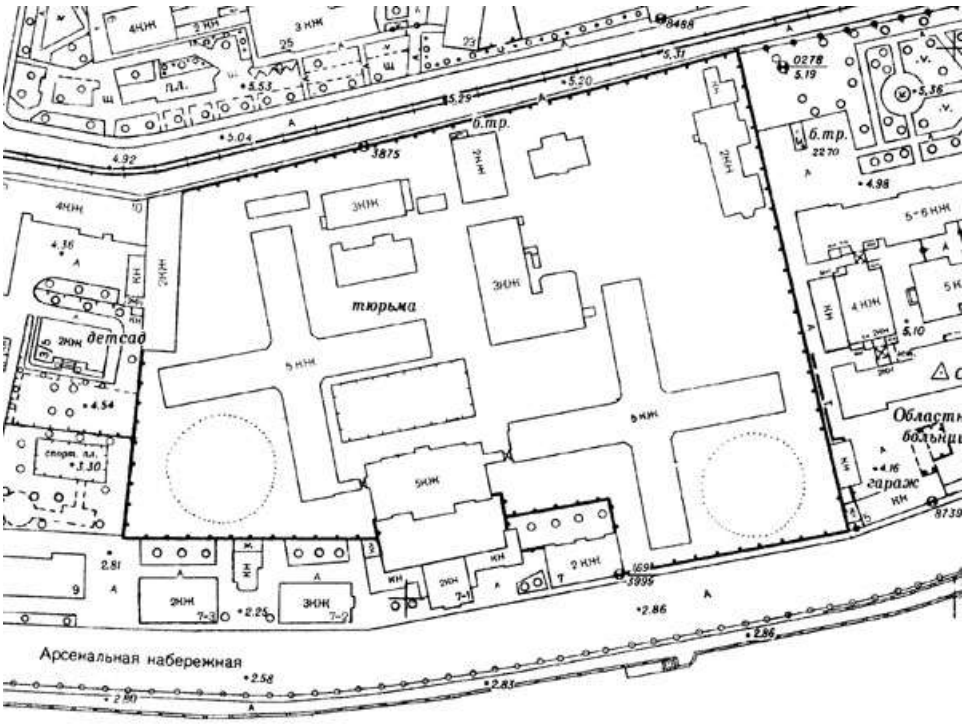


Фиг. 6. *Волковската богаделня, 1880-1881 г., Санкт Петербург*
Посочено по: http://kupchinonews.ru/2019/04/09/volkovskaya-bolnitsa-stala-pamyatnikom-regionalnogo-znacheniya/

Значението за България на архитекта Антонин Томишка за проектирането на този единствен по рода си паметник, може да бъде сравнено с това на военно-полевия хирург Николай Пирогов. В двата тома на своя капитален труд [11] лекарят описва болничните съоръжения на полето на бойните действия и своята работа за войниците, които е спасил. Военните медици, чиито имена са увековечени на стените на увенчаната със саркофаг пресечена пирамида в Докторската градина в София са лекари като Пирогов. За разлика от работата на Николай Пирогов, приносите на архитект Томишка остават непознати и недооценени, а неговата обширна проектантска дейност е изцяло неизследвана в България. Неговата популярност в Русия като известен проектант на обществени и жилищни сгради и най-вече на сградите за наказания, каквито са затворите, е огромна. Приживе работата му в България по проектирането на църквата край с. Шипка и Докторския паметник става известна в голям брой професионални и популярни периодични издания, издадени в Русия, близки до времето създаването на съоръженията. [12]

Ангажиментите на Антонин Томишка в България съвпадат с началото на неговия творчески път. По отношение на проектирането в България забележително е неговото участие през 1881 г. в конкурса за строителство на руската църква „Рождество Христово“ в Шипченския манастир при село Шипка и впоследствие за Докторския паметник в София.

Към 24.11.1881 г. за храм-паметника са представени осем проекта, сред които са и на преподаватели на Томишка: на Александър Иванович Резанов, Давид Иванович Гримм, Роберт Андреевич Гьодике, Рудолф Богданович Бернгард, инж. Е. Н. Жибер. На 19.01.1882 г. са премирани са проектите на Антонин Йозеф Томишка, Виктор Александрович Шрьотер и Александър Францович Зигелберг. Проектът на Томишка е поставен на първо място с премия от 1000 рубли.



***Фиг.** 7. Планова схема на затвора „Кръстове", 1884-1889/1890, Санкт Петербург*

Посочено по: https://www.save-spb.ru/page/houses/houses/kresty_sledstvennyi_izolyator.html?section=houses/houses

Сведението е отразено в изданието „Кишиневски епархиални ведомости" като отчет на комитета по изграждането на православен храм за вечно възпоминание на падналите по време на войната 1877-1878 г. През 1886 г. е положен основният камък. След прекъсване, строителството е подновено през 1897 г. под ръководството на арх. Александър Никанорович Померанцев (председател на строителната комисия е граф Николай Павлович Игнатиев), непосредствен ръководител на строителството е арх. Александър Николаевич Смирнов. В началото на 1882 г., когато е премиран проектът за строителството на църквата „Рождество Христово" при с. Шипка, Томишка се съгласява да ръководи строителството, при условие, друго лице ще осъществява надзорът. По това време се водят разговорите за Докторския паметник. В същото време в Санкт Петербург се довършва зданието на Волковската богаделня.

Църквата „Рождество Христово" в Шипка е първото му участие, извън стипендиантското му проектно отчитане с „гръцка" църква за 3000 човека. Всъщност Томишка излиза с едно доста смело решение – да проектира една сложна композиция в „руски стил" върху дарения от българите терен. Интересното при строителството на тази църква е, че той отказва да обследва мястото като изпраща архитект Роберт Марфелд, изпраща и друго лице да направи подробна сметка на стойността на строителството и иска голямо възнаграждение за надзора. Накрая със строителството се заема Т. И. Губонин [13], който скоро се отказва и препоръчва запасен капитан Ф. Е. Струве. [14]

През 1881 г. Томишка не пътува до с. Шипка, тъй като или е зает с Докторския паметник или знае, че през следващата 1882 г. трябва да посети Източна Румелия, както отбелязва капитан Христо Чаракчиев. [15] Поради обвързването във времето на двата проекта, може да се предположи, че Антонин Томишка получава едновременно или обвързва двете задачи.

Докторският паметник в София, може да бъде определен като специфично монументално и поменално съоръжение. Двете събития – проектирането и обследването на терена за църквата край с. Шипка и проектирането на Докторския паметник са обвързани във времето.

3. Проекти за Докторския паметник в София

За да се достигне до проектирането и издигането на паметника е осъществена огромна подготвителна работа. На 25 юни 1878 г. лекарите от действащата руска армия вземат решение за събирането на средства сред медицинските съсловия в Русия за изграждането на възпоминателен паметник на загиналите във военните действия медици, ветеринари и фармацевти, който да се издигне в Търново на територията на бившата Османска империя. И тъй като това е времето на временното руско управление в България, в съответствие с реда, въпросите са отправени чрез съответните документи до правителствената канцелария в град Николаев за ходатайство, императорът да разреши да се издигне паметник на медицинските чинове от действащата армия не само в България, но и в Азия. За целта „висшата военномедицинска управа" се обръща и към полевия военномедицински инспектор в Сан Стефано по въпроса за построяването на паметници в Европейска и Азиатска Турция на загиналите от медицинския персонал в Руско-турската война. [16].

Топосите Санкт Петербург, столицата на Руската империя – София, столицата на княжество България, където има временно руско управление по силата на Сан Стефанския мирен договор (19 февруари/3 март 1878 г.) – и самото Сан Стефано, където е разположена главната квартира на руските войски (след изместването й от Одрин, където е била канцеларията след приключването на военните действия от руско-турската война на 19/31 януари 1878), стават главни опорни точки при вземането на решенията за строителството на двата паметника. Средствата се събират от юли до септември 1878 г. по медицински звена. Създадена е комисия по организацията за издигането на паметника, [17] която впоследствие съставя идеята за неговия образа (идейното задание), организира конкурса и осъществява строителството му. Тя преценява, че за да се вземе решение за височината и формата на паметниците, са необходими сведения за броя на надписите и за техния размер, което също така е свързано с представянето на сведения от Главното медицинско управление за броя на загиналите, времето и мястото на смъртта им.

Следващата стъпка за оформянето на идеята за издигането на Докторския паметник са списъците с имената на загиналите, които постъпват до Главното медицинско управление. Те са от Тифлис с информация за Кавказ, от Одеса – за Румъния, България и Европейска Турция. Императорът издава своето благоволение на 12 юли 1878 г. , а през май 1881 г. комисията завършва своята работа и докладва на Главния военно-медицински инспекторат.

Комисията решава да се изгради паметник само в България, а азиатският да се отнесе към разпореждането на Окръжния медицински инспектор на Кавказкия военен окръг със съдействието на Императорското Кавказко медицинско общество като част от събраните средства ще се предадат пропорционално на починалите там медицински чинове. Издирените имена на загиналите на местата на военните действия в Европейска Турция са 543, а в Азиатска Турция са 202, затова средствата за европейския паметник трябва да бъдат две-трети от събраните.

През 1880 г. на комисията са предадени и идейни скици (не са открити) на два частни проекта: единият на лекар – доктор Н. Дворяншный, който „напомня фантастични ескизи", вторият е дело на инженер-полковник, някой П. Бенард. Описан е като „красив", но не може да бъде оценен в художествено отношение без

участието на тесни специалисти, тъй като за него липсва финансова обосновка. Той е определен като беседка за отдих и не напомня на монумент. Има купол но за да бъде видим, трябва да бъде повдигнат на около 20 м. [18]

В документите липсва описано строително задание, но от коментарите става ясно, че частните и безвъзмездни проекти не са на необходимата висота. За да получи удовлетворяващ проект за паметник, Комисията се обръща към Обществото на Санктпетербургските архитекти за да бъде организиран конкурс. Проектът на полковник Бенард е отклонен поради разминаването му с идеята за паметника и тъй като превишава средствата от 15 хиляди рубли. [19] Едва тогава комисията оформя изискванията си за проекта за паметника. Желателно е той да бъде от твърд камък и да няма мазилка. При строителството да не се употребяват метали, връзките да бъдат каменни, покривът да бъде от скалата аспид, която има тъмносин цвят, за фундамента да се използва портландски цимент, мраморната плоча с надписите да не бъде прикрепена с желязо.

За провеждането на конкурса са посочени определени условия: имената са най-съществената част от паметника; да бъде изразено самоотверженото изпълнение на дълга на медицинските чинове; мястото е град Търново; паметникът да бъде построен така, че да не се нуждае от ремонт; на паметника да са изписани имената, презимената и фамилиите на лекарите, ветеринарите и фармацевтите и имената и фамилиите на загиналите фелдшери. Единственото условие е фамилиите да са изписани напълно. Уточнено е, че ако имената не могат да се съберат, това може да стане на допълнително разположени плочи. Също така, ако е възможно да се посочи с кръст и мястото на погребението. Сумата за строителството е 15 хиляди рубли. Проектите да бъдат оценени съвместно с участието на Комисията по строителството и обществото на архитектите. Премираните проекти да бъдат три, да бъдат придружени с обяснителни записки, които да останат като собственост на Комисията.

На 4 октомври 1880 г. председателят на комисията съдебния медик и ръководител на катедрата по съдебна медицина, академик, професор Яков Алексеевич Чистович се обръща към Санкт Петербургското общество на архитектите, с молба да обявят конкурс за на проект. След получаването на 11 конкурсни проекта, съвместно със съответна комисия от архитекти тя избира и заплаща (за да станат нейна собственост) три от тях.

Първа премия получава проектът на художник-архитектът Александър Готгардович Гронвилд, участвал с мотото „Благодарна Русия за умрелите труженици". Втората премия е на академика по архитектура Антонин Йосифович Томишка с мотото „Непознат". Третата е на художник-архитекта Леонтий Николаевич Бенуа [20], участвал с мотото „В. Търново".

В крайна сметка е предпочетен вторият премиран – този на архитект Антонин Томишка, който при участието си в конкурса той нарича с мотото „Непознат". [21] И наистина той е непознат, тъй като до този момент не е проектирал паметник. Той е в началото на своя творчески път. Към 1881 г. току-що е постъпил на работа към Главното управление на затворите, а непосредствено преди това се е завърнал от продължителното си стипендиантство в Европа и е назначен за асистент по архитектура в Художествената академия в Санкт Петербург.

Спечелилият проект на Томишка и останалите, които стават собственост на комисията по строителството на паметника, не са известни нито като изображения, нито като идеи. От едно писмо от април 1882 г., когато вече е решено, че паметникът ще се строи в София, става ясно, че документацията е била придружена с фотографски снимки на макетите на проектите (в неизвестност). Очевидно проектът на Томишка в най-висока степен удовлетворява изискванията не само като идея, но и

като разходи за изграждане. Според идеята на комисията, на паметника трябва да бъдат изписани 550 имена с малко име, бащино име и фамилия, както и с местата на погребението на загиналите.

Следващото условие е всяко едно от имената да се чете от гледната точка на нормален човешки ръст. Двете задължителни условия създават затруднения и Комисията взема решение местата на смъртта и погребението да не се изписват на монумента. По този начин паметникът е лишен от основната си същност да бъде паметен документален мемориал. И тъй като имената са много, Комисията решава те да бъдат изписани по групи – лекари, фармацевти, ветеринари, фелдшери. Имената на покойниците да се разположат по азбучен ред по фамилия, на фелдшерите да се изпишат малките имена с първата буква, а на останалите буквите на първото и второто име. Липсата на място предопределя промяната на идеята – да се премахне емблемата от средата на пиедестала. [22]

Проектът на Томишка е избран поради неговата целесъобразност – еднороден материал като залог за голяма трайност на съоръжението; наличие на площ на стените за изписване на имената; съответствие между идеята и формата. Специално е отбелязано наличието на саркофаг, определен като римско-християнски и насочващ към поменалния характер на съоръжението. Дружеството на архитектите оценява проекта като надхвърлящ определената сума, но строителната комисия силно подкрепя идеята за облика му.

Същественият въпрос, който стои пред комисията е за изпълнителя на паметника, като тя предпочита той да бъде от България. В България по това време работят няколко руски инженери, които според комисията биха могли да се ангажират с техническото изпълнение на паметника.

3.1. Определяне на мястото на паметника

Следващата стъпка е работата по определянето на мястото на паметника в Търново. Тук комисията отново изпитва затруднение тъй като градът е тесен, а паметникът трябва да бъде издигнат на площад, при влизане в града или до път за да бъде видим. За целта е предписано да се извърши заснемане на местността, където ще бъде разположен. Да се събере информация за намирането на подходящ материал за строителството, който трябва да бъде камък, но находището му да бъде близо до града за да бъде по-евтина неговата доставка и да може да се обработва на части за да могат да се издълбаят надписите. Камъкът да не е полиран, защото корпусът и пиедесталът трябва да бъдат грубо обработени. Ако основният материал търпи фина полировка, то и саркофагът в горната част на пиедестала може да бъде от същия материал. Освен определянето на стойността на камъка, да се опреди цената на варта от висок клас и цената на труда на каменописеца и на майстора, който ще гравира надписите с имената. За по-голяма устойчивост могат да се допуснат още стъпала към пиедестала. Също така е предвидено след избора на мястото, ако основата не е камениста, фундаментът да бъде достатъчно дълбок и широк. Цялата зидария да бъде от камък с вар или цимент и да не се използва кирпич. Вътрешността да бъде от ломен камък, а облицовката - от обработен. Мястото на издигането на паметника да бъде безплатно „а ако това е невъзможно, въпросът да бъде изясняван с местните власти".

През 1880 г. един от военните инженери в София от времето на временното руско управление е Николай Копиткин. По времето на първото българско правителство е губернски инженер и правителствен строителен инженер. Негови помощници са архитект Вацлав Роубал (градоустройствен техник) [23], инж. Георги Прошек (автор па първия регулационен план на София), инж. Йозеф Прошек и инж.

Йероним Богутински. Заедно с тях работят и архитектите Антонин Колар и арх. Пешел. [24]

През есента на 1881 г. Николай Копиткин като „управляващ публичните работи на Българското княжество" [25] по времето на режима на пълномощията на княз Александър Батенберг, изпраща писмо до генерал Иван Василиевич Гримм. То е знаково по отношение на идеята и изпълнението на паметника. По това време Гримм е инспектор по медицина към Министерството на вътрешните работи в Княжество България и съставя Гражданските медицински закони. [26] От това писмо става ясно, че председателят на комисията по строителството Яков Чистович е задал въпроси на Копиткин, които интересуват и Гримм. Копиткин отговаря на Иван Гримм, че ако паметникът се строи в Търново, най-доброто място за него е на Марино поле на новостроящия се булевард. Ако се реши паметникът да бъде в София той може да се постави на един от градските площади, или в малка обособена градинка срещу новостроящата се болница. [27]

Спомената болница би трябвало да бъде Александровската [28], строителството на която е завършено през 1884 г. [29] За изграждането ѝ Софийската община отделя парцел още през 1879 г. Идеята паметникът да се издигне срещу болницата показва, че през 1881 г. за него не е имало решение на общината за безвъзмездно предоставяне на място, тъй като все още не е ясно къде ще бъде построен той. (Така именно е построен паметникът на загиналите медици по време на Кримската война – в двора на военната болница „Кралица Виктория" в град Нетли.) По това време около Търново има находище на син на цвят (сиенит) и доста твърд камък, но той не може да се полира и затова материалът за саркофага трябва да се достави от Карара. Ако паметникът се строи в София, той може да се построи от ташкесенски пясъчник и саркофагът да се изработи от гранит. [30]

Следващо предпочитание на комисията е ръководството и контролът на строежа да се възложат за безвъзмездно изпълнение на инженерите от българското правителство.

В заключение Копиткин споделя, че си позволява да изрази личното си мнение, което се състои в това, че най-доброто решение по въпроса за строителството на паметника е той да се построи от местни породи от камък, саркофагът, средният камък с изображението на змия и чаша и долните плочи, където са разположени главните надписи и венците, да се направят в Карара. Надписите на долните плочи да са от металически букви вградени в мрамора. И тъй като в България няма нито един ваятел и „даже представа нямат за това, какво е полиран камък" ще се наложи довеждането от Италия на работници, специално за този паметник, което ще бъде несравнимо по-скъпо.

Писмото на инспектора по медицина към Министерството на вътрешните работи в Княжество България Иван Гримм до Началника на Военно-медицинското направление в Санкт Петербург Орест Иванович Рудински от края на 1881 г. дава окончателен отговор на въпросите за мястото на строителството.

Паметникът ще се построи в София в градинката срещу фасадата на Александровска болница и мястото за него ще бъде предоставено безплатно. За да се стигне до този извод, е имало и друга кореспонденция която липсва в архивите, тъй като в писмото е казано, че решението е на княза. Материалът ще бъде местен, а за саркофага ще бъде необходим карарски мрамор.

Тежестта на организацията на строителството пада на д-р Иван Гримм. Четири-шест месеца по-късно става ясно, че в България няма на кого да се повери строителството и добиването на строителния материал, не може да се изясни неговата цена и цената на труда. Затова военният министър Александър Каулбарс е помолен от военния министър на Русия за съдействие пред Негова Светлост

Българския княз да посочи безплатно място за строителството на паметника, както и да бъде посочено правителствено учреждение да се заеме с постройката на паметника, а също така да се фиксират разходите. През късната есен на 1882 г. продължават потвържденията във вид на телеграма, подадена от Иван Гримм, че князът на България желае да има паметник на руските лекари в София.

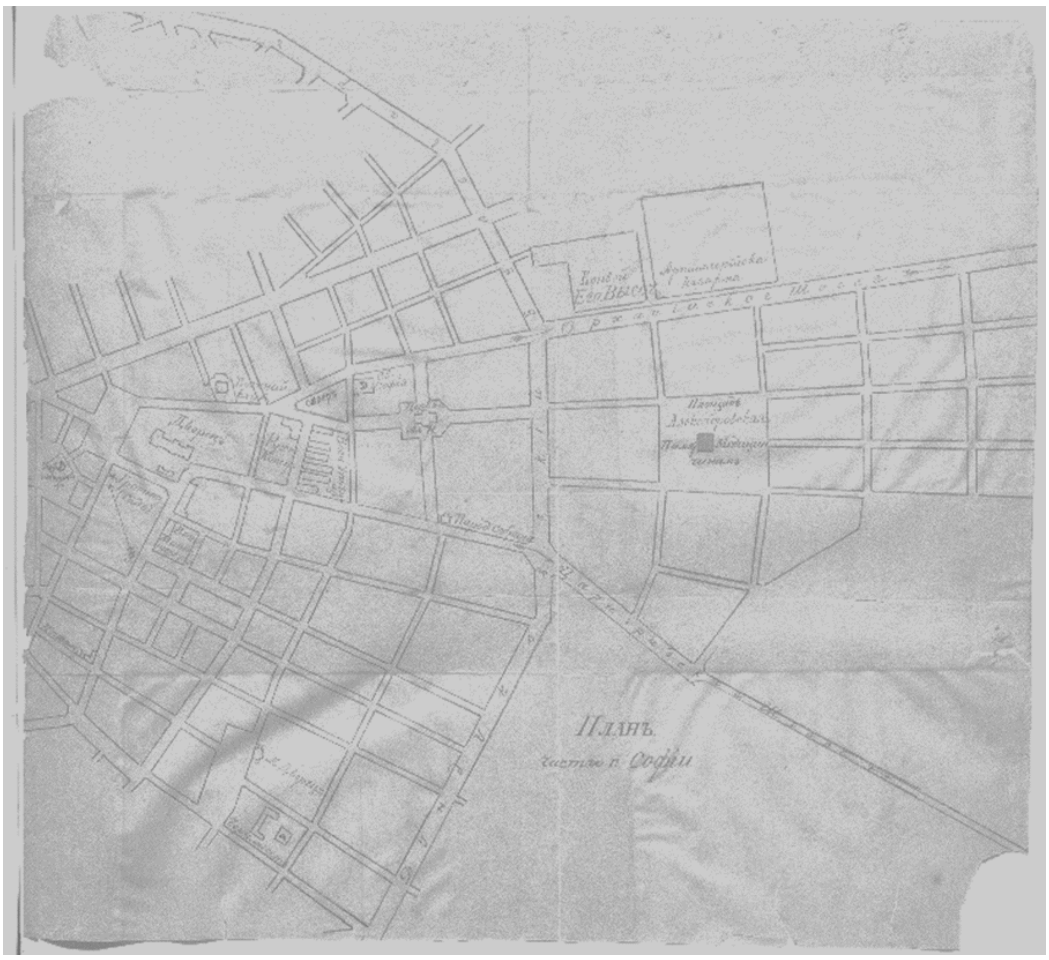
И така, според договора за построяването на паметника е избран площад в новата част на София и градският съвет отстъпва мястото на паметника безплатно. Това се случва между края на 1882 и началото на 1883 г., съдейки по писмото на завеждащия инженерното отделение Копиткин до военния министър на Русия.

Хандрисът от Руския държавен исторически архив показва една много любопитна гледна точка към мястото на строителството на болницата и паметника. В свое писмо от 9 февруари 1885 г. военният министър княз Михаил Кантакузин пише до началника на Военно-медицинското направление в Санкт Петербург Орест Рудински, че паметникът е завършен на 27 април 1884 г. и при негово желание може да му изпрати фотографии. „Заедно с това имам чест да препратя на Ваше Превъзходителство обратно два чертежа на паметника, изготвени от академик Томишко, предадени ми във Военното министерство на 8 юли 1882 г. под № 164.“ [31]

В тази преписка е запазен е само единият чертеж на Томишка. На него е посочено мястото, където сега са намира Докторския паметник. Отбелязано е името на площада „Александровски“ и наименованието −„Памятник медицинским чинам“ [32]. Разположен е между „Орханийско“ (сега бул. „Янко Сакъзов“) и „Цариградско шосе“. Отвъд Орханийското шосе са „Конвоят на Негово височество“ и „Кавалерийските казарми“ [33]. На хандриса е отбелязано и мястото на църквата „Св. София“, храм-паметника „Св. Александър Невски“ [34] и Народното събрание [35]. Сегашният булевард „Васил Левски“, или радиалната на широкия център, е отбелязан като „Дундуковски булевард“. Мястото на паметника на Васил Левски не е отбелязано [36], тъй като той вероятно е бил извън неговата задача. От скицата личи, че с отбелязаните сгради е потърсена връзката по оста до Двореца като централна сграда в столицата. Отбелязването на сградите на Народното събрание и на храм-паметника „Александър Невски“ показва, че вероятно тя е трябвало да илюстрира неговото местонахождение пред Военното министерство в Санкт Петербург.

4. Договор за строителството – технологии и материали

Договорът за строежа на Докторския паметник е сключен на 22 декември 1882 г. между Военното министерство на Княжество България в лицето на инж. Николай Копиткин и предприемача Луиджи Фарабоско. Според него последният се задължава да построи паметника по чертежа на Томишка и приложения модел. Моделите представляват половин фасада, половин разрез план на съоръжението. Сумата за строителството е 15 хиляди кредитни рубли. Договорът хвърля светлина върху зидариите, разтворите и материалите. Вътрешността на паметника да бъде изградена от дялан камък, иззидан с плътно разтворен варов разтвор с две части пясък. За крепеж се използват железни връзки (анкери) с размер 1х5 см, а също и с дължина 1 м. Облицовката да бъде от избран бял ташкесенски камък [37]. Дълбочината на камъните от облицовката (в случая квадрова зидария) да бъде не по-малка от 50 см, която е необходимо заради закрепването. За зидарията е посочено, че стъпаловидното разполагане на облицовъчните камъни от всеки по-нисък ред трябва да бъде 20 см. Подливането им да бъде с портланд цимент Робенс с примес 2 части пясък, а разливката да бъде с цимент с примес от една част пясък.



Фиг. 8. *Хандрис с посоченото място на паметника, РГВИА, ф. 546, оп. 1, д. 95, л. 165.*

Саркофагът, който увенчава композицията да бъде от сив гранит с прецизна обработка и полиран. Като цяло във всички точки на договора е споменато, работата да бъде извършена прецизно с фини контури и чистота. Символът на медицината, който многократно е обсъждан в документите е запазен като присъствие и в договора специално е поставено изискването да бъде от бронз, а също така и венците, които са по два в долната част на всяка стена на пирамидата. Към финансовото осигуряване на договора е необходимо да посочим и задължителната гаранция от 1500 лв., което е 10% от стойността на предприятието и се внася от предприемача с цел да спазва стриктно условията по договора. [38]

5. Опазване на движими произведения

Макети на Докторския паметник се съхраняват във колекцията на Военно-медицинския музей в Санкт Петербург. Те не се отнасят към разгледания исторически период и са били създадени за популяризирането му.

6. Заключение

Докторският паметник в София е уникален като архитектурно съоръжение. Неговата мемориална семантика е постигната благодарение на използването на древната тектонична форма на обелиска. Композицията му е триделна, подобно на ордерното членение и се състои от база, пирамида и антаблеман, увенчан с фронтон (капакът на саркофага). От своя страна саркофагът също е триделен, основан на

логиката на античния ордер и има база, колони и съответно, фронтон. По този начин той се превръща в класическо съоръжение, а увенчаващият го саргофаг – в символична мощехранителница на разпилените по пътищата на битките тленни останки на медиците

Благодарности

Към Руския държавен военно исторически архив за предоставените копия на документи, които за първи път се публикуват в България. Основната част от изследването е публикувана в книгата, съдържаща и получените документи „Докторският паметник в София. Изследвания и документи. С., 2023, Матадор-74 с автори Благовеста Иванова, Елена Бугарчева, Нина Дюлгерова, Радосвета Кирова, Радослав Симеонов.

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[4] Crimean War Memorial. https://www.netley-military-cemetery.co.uk/hospital-buildings/the-crimean-war-memorial-and-the-officer-s-mess/ (достъп 5.03.2024).

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[6] The National Heritage List for England. https://historicengland.org.uk/

[7] https://depositphotos.com/photo/medical-heroes-monument-bucharest-romania-135464140.html?ref=26681138 (достъп 5.03.2024).

[8] Военни паметници в административна област София. С., 2021, Министерство на отбраната, с. 46, под номер 140.

[9] Създаден е като паметник на руските войни, а не като паметник на загиналите медицински чинове, както първоначално е замислен. След Първата световна война територията остава в Турция и е разрушен.

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<https://viewer.rusneb.ru/ru/000199_000009_003838489?page=451&rotate=0&theme=white>(достъп 5.03.2022).
[13] При издирването беше установено, че става въпрос за Пьотър (Пётр) Ионович Губонин. Изводът за неговото потенциално участие при строителството е било свързано с финансирането, от което, както става ясно от текста, той се отказва. Пьотър Губонин е известен като меценат и строител на църкви и железопътни линии. Свързан е с издигането на параклиса в памет на загиналите в битката за Плевен руски гренадери. Проектант на параклиса е арх. Владимир О. Шервуд. През 1887 г. той е построен в Москва, след охлаждането на отношенията между България и Русия. Гранитните детайли за него са изработени в ателието на Губонин. Вж. Боголепова пустынь — Сайт Успенского храма.
<http://bogolepovapustin.ortox.ru/?p=1170>(достъп 5.03.2022).
[14] Храм-памятник у полите на Балкана за вечното възпоменание на православните войници, паднали в Руско-турската война през 1877-78 г. Прев. от рус. Военен сборник кап. Чаракчиев. Приложение към в. Известия, бр. 67, 1902, с. 6. Вероятно става въпрос за Аманд Егорович Струве (1835-1898), инженер, мостостроител, който през 1873 г., един от учредителите на „Дружеството за съдействие и разпростанение на удобствата за живота в град Ялта“ („Товарищество для содействия и распространения удобств жизни в городе Ялта“) и работи съвместно в това дружество с предприемача Петър Губонин и от 1879 г. е генерал-майор. За него като инженер вж. и Печёнкин, И. Е., О. С. Шурырина. Купол „Метрополя“ и другие. К истории перекрытии в отечественной архитектуре рубежа XIX-XX веков. – В: IV Ханмогамедовские чтения Материаалы научной конференции. М., СПб., 2020, с. 149-150.
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[17] РГВИА, ф. 546, оп. 1, д. 95, л. 3-4.
[18] РГВИА, ф. 546, Оп. 1, д. 95, л. 31–47
[19] РГВИА, ф. 546, оп. 1, д. 95, л. 31–47.
[20] Леонтий Николаевич Бенуа (1856-1928) по това време (1879) е току що завършил Художествената академия в Санкт Петербург със златен медал и благодарение на своя тъст е получил като зестра прочутата картина на Леонардо да Винчи, станала впоследствие известна като „Мадона Бенуа“ (сега в Ермитажа).
[21] Наличието на мото означава, че проектите са анонимни.
[22] РГВИА, ф. 546, оп. 1, д. 95, л. 31–47.
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[26] Генерал-инспекторът обединява санитарното управление на всички ведомства. Под негово разпореждане е цялата гражданска и военна медицинска организация, която се намирала под ведомството на министерствата на вътрешните дела и на войната и се оглавявала от главни лекари. Той има права на министър — назначава се направо от княза, лично му докладва по въпроси, свързани със санитарните дела, надзирава цялостната санитарна дейност в страната, взима участие в Министерския съвет по санитарни въпроси, издава заповеди, по право е председател на Медицинския съвет, следи за спазването на медицинските закони и разпоредби, има право да прилага при нужда хигиенни и медико-полицейски мерки, да извършва ревизия на здравни заведения и да издава прикази в сферата на здравеопазването (според чл. 1–12 на съставените от него Граждански медицински закони). Вж. Божилова, К. Документално наследство на Главна дирекция на народното здраве. Електронна библиотека по архивистика и документалистика. <https://www.electronic-library.org/articles/Article_0192.html>(достъп 7.03.2022).
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[29] Става въпрос за мястото, където сега се намира.
[30] РГВИА, ф. 546, Оп. 1, д. 95, л. 96–98.
[31] РГВИА, ф. 546, Оп. 1, д. 95, л. 134.
[32] Площад „Цар Освободител“, според плана на София от 1892 г., където в легендата под номер 35 е отбелязано „Паметник на загиналите доктори през Руско-турската война“,
[33] По плана на Йохан Бартел от 1897 г. са „Лебгвардейски казарми“ и „Артилерийски казарми“. Именно този градоустройствен план в случая е използван за сравнение, тъй като в него са поставени
[34] Основният камък е положен на 3 март 1882 г.
[35] Архитектът Константин Йованович предлага проекта си на 7 май 1884 г. Основния камък е положен на 25 ноември същата година.
[36] Основите на паметника на Васил Левски са поставени през 1878 г., работата след прекъсване е подновена през 1884 г. и той съществува на плана на Бартел от 1897 г.
[37] Мястото на добиване на материала е днешното село Саранци.
[38] РГВИА, ф. 546, оп. 1, д. 95, л. 96–98.

DESTRUCTIVE EARTHQUAKES AND RELATED ADVERSE EVENTS

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Abstract – The paper lists some very important earthquakes in the world, and related adverse events. The impact on the specific structures and buildings, such as nuclear power plants and bank revetments was discussed. The examples of the tsunami and hurricane impacts were mentioned. Also mentioned were liquefactions, mechanism, calculation of safety coefficient and lateral spreading.

Keywords – Kobe earthquake, P-Δ effect, Liquefaction, Lateral spreading, tsunami, flood, hurricane, Vrancea earthquake, Mexico City earthquake, Loma Prieta, Response spectra, negative friction, sensitive clay.

1. Introduction

Earthquakes are by their nature stochastic processes and it is very difficult to precisely determine the intensity and time of their occurrence, but rather the probability of occurrence is provided through a certain return period and a seismic map (of hazards) on a specific location. Understanding of earthquakes was considerably improved by the discovery of tectonic plates, their movement and collision borders. Many researchers dealt with earthquakes or their various effects on structures, such as: Aničić, [1] Dowrick [6], Baker and Cornell [2], Ćosić and Brčić[3], Duan and Chen [7], Elnasai and Di Sarno [8], Folić, B.[12], Hrasnica [15], Janković [16], Petronijević and Schmid [22], Kwong and Chopra [17], Meymand [18], Milović and Đogo [20], Newmark and Rosenblueth [21], Schnabel, Lysmer and Bolton Seed [23], [9] etc. The places where the tectonic plates meet or collide are called faults, and they are usually situated in the oceans or on the edges of continents, and there are various motions of plates. One of the most famous faults is San Andreas, which is a part of the Great Pacific Ring of Fire, is located in North America and it cuts across California and it is characterized by the shearing horizontal plate motion. The name Ring of Fire comes from the presence of a large number of volcanoes on the ocean's rim. The east part of this ring constitutes the Andes in South America, which are the result of the subduction of the Nacza (neighbor Pacific) plate under the rising Andes. In Middle America there is the Cocos plate, whose motion in 1985 caused a relatively moderate earthquake, and its effects were catastrophic for certain buildings in the Mexico City. Indonesia, Japan and New Zealand are also a part of this Ring, and they are characterized by frequent and considerably intensive earthquakes. For instance, the earthquake in Japan 2011 was IX on the Richter scale. It is believed to have shifted the Earth's axis of rotation for around 10 to 30 cm.

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There is a fault between America, on one side, and Europe and Africa on the other side in the Atlantic Ocean, which passes through Iceland. For this reason, Iceland is very frequently exposed to volcanic eruptions. There is a difference between volcanoes which have global impacts and those with a local impact. One of the criteria for the global impact is the quantity and chemical compositions of ash and dust ejected into the atmosphere. The eruption of the Eyjafjallajökull volcano, on Iceland in April 2010, ejected so much ash that air traffic was cancelled. The contact of the water and lava in the grotto of a volcano can considerably increase the height and amount of the ejected ash, as well as the explosion intensity of the eruptions, like a pressure cooker. Volcano Santorini, around 1500 BC, caused a tsunami with the eruption, and the ejected ash cause a mini ice age, since there was not sufficient sunlight for a normal harvest. These eruptions were confirmed in two ways, by taking samples from old trees, interpreting the tree rings, and from the sampling of polar ice. The third way is from the geological composition of soil from the sediment layers, and it is used from the historically distant events.

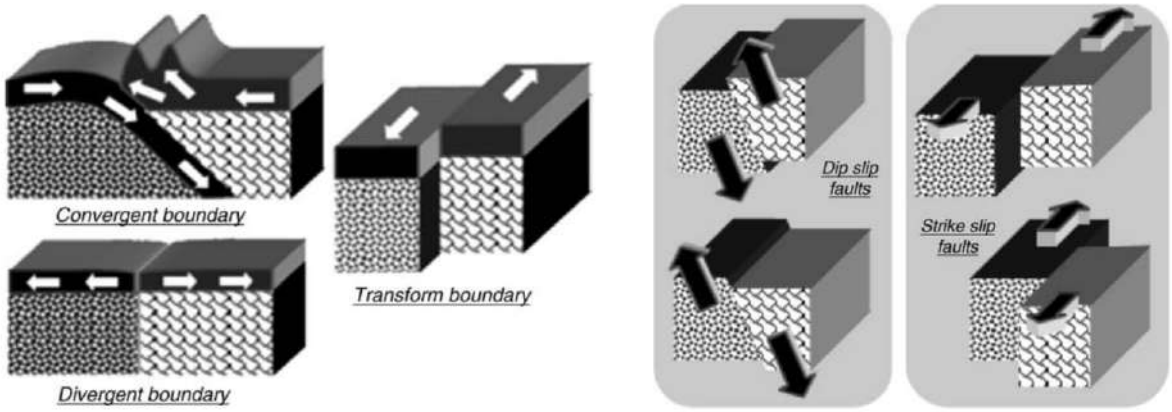


Fig. 1 Types of interplate boundaries. Types of fault, after Datta, [4]

2. Scales for description of earthquakes

Different scales are used to describe earthquake intensity, [1], [4]. One of the most frequent is the Richter scale and it has a direct physical measure. The measure of the earthquake intensity is Magnitude, defined by (in 1935, it is called the local magnitude M_L). It represents the value of displacement amplitude in a logarithmic scale with 10 as a base, and it is expressed in microns. The amplitude is measured on a standard seismograph Wood Anderson, at a distance of 100 km from the epicenter. There are other appropriate magnitudes defined by seismologists, such as the magnitude of spatial (P) waves M_b (Body Wave Magnitude, suitable for deep of earthquakes), surface waves M_S (Surface Wave Magnitude, suitable for measuring of shallow earthquakes distant over 1000km). These scales intensities only partially coincide, only in certain intervals, at that. Also used in geology is the physical parameter such as the seismic moment. There is a number of empirical formulas describing relations between energy of seismic waves and the energy in the focus, or the relation between the amplitude and the length of the fault. One of the most important measures of earthquake effects is the peak ground acceleration (PGA).

There are scales which do not describe the physical intensity of an earthquake, but rather its destructive effects and the damage on the structures such as the Mercalli scale or modified Mercalli scale or MSK (Medvedev Sponheuer Karnik), and seldom MCS (Mercalli Cancani Sieberg) is used. All three scales have approximately same intensity (12 degrees) [1] even though some authors consider them to be shifted for half a degree.

Destructive effects of earthquakes and related adverse events

There can be special problems when the earthquakes cause tsunami, floods and fires. An example of a flood is the earthquake Fukushima in Japan, on March 11th 2011. An example of the fire is the Kobe earthquake in Japan in 1995 or San Francisco in USA in 1906.

Tsunami and floods

.In the case of subduction, when one plate sinks beneath the other, when this vertical movement becomes sudden during a strong earthquake, it can cause a tsunami. A tsunami is relatively low at the open sea or ocean, it can be only a few meters, but as it approaches the coast, its height increases to tens of meters, which increases its destructive power. In Japan in 2011, there were protective walls against the tsunami, but in the vicinity of Fukushima, where a nuclear power plant with uranium and plutonium reactors is located, there was an overflow, aggregates and cooling pumps stopped operating, and the damage to the reactor occurred due to overheating. Citizens were evacuated due to the danger of radioactive contamination. On the northern part of the island, the tsunami protection walls were only slightly higher, but sufficient to protect the coasts of that part of the island. It should be noted that the walls collapsed (without breaking) in the vicinity of Fukushima, and this also contributed to the disaster caused by the tsunami.

Tsunami in some cases can have the same effects as a typhoon or hurricane.

The pumps also stopped working due to the overflowing of protective walls (due to the hurricane Katrina) in New Orleans. Since a large part of the city was below the sea level because of settlement and sinking, a part of the perimeter was protected by the wall, that is, bank revetments, however it was not sufficient and the pumps were submerged. The pumps should have pumped out the water from the low parts back into the sea, but they were incapacitated because of submerging, and the catastrophic floods ensued. After the disaster, the pumps and generators were elevated to a higher level, so that the floods would not submerge them and interrupt their operation.

Modeling of an optimal shape of breakwater in the hydrodynamic laboratory

An interesting example is a breakwater for a nuclear power plant in the north of Great Britain, construction is underway. The wall is made of prefabricated elements, and at the top it has a curved shape towards the sea, like a beak, so that the part of the wave that reaches the top of the wall returns back to the sea. It is possible to break the top of the next wave by this reversal if the periods of the waves interfere. The most favorable shape of the wall was tested on a model in the hydrodynamic laboratory.

Liquefaction

Liquefaction is a phenomenon known for centuries, but its mechanism was only explained in 1964 after the earthquake in Niigata, Japan. Namely, the images of overturned and practically undamaged buildings in Niigata in 1964, were deeply etched in the memory of everyone who saw them.

Liquefaction is a phenomenon that mostly occurs on poorly compacted soil, primarily sand or sandy soil that may contain a certain percentage of dust. Namely, during an earthquake, soil particles oscillate, so if the granular skeleton is weakly connected and submerged in groundwater, there may be a loss of contact between the particles, and the load-bearing capacity of the skeleton ceases, and the soil then behaves like a heavy liquid. Therefore, it can be said that the pore pressure of the ground water (due to the buoyancy force) overcame the shear resistance of the soil. The geotechnical and stress conditions with which the soil resists the effect of an earthquake is designated as CRR (cyclic resistance ratio), and the effect of an earthquake that tries to collapse the granular skeleton is called CSR (cyclic stress ratio). The CRR/CSR ratio gives us a factor of safety against liquefaction for a given earthquake.

A detailed method of testing of pile damage during liquefaction (Kobe) was provided in Tazoh and all. [24].

Negative friction the soil layers

Negative friction is a phenomenon when weakly bearing soil layers, instead of transferring the load from the deep foundation to the adjacent soil, hang on the deep foundation. In Kobe, a pile protection system against negative friction was applied, namely a pile driven into the ground, usually steel, coated with tar or special bitumen, along the places where negative friction is possible in the soil layers. This prevents mud or poor soil from "hanging" on the pile. This pile processing system patent was protected in Japan.

Sensitive clays

Sensitive clays were originally registered in Scandinavia, however, they are also registered in Canada and other parts of the world. It is the phenomenon that clays that have a good static bearing capacity, and are exposed to the effect of certain vibrations, suddenly lose their bearing capacity. These vibrations can be caused by passing trains, not necessarily by an earthquake. The phenomenon is similar to liquefaction, but it is not necessary for the clay to be saturated with water. Also, with sensitive clays and silty soils, errors may occur during pile construction (Milović and Đogo, [20]) which compromises stability and safety of such structures.

Mexico City 1985, September 19th

At the epicenter, it was tge magnitude 8.1 earthquake. The consequences of a relatively weak earthquake (COCOS plate) on Mexico City in 1985, about 400 km away, were catastrophic in certain locations ("Lake Zone"). Namely, the filtering of the earthquake through the soft layers of the soil resulted in the extension of the period of predominant oscillations of the seismic waves. The collapse of high-rise buildings in Mexico City occurred as a result of the structure-ground interaction, namely under the city there are several locations where there were once lakes or ponds, which were buried due to the expansion of the city and the need for construction land. These locations are an additional problem not only because of refraction and dispersion, but because of the reflection of seismic waves in the soil deposit from the rigid boundary. This is similar to a standing wave and self-resonance. The influence of such soft soil layers on tall buildings in the response spectrum is represented by Mexico City as an extended plateau [13], [14], [19], [18] and [1].

Earthquake Kobe, Hyogoken-Nanbu 1995 (Hansin-Awaci), 17th January

In Kobe there is a number of sites with layers of varying thickness of soil deposits, and one major fault that has merged with a nearby local fault. The port of Kobe was expanded over time by sand filling, so in 1995 liquefaction occurred in a number of locations. The number of victims during and after the Kobe earthquake is not only the result of engineering "errors" in important buildings, but the confluence of a series of factors, which simply converged at a given place and time. Therefore, the "errors" were the result of a lack of knowledge, that is, the engineering structures that were built at the time, were made at the highest level of the technical-technological development of the time, but after a couple of decades, they were surpassed by the progress in these fields. Therefore, irrespective of Japan's wealth, the scientific and technological progress could not be observed by the demolition of capital buildings and the construction of new ones, in accordance with innovative regulations and new knowledge. A brief review of the global economy: after the malfeasance of the American FED which abolished the gold standard, as early as in 1972, the leadership on the planet was taken over by Japan as the real economy, in a period of about two decades. Therefore, regardless of such economic power of Japan, capital buildings could not be demolished just like that, rather they are possibly strengthened, if there is knowledge about such needs. Let us note, that it is often the case in the USA that a certain type of structure, e.g. prefabricated bridges with steel pendulum for connection cantilever and simple beem, was systematically and extensively inspected only after the demolition of a representative structure. And if it is determined that such a system is very difficult to control and maintain, or that it is impossible to protect it with additional measures, then its further use is discontinued.



Fig. 2 Damage due to the loss of support (Kobe, 1995, after [18])

The number of casualties, which was relatively large solely due to the traditional Japanese houses construction was not immediately available to the professional public. Namely, Kobe is an elongated port city constrained between the Rokko hill on one side and the sea, i.e. Osaka Bay on the other side, so the residential quarters are expanded on the outskirts. It is the seventh largest city in Japan, and had about one and a half million inhabitants. Due to the narrowness of the city center (about 5 km wide), the highways were elevated on piers and turned into long overpasses or bridges, and the soil was filled in and reclaimed from the sea. At that time, there were about 160 km of elevated highways on piers in Kobe, and quay walls and artificial islands were also built.

Night before the main shock of Kobe 1995

On 16th January at 18:26 h, seismic activity of 3,6º Richter was registered, which activated the sensors, but which is almost imperceptible to people. At 18:49 h an even smaller seismic activity of 2,5º Richter, was registered that activated sensors again. Sensors located the focus of the fault 15 km southwest of Kobe, in the Akashi channel separating Awaji island from the mainland. At 18:55h a third tremor is registered, an even smaller seismic activity of 1,5º Richter. At 23:49 h there was fourth tremor. On Tuesday, 17th January at 5:46 h, the main shock of the earthquake hits the area (M 7.2-7.3-7.5º).

In Kobe, there is a fault that runs almost parallel to the coast. Until 1995, the faults on the island of Awaji and in the city of Kobe were known, however, the fault in the channel, which was then activated and which connected these two faults, was unknown until then, so these faults were considered exclusively as separate phenomena. The intensity of the earthquake was 7.5 magnitude on the Richter scale, but the fault and the source were at a depth of only 17 km, directly below the city.

If we assume that the fault in Kobe was approximately 20 km long, on the island 15 km, and in the channel 5 km, and we calculate the seismic moment, assuming that the height of the fault surface and the shear modulus are the same, we get an increase with l (length) squared, so the seismic the moment grows from a length of 20 to 40 km. The increase is (40/20) squared, which is 4 times the original. We stress that the length of the San Andreas fault is about 7000 km, so the previously assumed length is very small. If the expression for the seismic moment is used, it should be compared with the relationship between the length of the fault and the magnitude, and it should be taken into account that there are formulas related to certain locations, and therefore they may differ (M~ 6.5 to 7).

The residential houses were made as a wooden or bamboo frame without proper braces or ties, and for defense against typhoons, heavy traditional roof tiles were used, which cannot be blown away by the strong wind (suction effect). Due to the P-Δ effect, and due to the lack of horizontal support, these heavy roof tiles fell during the earthquake and caused the greatest number of casualties. Wooden houses also caught fire, the fires were caused by fire from the hearth and failure of the installations. The finite number of fire engines could not adequately respond to numerous fires. The earthquake caused 140 fires in the suburbs, out of a total of 5,500 casualties (there are more data on the total number of casualties, the figure ranges from 5,052 to 7,000), about 4,900 were in the suburbs. Kobe is a predominantly port city, one of the largest in Japan, and was organized as a large distribution center for goods (receiving and shipping), so help was slow to arrive in such a city when the port was badly damaged. This, regardless of the efforts of the citizens, increased the number of casualties.

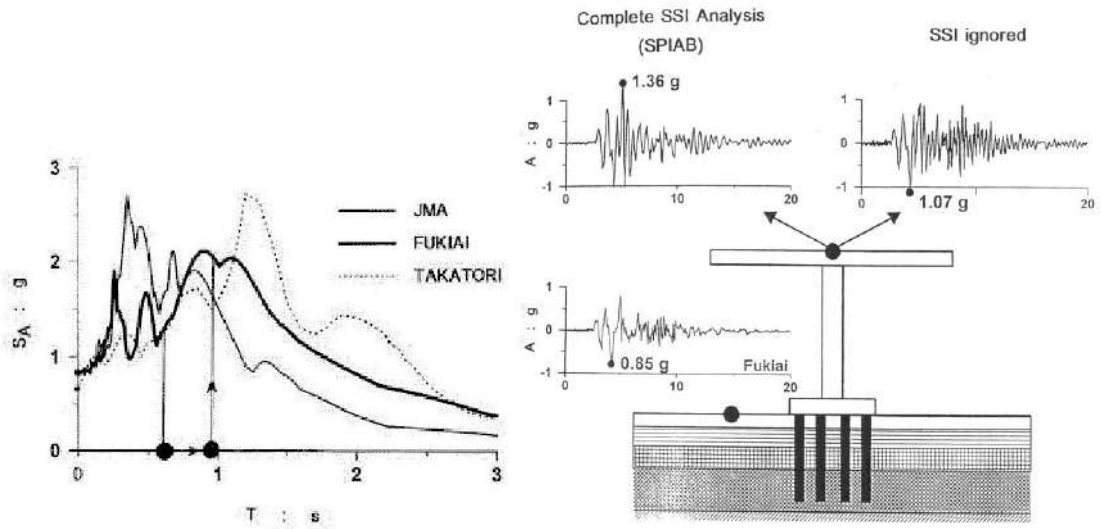


Fig. 3 Response spectra of different locations in Kobe. Accelerograms with and without the interaction of soil and structure, after Mylonakis and Gazetas 1998 [13])

The increase in the period of the seismic response of the bridge-column-piles system occurred primarily because the surface layer of the soil, about 3.5 m thick, has extremely soft characteristics (SPT, N from 5 to 20 shocks), so due to the interaction with the soil, it was necessary to strengthen the horizontal tension reinforcement (stirrup) due to the increase of the required ductility.

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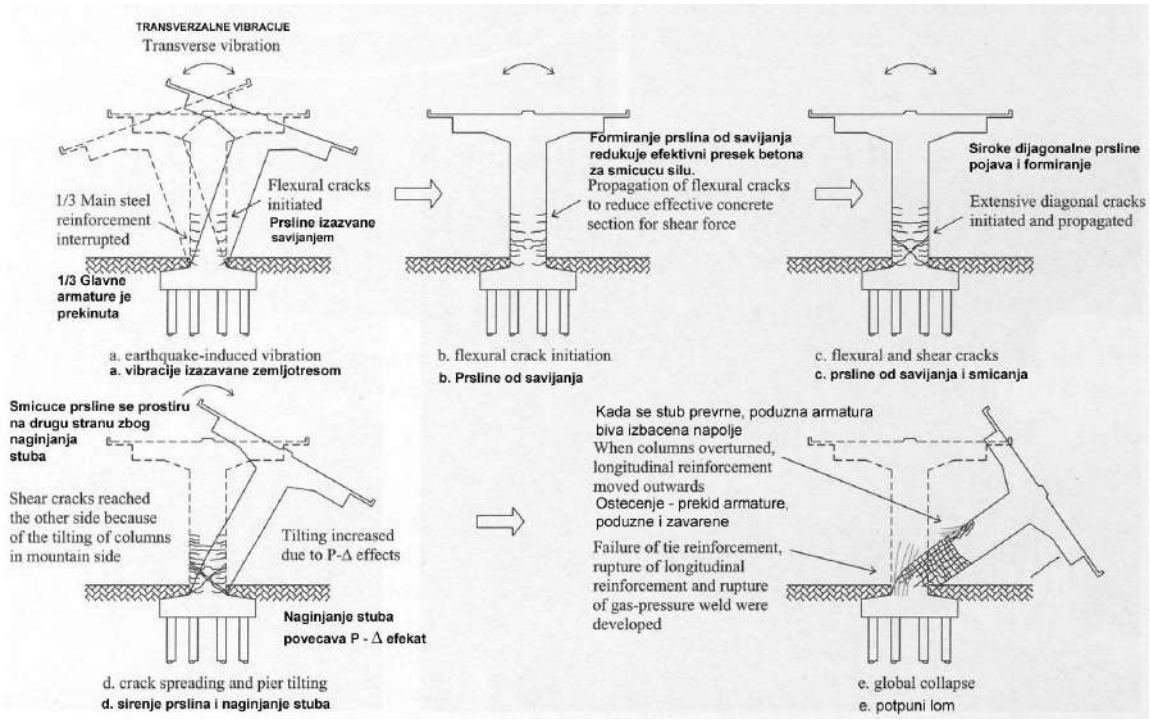


Fig. 4 Mechanism of the collapse of Hansin expresses way columns **(**modify after Elnashai A., Di Sarno L., 2008, [8])

The port was largely disabled, and aid deliveries did not begin until the third day. Some buildings were built according to regulations from the 60s, so they did not have seismic resistance like the latest buildings. After the 1960s, Japan improved regulations several times, and also after the earthquake in Kobe in 1995.

It should be noted here that the traditional architecture of the wooden pagoda had a very high resistance to the effects of earthquakes, firstly because of the rigid wooden core, secondly because of the support system of wooden roof consoles that dissipate seismic energy by relative movement, and thirdly, just as importantly, the characteristic period of pagoda oscillation does not coincide with the dominant periods of earthquakes in that Asian region. Also, the woodwork of the structural elements and supports of the wooden materials are done in such a way that ventilation and conditions of constant low humidity are enabled, so that there is almost no occurrence of rot, drying or other diseases.

San Francisco earthquake 17th October 1985

The 1985 San Francisco Earthquake on October 17th, shortly after 5 p.m., the San Andreas Fault was triggered. An overpass has collapsed in Oakland. On the bridge in San Francisco, one span collapsed at the pier. The mobility of the bottom of the bay was higher than (long ago) assumed. One of the main causes of extensive damage are the consequences of the 1906 earthquake. Namely, during the earthquake of 1906, on April 18th, a fire broke out in San Francisco, which lasted for days. Due to the poor living conditions, a large number of residents moved out. In order to revive the city, the city officials filled in the surrounding ponds and swamps, which gave them new development land for the construction of housing and related facilities. These were good sites for low wooden houses, but not for the tall reinforced concrete or steel structures that would later rise on those sites, or for the city harbor as shown in the figure below. According to Meymand [18], the filling up of San Francisco's harbor began in 1850, so the biggest damages then were on the land reclaimed parts of the harbor. Let us remember that in the 19th century, the "gold fever" raged in California, so the tailings from excavations were used to backfill the port, as well as sand, stone, etc. More detailed investigation revealed 20 (about 23) different ways of backfilling the port of San Francisco.

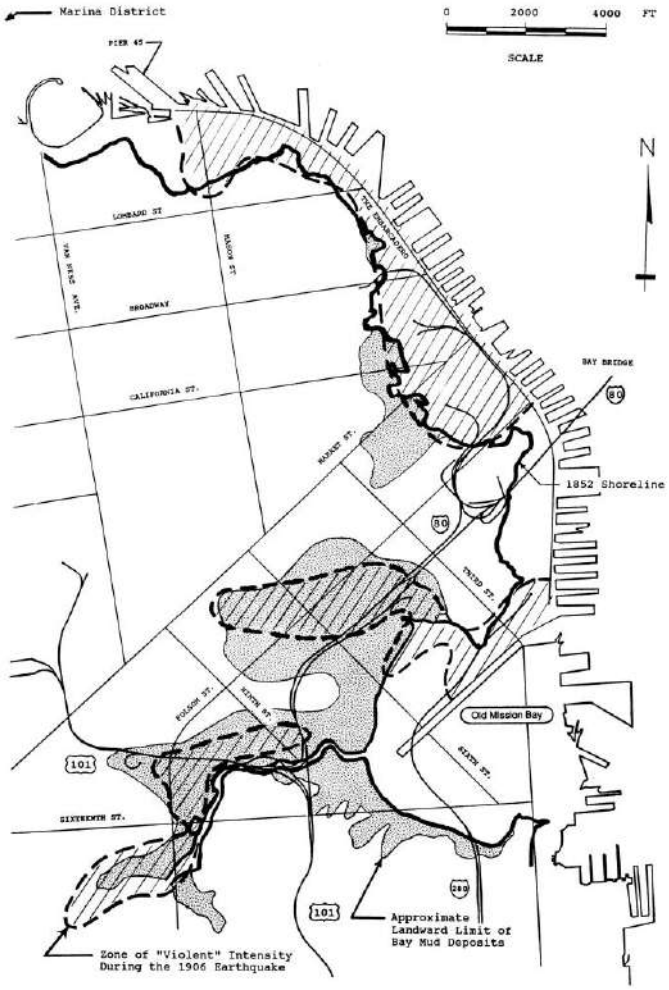


Figure 5 - Regions Most Intensively Damaged During the 1906 San Francisco Earthquake, and the Historic Shoreline (after [18] Meymand, 1998)

Los Angeles, Northridge 1994

At 4,30 in the morning of 17th January 1994, the valley of San Fernando, near city of Los Angeles was hit the Northridge earthquake, with a magnitude of 6.7 on a Richter. On high-rise buildings, the steel welded joints of beams and columns have shown their weaknesses during earthquakes. This type of joint was widely used in California from 1964 to 1989. Due to the 1985 earthquake, San Francisco was also inspected, and only 6 damaged towers were found. These inspections followed after it was established that during welding the workers were sitting on the beams, and that due to their awkward position, they had a much higher number of breaks during welding, thus also interrupting the welding. The joint would not be continuous and of high quality as in controlled conditions. In places where the welding seam is continued, and if it is not done properly, cracks may occur.

Loma Prieta Earthquake 1989 October 17th

During the Loma Prieta Earthquake, magnitude 7, in 1989, there was also collapse and damage to a number of structures, such as damaged batter piles at Port of Oakland and Port of San Francisco. The picture below shows the collapse of the crossing in the valley, the alluvial deposit. Alluvial soil deposits usually have poor resistance to lateral earthquake action. The static resistance was satisfactory, but due to large displacements, the piles in contact with the bridge failed, and the road slab was punched through by the tops of such piles. (Meymand, [18])



Fig. 6 Horizontal displacement of pile tops due to earthquakes and loss of contact with the soil in the upper layers. Highway 1 Crossing Struve Slough near Watsonville Collapsed during the 1989 Loma Prieta Earthquake. (after [11], [18]).

Showa Bridge Niigata 1964

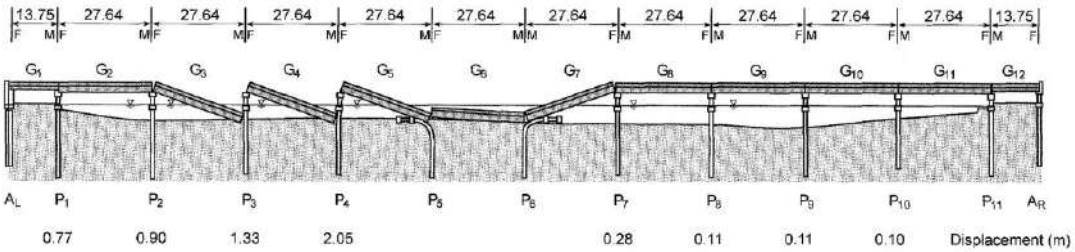


Fig. 7. Elevation or the figure represent horizontal displacement of the ground at each pier, after Yoshida, et al.[26])

Showa bridge collapsed during the Niigata 1964 earthquake in Japan, but the collapse started only around 60 (Meymand 1998, [18]) to 70 seconds (Yasuda 2007, [26]) after the main shock, and lasted around 17 seconds. Niigata Earthquake was the magnitude 7.3. Most likely, the collapse was due to the horizontal soil expansion, but let us see what is the account of Yoshida et all, i.e. JSCE, after [26]: 1) Horizontal movement of the left bank side ground of about 3 m towards the center of the river, 2) liquefaction of the ground, 3) horizontal external force worked on the bridge girder resulting from horizontal movement of the abutment on left bank side, 4) large horizontal displacement of the foundation resulting from round displacement, 5) reduction in lateral ground resistance due to liquefaction, and 6) loss of bearing capacity due to poorly welded pile joints. In this paper [26] a number of detailed explanations were provided about the collapse mechanism and the impact that the Showa Bridge in the course of the Shinano River was exposed to during the earthquake. Among other things, the method of determining the delay

of the collapse in relation to the main impact was explained, according to the statements of eyewitnesses, who were crossing the bridge at the time, or were in the immediate vicinity.

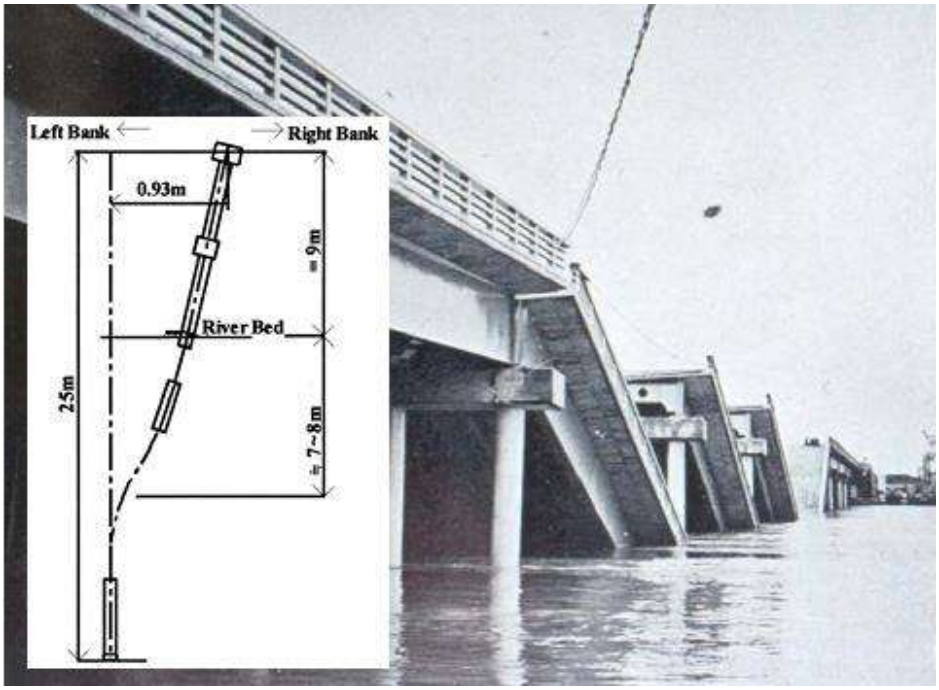


Figure 8. The failure of the 303,9 m long Showa Bridge in Shinano River was a direct consequence of liquefaction of the upper loose sand layer, which in turn laterally flowed against and displaced the 60 cm diameter steel pipe pile-supported bents, unsupported and dropping several spans. Niigata 1964. (after [11], [18]).

Response spectra

Response spectra are usually given for acceleration, or in combination with velocity and displacement, and "tell" us how much acceleration amplification is possible due to an earthquake at a specific location.

In relation to standard response spectra, Mylonakis and Gazetas 2000 [19], provide response spectra for very specific locations and earthquakes, such as Vrancea-Bucharest Romania 1977 [10], Kobe Fukiai 1995, Kobe Takatori 1995 and Mexico City 1985 [18]. Fukiai and Takatori are the same earthquake and the city, but two locations of measuring stations, which have a significant difference in the response spectrum. Earthquake directions are marked EW east-west and NS north-south. The same authors considered the interaction of structure-foundation-soil-interaction SFSI [13], as well as Gerolymos and Gazetas [14].

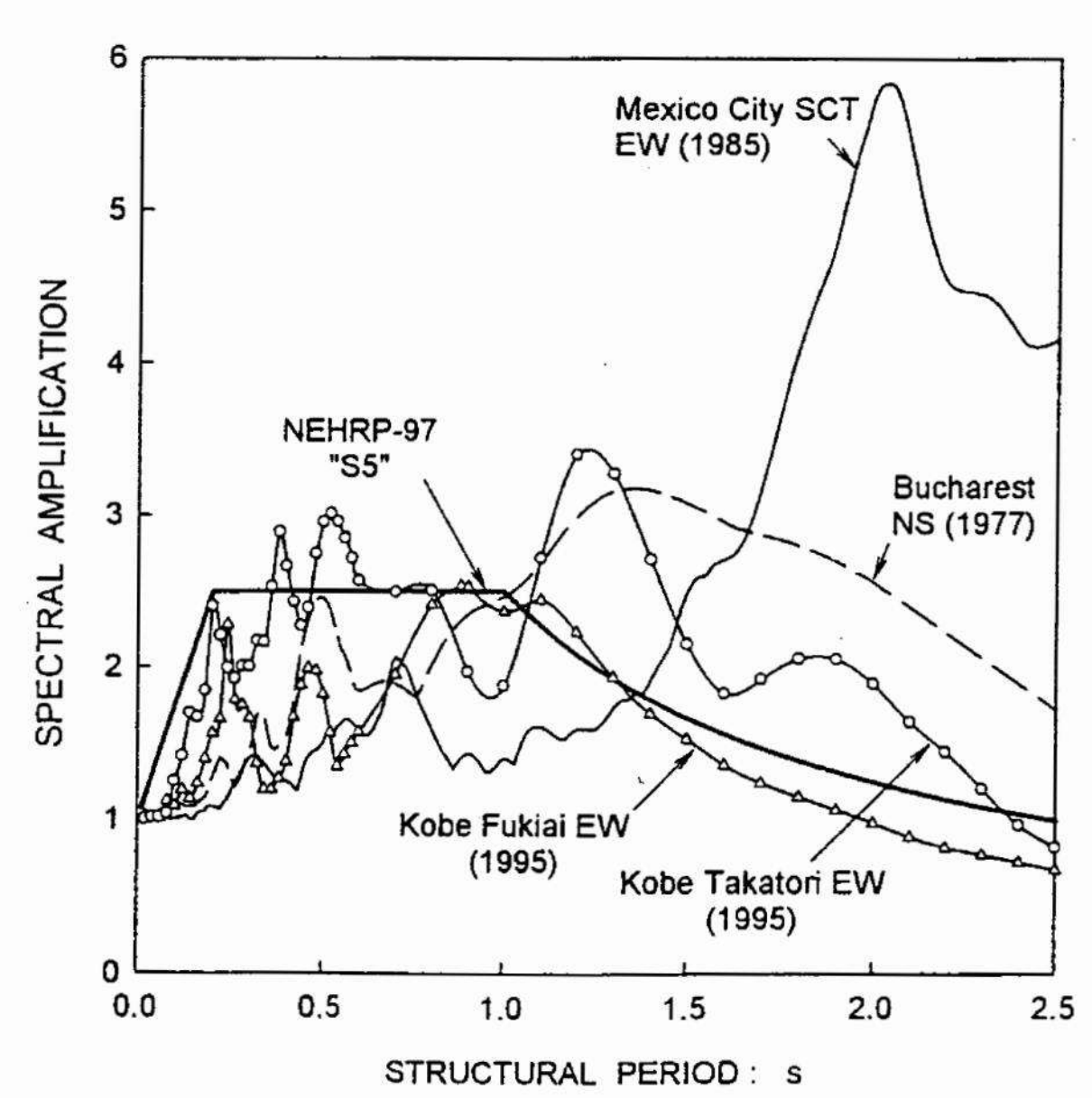


Figure 9 Comparison of a typical seismic code design spectrum to actual spectra from catastrophic earthquakes with strong long-period components; β = 5%, after Mylonakis and Gazetas 2000, [19].

6. Concluding remarks

When considering seismic effects on capital buildings, it is necessary to include in the concept of robustness the related adverse events as possible cases of loading, such as fires, floods, weather events, torrential rains, volcanoes, tornadoes, hurricanes and the like. Of course, one should take into account what specific phenomena can occur at the given location where the structure is located. However, integral design requires additional determination and provision of critical roads in the event of a disaster.

As far as the residential and office buildings are concerned, it is preferable, for the sake of evacuation, to make a rigid staircase core, which should be protected or surrounded by RC bearing walls. Regardless of whether pressurized air is pumped into the staircase or not.

What is noticeable is that due to climate change, all positions of equipment that regulate the movement of water or supply energy should be revised. In this case, those are pumps and power generators.

Regarding the interaction structures-piles-soil, it is interesting to note the lecture of Prof. Trifunac [25], where for the building of the famous hotel in Los Angeles, VN7SH, which was equipped with a suite of instruments for many years, he gives the example that after one strong earthquake, torsional tone appeared in the foundation at the ground level (for ambient soil-piles vibrations).

In the paper Folić et al. BASA 2022, various damages and their causes, as well as the rehabilitation of buildings were considered. In this paper, adverse phenomena that accompany earthquakes are discussed in more detail, such as: direct effects, ground failure, surface faulting or fault rupture, ground lurching, differential settlement, lateral spreading - flow failures, tsunamis, landslides, collapses, liquefaction, and other adverse events. Also, in BASA 2022, a very important article by McDonald, R., Introduction to Natural and Man-made Disasters and their Effects on Buildings, AP, Amsterdam, 2003, was presented, with a description of some specific damages, so they are not repeated here.

Acknowledgements

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EUROPEAN CULTURAL HERITAGE ON STAMPS – BRIDGES

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Abstract: *Information about the European bridges on the stamps within the project Europa stamps in 2018. Bridges as a part of European architectural scene. Modern bridges and very old or ancient bridges. Many of them are national cultural heritage of given country.*

Key words*: national cultural heritage, European bridges, stamps*

1. Introduction

The bridges were theme for EUROPA stamps in 2018 and there were many great looking designs from the many participating countries. The countries have chosen everything from the large modern bridges to the small older timber bridges. They are really worth a closer look. Bridges are a powerful element of the European architectural scene, which can reflect the cultural diversity present across the continent. Bridge symbolises the spirit of unity between nations. During writing this paper the names and locations of all bridges on the stamps were carefully identified by the authors. The photos of all bridges given on the stamps may be easily found in the Internet thanks to information given below stamp pictures. This fact enables to see also many bridges which are not on the stamps.

2. EUROPA stamps in 2018, theme bridges

Results of the voting for the most beautiful postage stamp EUROPA 2018 with bridge: a) Turkish Post PTT (Turkey - Gold), Fig. 31, bottom left; Poczta Polska S.A. (Poland - Silver), Fig. 30, left and Georgian Post (Georgia - Bronze), Fig. 15, top left, were winners of the 2018 EUROPA Stamps online competition through public voting; b) winners of the EUROPA Stamps Jury prize competition, selected by group of Philatelic experts were: bpost (Belgium - Gold), Fig. 8 left; Magyar Posta & Posti Group (Hungary & Finland - Silver), Figs. 17, top left and Hrvatska Pošta d.d. (Croatia - Bronze), Fig. 11, bottom left. There was a tie for the second-place, which went both to Magyar Posta (Hungary), Fig. 17, top left and Posti Group (Finland), Fig. 14, top center.

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Fig. 1. Bomarsund Bridge; **Åland**, autonomous and demilitarised region of Finland.

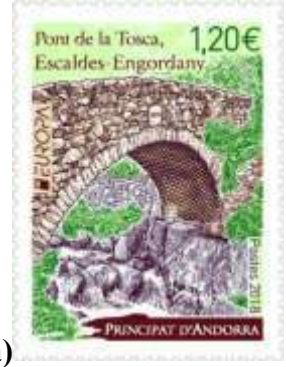


Fig. 2. Souvenir sheet. **Center:** Mesi Bridge over Kir river in Mes; **top left:** Kasabashit Bridge over Vlosh river in Çorovodë; **top right:** Hoshtevës bridge; **bottom left:** Tanners' Bridge over Lanë stream in Tirana; **bottom right:** Kamares Bridge over Skumbin river in Mirakë; **Albania**.

a) 

b) 

Fig. 3. a) Gorica Bridge over Osum river in Berat; **Albania; b)** Tsakkar natural bridge in Tsakkar; **Armenia.**

a) 

b) 

c) 

Fig. 4. a) Bridge of la Tosca; **b)** Pont d'Engordany; both bridges in Escaldes-Engordany Parish: **Andorra; c)** Josef von Schemerl Bridge over Danube Canal in Vienna; **Austria.**



a) **b)** **c)**

Fig. 5. a) Sultan Bridge from 1751 over Akhynjy river in Tovuz; b) Qanli Bridge from XIX. century, Gadabay; c) Lekedakh Bridge from XVI.-XVII. century, Julfa District; **Azerbaijan.**



a) **b)**

Fig. 6. Bridges on São Miguel island**: a)** Despe-te que suas Bridge; b) Bridge of 8 Arcs, Ribeira Grande; **Azores**, an autonomous region of Portugal.



a) **b)** **c)**

Fig. 7. a) Disna Bridge over river Disna, Vitebsk region; **b) and c)** Bridge over Svislach river in Minsk; **Belarus.**



Fig. 8. Left: Souvenir sheet. Pont des Tous over Scheldt river in Tournai; **Belgium**; **top right:** Roman Bridge over Bosna river near Ilidža; **Bosnia and Herzegovina; bottom right:** Šeher-Ćehaja Bridge over Miljacka river near Sarajevo; **Bosnia and Herzegovina**.



Fig. 9. Top left: Roman Bridge over Šujica river; **Bosnia and Herzegovina; bottom left:** Roman Bridge over Bosna river near Ilidža; **Bosnia and Herzegovina; top center:** Arslanagić Bridge over Trebišnjica river; **Bosnia and Herzegovina**; **bottom center:** Mehmed Paša Sokolović Bridge over Drina river; **Bosnia and Herzegovina; top right:** Kadin Bridge over Struma river at Nevestino; **Bulgaria; bottom right:** Covered Bridge over Osam river in Lovech; **Bulgaria.**



Fig. 11. top left: Stone Bridge, Novigrad na Dobri; **Croatia**; **bottom left:** Dr. Franjo Tuđman Bridge, Dubrovnik; **Croatia**; **top center:** Old Venetian bridge of Kelefos (or "Tzielefos"), Troodos mountain; **Cyprus; bottom center:** Akapnou Venetian Bridge, district of Limassol; **Cyprus; top right:** Değirmenlik (Kythera), Lefkoşa District; **North Cyprus; bottom right:** Ortaköy, Lefkoşa District; **North Cyprus.**



a) **b)** **c)**

Fig. 12 a) Podolsko Bridge and Chain Bridge both over Vltava river near Podolsko in **Czech Republic; b)** Arch bridge; **c)** Stone bridge, both in Tartu; **Estonia**.



Fig. 13. Souvenir sheet. Great Belt Bridge crossing the Great Belt strait between the Danish islands of Zealand and Funen; **Denmark**.



Fig. 14. top left: Streymin Bridge conects Streymoy and Eysturoy islands; **Faroe Islands; bottom left:** Bridge over Sandá river between Tórshavn and Argir, island Stroymoy; **Faroe Islands; top center: Finland; bottom center: Finland**; **top right:** Passerelle Mimram over Rhine river, bridge of two banks between Strasbourg and Kehl; **France; bottom right:** Kniebrücke over Rhine river, Düssledorf; **Germany.**



Fig. 15. top left: Besleti Bridge (Queen Tamar's Bridge) at Sukhumi; **Georgia; bottom left:** Bridge of Peace over Kura river, Tbilisi; **Georgia; top center:** Montagu Bridge; **Gibraltar**; **bottom center:** Windsor Bridge; **Gibraltar; top right:** Corral Bridge; **Gibraltar; bottom right:** Lanport Bridge; **Gibraltar,** British Overseas Territory.



Fig. 16. top left: Plaka Bridge over Arachthos river, at the borders of Arta and Ioannina prefectures; **Greece; bottom left:** Rio–Antirrio Bridge (Charilaos Trikoupis Bridge) crosses Gulf of Corinth near Patras; **Greece**; **top center:** Renovated bridge over the river in Qaqortorq; **Greenland; bottom center:** Glacier bridge; **Greenland,** autonomous territory of the Kingdom of Denmark; **top right:** Bridge over St. Saviours Reservoir; **Guernsey; bottom right:** Castle Cornet Bridge; **Guernsey,** channel island, a self-governing dependency of the British Crown.



Fig. 17. Souvenir sheet. Top left: Megyeri Bridge over Danube river, Szentendre island; **top right:** Kőröshegy Valley Bridge; **In the selvedge of the miniature sheet**: the detail of Móra Ferenc Bridge over the river Tisza to the north of Szeged; **Hungary.**



Fig. 18. Left: Footbridge over Elliðaá estuary, Reykjavik; **right:** suspension bridge over Jökulsá river on Breiðamerkursandur; **Iceland**.



Fig. 19. Top left: Mizen Head Footbridge, Cork; **Ireland**; **bottom left:** Boyne Viaduct over Boyne river, Drogheda; **Ireland; top center:** Medieval **b**ridge over Metauro river**,** Fermignano; **Italy; bottom center:** Roman Stone Bridge (Ponte Pietra in Italian) over Adige river, Verona; **Italy; top right:** Havre Des Pas Pool Bridge, Saint Helier; **Jersey; bottom right:** Bridge over Queen`s Valley Reservoir; **Jersey,** a self-governing British Crown Dependency.



Fig. 20. Top left: Stone Bridge over Bistrica river, Prizren; **Kosovo; bottom left:** Bridge over Jugla river between Avoti and Upeslejas; **Latvija; top center:** Pedestrian Atyrau Bridge over Ishim river, Astana (Nur-Sultan); **Kazakhstan; bottom center:** Ballure Bridge; **Isle of Man** (Mann), a self-governing British Crown Dependency; **top right:** Stone Bridge over Peć Bistrica river in Rugova Canyon; **Kosovo; bottom right:** Bridge over Gauja river; **Latvija.**



Fig. 21. Left: Old Rhine Bridge, Vaduz; **Liechtenstein; center:** Sainte Dévote Bridge; **Monaco**; **right:** Balzers Footbridge; **Liechtenstein.**



Fig. 22. Top left: Trakai Island Castle Bridge; **Lithuania; bottom left:** Bascule Castle Bridge (Charles' Bridge), Klaipeda; **Lithuania; top center:** Grand Harbour Breakwater Bridge (St. Elmo Bridge); **Malta; bottom center:** Madliena Bridge; **Malta; top right:** Bridge of Jarvanes over Tumi river, near Tumi; **Republic of Artsakh (Nagorno Karabakh); bottom right:** Bridge of Karavaz (Khudaperin) over Arax river; **Republic of Artsakh (Nagorno Karabkh).**



Fig. 23. Top and bottom left: Bridge over Babuna river, Veles; **top center:** Deer Jump Bridge, Mogorche; **bottom center:** Gorenica Bridge, Debar; **top and bottom right:** Bridge over Gradeska River, Mariovo; **Macedonia (North Macedonia).**



Fig. 24. Left: Eiffel Bridge over Prut river, Ungheni; **Moldova; center:** Gorenica Bridge, Debar; **Macedonia (North Macedonia)**; **right:** Flying wooden bridge over Raut river at Pohorniceni, Orhei district; **Moldova.**



Fig. 25. Top left: Bridge over Socorridos river, Câmara de Lobos; **Madeira,** an autonomous regions of Portugal; **bottom left:** Bridge over Ribeira da Metade valley, Faial; **Madeira,** an autonomous regions of Portugal; **top and bottom center:** Roman Bridge over former, today dried Moštanica river in Nikšić; **Montenegro; top right:** swing Draaibrug Bridge over canal in Dedemsvaart; **the Netherland; bottom right:** wooden drawbridge Kwakelbrug in Edam; **the Netherland.**



Fig. 26. Left: SNP Bridge over Danube river in Bratislava, **the Slovak Republic; Right:** The Hardanger Bridge across the Eidfjorden branch of the main Hardangerfjorden in Vestland county; **Norway**.



Fig. 27. Left: Soaring Bridge over Moscow river, Zaryadye Park, Moscow; **Russia; Right:** Maza bridge in San Vicente de la Barquera, Cantabria; **Spain.**



Fig. 28. Top left: Vasco da Gama Bridge over Tagus River in Parque das Nações in Lisbon; **Portugal; bottom left:** Trajan's Bridge over Tâmega river in parish of Santa Maria Maior, Chaves; **Portugal**; **top center:** Anghel Saligny Bridge (King Carol I Bridge), is a complex of two railroad truss bridges over Danube River, Cernavodă; **Romania**; **bottom center:** Bridge of Lies in Sibiu, Transylvania; **Romania; top right:** Gazela Bridge over Sava river in Belgrade; **Serbia**; **bottom right:** Žeželj Bridge over Danube river in Novi Sad, Vojvodina, **Serbia.**



Fig. 29. Top and bottom left: Fontevecchia Viaduct (Valdragone railway bridge), Borgo Maggiore; **San Marin**o; **top center:** The Cobblers' Bridge or the Shoemakers' Bridge over Ljubljanica river in Ljubljana; **Slovenia; bottom center:** Puch Bridge over Drava river near lake Ptuj; **Slovenia; top right:** Spreuer Bridge over Reuss river in Lucerne, **Switzerland; bottom right:** Žeželj Bridge over Danube river in Novi Sad, Vojvodina, **Serbia**.



Fig. 30. Left: Bridge over Vistula river in Tczew; **Poland; right:** Øresund or Öresund Bridge across Øresund strait between Denmark and Sweden; **Sweden**.



Fig. 31. Top left: Double Bridge **(**Çifte Bridge) is the name for two small adjacent historic bridges in Artvin Province. They span two small rivers, Kamilat and Soğucak, just above their confluence; **Turkey; bottom left:** Long Bridge **(**Uzunköprü, formerly Cisr-i Ergene), over the Ergene river in Edirne Province; **Turkey; top center:** Viaduct Plebanivka village near Terebovlya; **Ukraine; bottom center:** Bridge of Four Evangelists in Shenborna Park, Voivodeship; **Ukraine**; **top right:** St. Peter and St. Angelo Bridge or Ponte Sant'Angelo (originally Aelian Bridge or Pons Aelius) over Tiber river in Roma, Italy; **Vatican City; bottom right:** St. John Nepomuk and Charles Bridge over Vltava river, Prague, Czech Republic; **Vatican City.**

Conclusion

Bridges are structures built to span physical obstacles without closing the way underneath such as a body of water, valley, or road, for the purpose of providing passage over the obstacle. There are many different designs that each serve a particular purpose and apply to different situations. Designs of bridges vary depending on the function of the bridge, the nature of the terrain where the bridge is constructed and the material used to make it [1].

Terminology based on several ISO standards (e.g. on [2]) related to buildings and civil engineering structures including bridges and their parts, may be found in [3], where there are many photographs and schemes as well.

The present contribution creates perfect supplement of [3] for Slovak and foreign students at Department of Languages, Faculty of Civil Engineering, Slovak University of Technology in Bratislava. Students may try to identify type of bridges, their age shown on

the stamps. In addition, students will become familiar with the countries where the bridges are located. Some of these places and countries are practically unknown. Purpose of the paper is to help students and to inform designers in practice and students about admirable civil engineering structures built in the different eras.

During the lessons, some students showed to their classmates the bridges from their countries. Some students are thinking about collecting civil engineering structures on the stamps.

Acknowledgement
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HIGH RISE STRUCTURE AND TALL BUILDINGS IN BRATISLAVA SLOVAKIA

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***Abstract**: This paper deals with the development of high-rise buildings on the territory of the Slovak Republic. Examples of high-rise buildings from the past and present are briefly documented. In the case of current high-rise buildings, the load-bearing system and brief results from the verification of mechanical resistance are documented.*

***Key words**: High-rise buildings, Load-bearing system, Structural assessment*

1. History of the construction of high-rise buildings in Bratislava

The first high-rise building in Bratislava is Manderla, built in 1935-1936. The building was designed by architects Christian Ludwig, Emerich Spitzer, Augustín Danielis. It has 11 floors and a height of 45 m. The building was ordered by the Bratislava businessman and butcher Rudolf Manderla, who was inspired by the slaughterhouses in Chicago. In addition to its location in the city centre, the residential building is also historically important for housing leading figures of cultural and social life. The load-bearing structure of the building consists of a reinforced concrete skeleton, eight rows of columns up to the fifth floor and four from the fifth floor.

The tallest building in the territory of Bratislava is the Television Tower on Kamzík with a height of 194 m, built in 1975. The tower was built according to the design of a collective of authors, Ing. Stanislav Májek (chief engineer), Ing. arch. Jakub Tomašák (architect), Ing. Milan Jurica, Dr. Ing. Juraj Kozák (chief structural engineer) a Ing. Ján Privitzer. The load-bearing structure is monolithic reinforced concrete up to a height of 13.7 m, from a height of 13.7 m to a height of 90.0 m is formed by a monolithic reinforced concrete core of square cross-section with plan view dimensions 7.0 x 7.0 m with a wall thickness of 400 mm [1].



Fig. 1. Period photography

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The core interacts for horizontal effects with sloping steel columns anchored by massive cross steel beams at 131.5 m level. The beams are anchored into the reinforced concrete core. From the level of 131.5 m to 169.5 m, a steel structure with a tubular cross-section is designed. The last part is realized as a fiberglass extension. The tower platforms are designed as composite systems, supported on inclined beams, founded on inclined steel columns and a reinforced concrete core. The largest plan dimension of the tower is at +45.5 m level with a size of 20.5 x 20.5 m. The tower is founded on a massive footing. The total weight of the steel structures is 1250 tons [2]. At a height of 70 m there is a restaurant with a revolving floor.

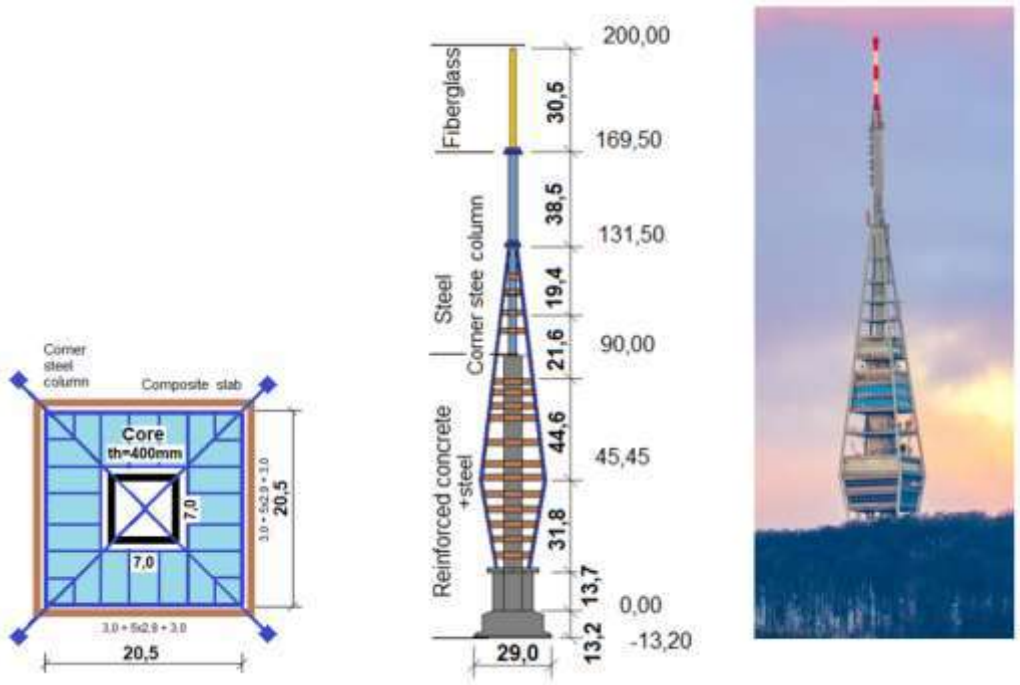
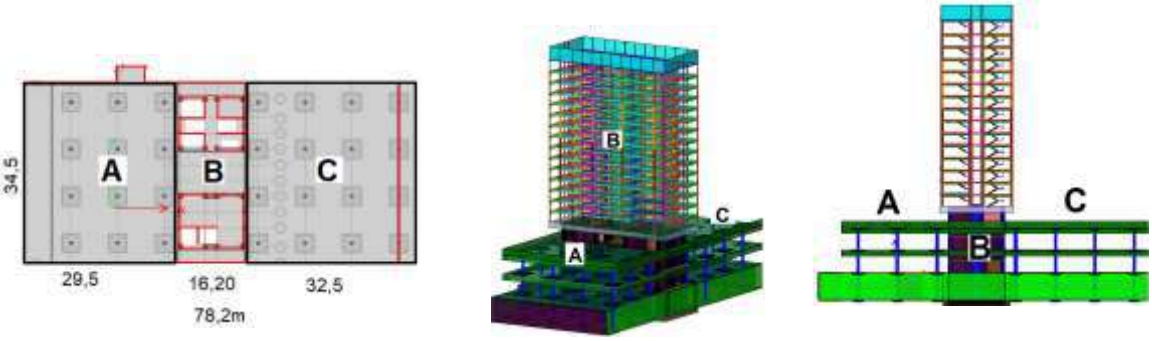


Fig. 2. The Television Tower on Kamzík: a) floor plan at +45.50 m; b) section; c) photo

In 1973, the construction of the Hotel Kyjev was realized. The building has three expansion units A,B, C. The load-bearing system of the building in parts A and C consists of a reinforced concrete monolithic skeleton with 500 mm diameter columns in a 9.0 x 9.0 m modular framework with 3.6 m cantilever spans.



a) b) c)

Fig. 3. Building layout - expansion units: a) floor plan; b) 3D view; c) longitudinal section

Ceiling load-bearing structures are realized as locally supported floor slabs with shallow column capital of dimensions 4.0 x 4.0 m with a height at the column location of 700 mm, including the slab thickness of 200 mm. The skeleton concrete is B250, which currently corresponds to C16/20 grade.

The foundations of parts A and C are formed by a thin slab 200 mm thick with a conical run-up in place of the columns with dimensions of 2.25 x 2.25 m with a thickness at the column location of 900 mm including the slab thickness. The foundation joint is at -6.500 m level in part A and at -5.400 m level in part C.

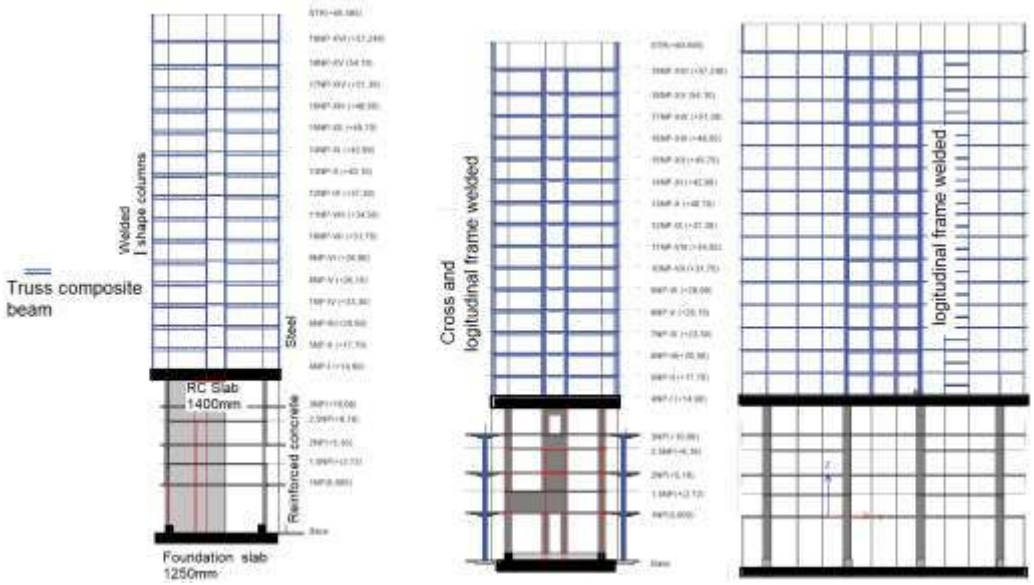
The load-bearing system of the high-rise building B is formed up to the ceiling level of the 4th floor (ceiling on the 3rd floor upper edge of reinforced concrete +14.900 m) by a reinforced concrete core and from the level of 14.900 m the load-bearing structure is formed by a steel structure with multi-story steel frames. The steel load-bearing structure is embedded in a 1,400 mm thick reinforced concrete slab with the upper edge at +14,900 m.

Ceiling load-bearing structures are on the floors below the level -14.400 m formed by monolithic reinforced concrete slabs 200 mm thick. The floor slab at the level of 14.900 m (3rd floor) is in the direction perpendicular to the longitudinal axis cantilevered 1.75 m on each side. In the direction of the longitudinal axis, the slab is cantilevered 3.0 m on each side. The 1,400 mm thick slab is made of B330 concrete, which corresponds to C25/30 concrete in the current marking. The slab is reinforced with steel rebars 10 426 with yield strength f_{yk}= 420 MPa.

The floor slabs of the typical floors are made of 110 mm thick reinforced concrete slabs made of B250 (C16/20) concrete, partially interlocked with steel ceiling beams. The ceiling beams are realized as trusses for a span of 6.9 m with the top flange interlocked with the reinforced concrete floor slab.

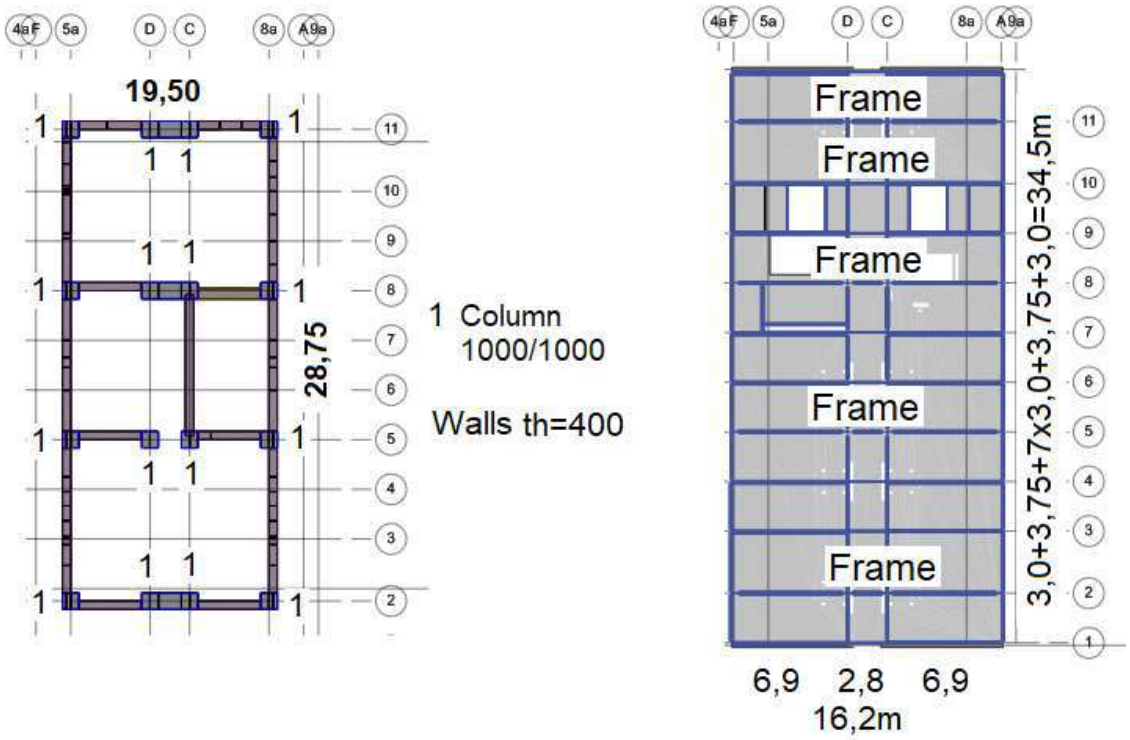
The high-rise building (part B) is based on a 1250 mm thick reinforced concrete slab made of B330 (C25/30) concrete reinforced with reinforcement 10 426. The foundation joint is at the level of -6.500 m.

The horizontal stiffness of the building is ensured by the stiffness of the reinforced concrete core. The reinforced concrete core is formed by 12 columns with a cross-section of 1000 x 1000 mm, which are connected by 500 mm thick reinforced concrete walls. The core supports a 1,400 m thick reinforced concrete slab with a upper edge of +14.900 m.



a) b) c)

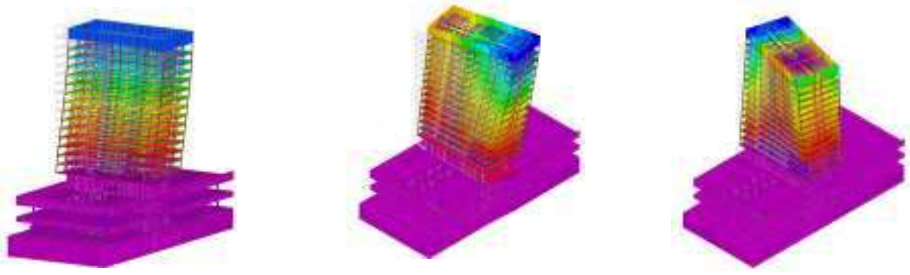
Fig. 4. Load-bearing structure part B: a) cross section; b) cross section at frame location; c) longitudinal frame series C, D



a) b)

Fig. 5. Load-bearing structure part B: a) reinforced concrete core; b) floor plan of a typical floor

The objects were designed according to the theory of allowable stresses, the original static calculation from 1966 has 222 pages, the frame calculations were performed on a MINSK 22 computer with 10,000 operations/second. Seismic loads were not considered in the design of the building. As part of the reconstruction, a structural assessment of the building was carried out in accordance with the currently valid EUROCODES standards. Part B (the high-rise building) met the limit state of load-bearing capacity and serviceability, which is rare in the assessments of existing buildings. Seismic load was specified by a linear response spectrum. Based on the geological survey, according to STN EN 1998-1/NA/Z2, the reference peak acceleration of a_{gr}=0.63 m/s^2 and a Category B bedrock with 5% attenuation spectrum Type 2, behaviour coefficient q=1.5 is determined. The response to seismic load was solved by the method of linear response spectra. The internal forces in the low-floor parts were solved in the original project by analytical methods based on the works of prof. Hruban. When assessed according to EUROCODES, the columns and parts of the floor slabs did not meet the reliability conditions.



a) b) c)

Fig. 6. Modal shapes - frequencies: a) f_1= 0.254 Hz; b) f_2= 0.283 Hz; c) f_3= 0.347Hz

2. New high-rise buildings in Bratislava

There are more than 54 buildings with more than 20 storeys and thirteen buildings higher than 100 m in Bratislava. The main development of construction of high-rise buildings has taken place in the period since 2002. Examples of the design of the load-bearing systems of several high-rise buildings are used to highlight the key issues and their possible solutions in verifying the reliability of the load-bearing system of a high-rise building. Three residential high-rise buildings built between 2018 and 2023 document the development of construction on the right bank of the Danube River. In this area, 10 buildings ranging from 33 to 46 storeys have been constructed. The tallest building is the Eurovea Tower with 46 floors above ground. The common denominator for the Panoráma City, Klingerka and Eurovea 2 residential buildings presented in the article is the support reinforced concrete structure and the use of higher strength concretes.



Fig. 7. Realized buildings on the right bank of the Danube River (excluding Eurovea 2)

2.1. From intention to implementation

The investor's intention to build on a plot of land in the wider city centre, adjacent to the National Theatre building and the large multifunctional EUROVEA 1 complex, was established as early as 2008. First high-rise buildings in this were Panoráma City. The architectural design was entrusted by the investor to the renowned Spanish architect Ricardo Boffil, (Ricardo Bofill Taller de Arquitectura RBTA (http://www.ricardobofill.com/). Local architectural support was commissioned by the investor from P-T, spol. s.r.o. under the leadership of the prominent Slovak architect Juraj Hermann, http://www.projekt-team.sk/. RBTA designed a complex dominated by two towers of triangular plan with fifty-four (54) floors above ground and four floors below ground. High-rise buildings with a height of 180 m above ground level had mixed use (up to 11 storeys administration, higher flats). In addition to the landmark buildings, the complex included a shopping arcade and a hotel. The high-rise buildings were tested for wind effects in the CPP wind tunnel, Fort Collins, Colorado, USA https://www.cppwind.com/. The test of high-rise buildings in a wind tunnel was carried out

for the first time in Slovakia. Monolithic concretes of higher strength class than C40/50 were designed for the columns and walls of the building for the first time in Slovakia on a larger scale, which allowed to reduce the cross-sections of the walls and columns and saved about 200 m^2 of usable area on each building. Prof. Ysrael Seinuk (1931-2010) of the USA, the renowned author of several extremely slender high-rise buildings (e.g. Trump World Tower, NY) and winner of the prestigious Fazlur Khan Prize (2010), was a consultant on the structural design of the buildings. http://yaseinuk.com/. The experience he passed on to the authors of the structural design of the buildings during the design process helped in many ways to convince the stakeholders of the progressiveness of the design and the correctness of the approach, whether it was the use of higher grades of concrete or the concept of addressing the response of the building to the horizontal effects of wind and seismicity. A geological survey was carried out for buildings with a height of 180 m. Under the core of each building, boreholes were drilled to a depth of 65 m, equipped with inclinometers to measure the deformation of the subsoil. The design of the buildings has been developed to the level of a design for a building permit. The change in market conditions also initiated a change in the investor's original intention. First the height of the buildings was reduced to 140 m with an extension of the floor plan, but even this proposal was not final.

2.2. High-rise buildings Panoráma City

The implemented solution was selected in 2013, where two high-rise apartment buildings with thirty-four (34) floors above ground and one underground floor were proposed with a building height of 112.60 m from the ground floor level.

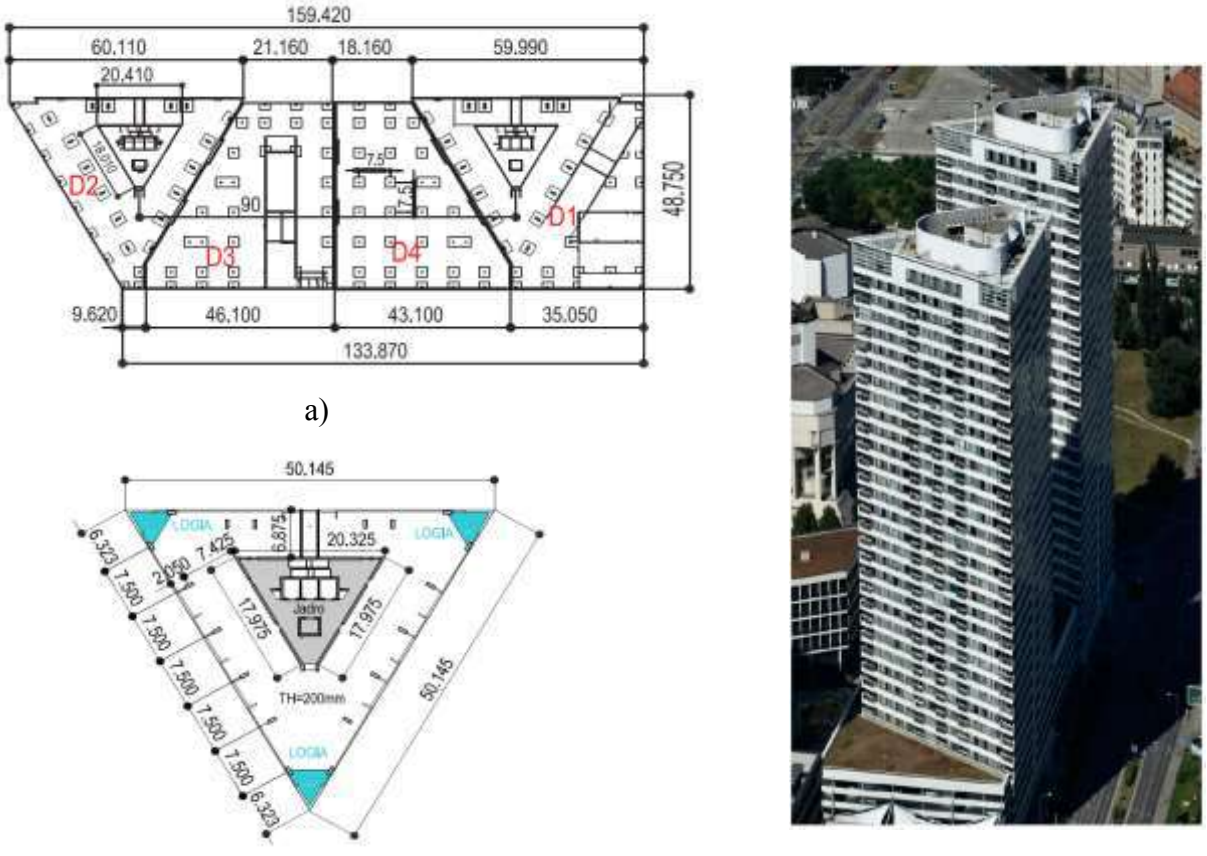
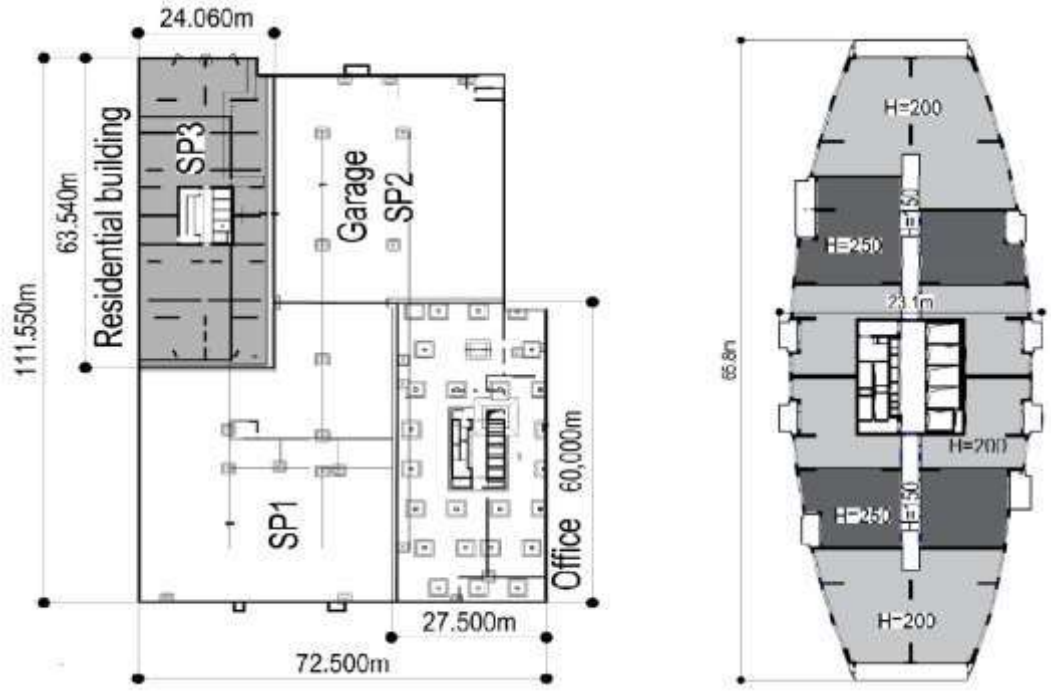


Fig. 8. High-rise buildings Panoráma City : a) division into expansion units, layout; b) floor plan of a typical floor; c) photo

The vertical load-bearing structures are formed by columns of rectangular cross-section with dimensions from 1400 x 500 mm C60/75 in the lower floors to 1000 x 350 mm C30/37 in the upper floors. The maximum design vertical force in the column is 24.0 MN. Walls are designed in the lower part of the core of 300 mm thickness and in the corners of the core of 400 mm thickness in concrete of C60/75 class, from the 21st floor onwards walls of 200 mm thickness are designed. The design of the relatively slender walls and the location of the openings for the ventilation at some of the lintels led to the need to use composite beams. The limiting factor for the design of the lintels was shear resistance. The foundation of the building is implemented on a combined slab-and-pile foundation above the water table. The foundation structures are designed watertight concrete system. The thickness of the foundation slab under the core of the high-rise building is 2000 mm, outside the core of the high-rise building 1500 mm. Under the garage building and the commercial passage (low-rise part from 1st floor to 4th floor), the thickness of the foundation slab is proposed to be 600 mm and the pile heads 900 mm. The slab is designed in C30/37 waterproof concrete, C40/50 at the columns and core. The deep part of the foundation is formed by drilled piles of 900 mm and 600 mm diameters. The lengths of the piles under the high-rise buildings range from 12.0 to 25.0 m. The design of the deep foundation was prepared by SPAI (http://www.spai.sk/).

2.3 High-rise building Klingerka

The residential high-rise building Klingerka is located in the wider centre of Bratislava as part of the Administrative–Residential Complex Klingerka 1. The administrative building has 11 above-ground floors and the upper edge of attic reaches 49.80 m in height. The residential building has 36 above-ground floors with the upper edge of attic reaching the height of 115.000m and the height of the last residential floor being +110.195m, making it the tallest residential building in the Slovak Republic at the moment. In terms of layout, the objects are connected through a four-storey car park. The characteristic feature of both objects is the elliptical plan view.



a) b)

Fig. 9. High-rise building Klingerka: a) division into expansion units; b) floor plan of a typical floor

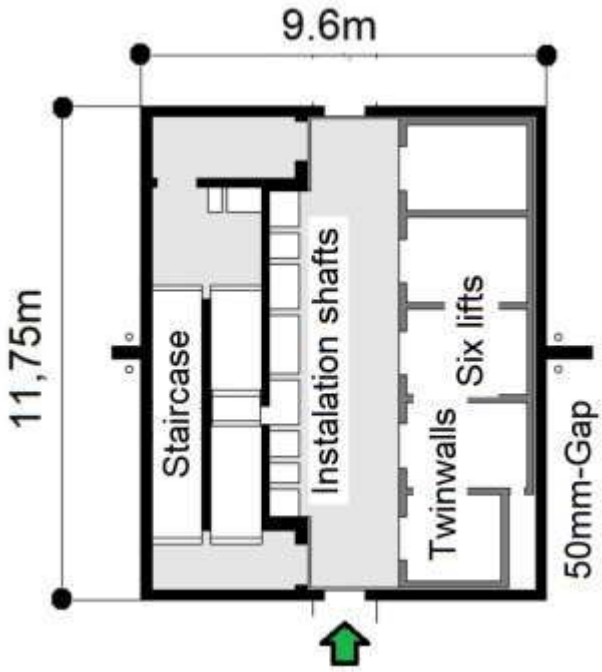


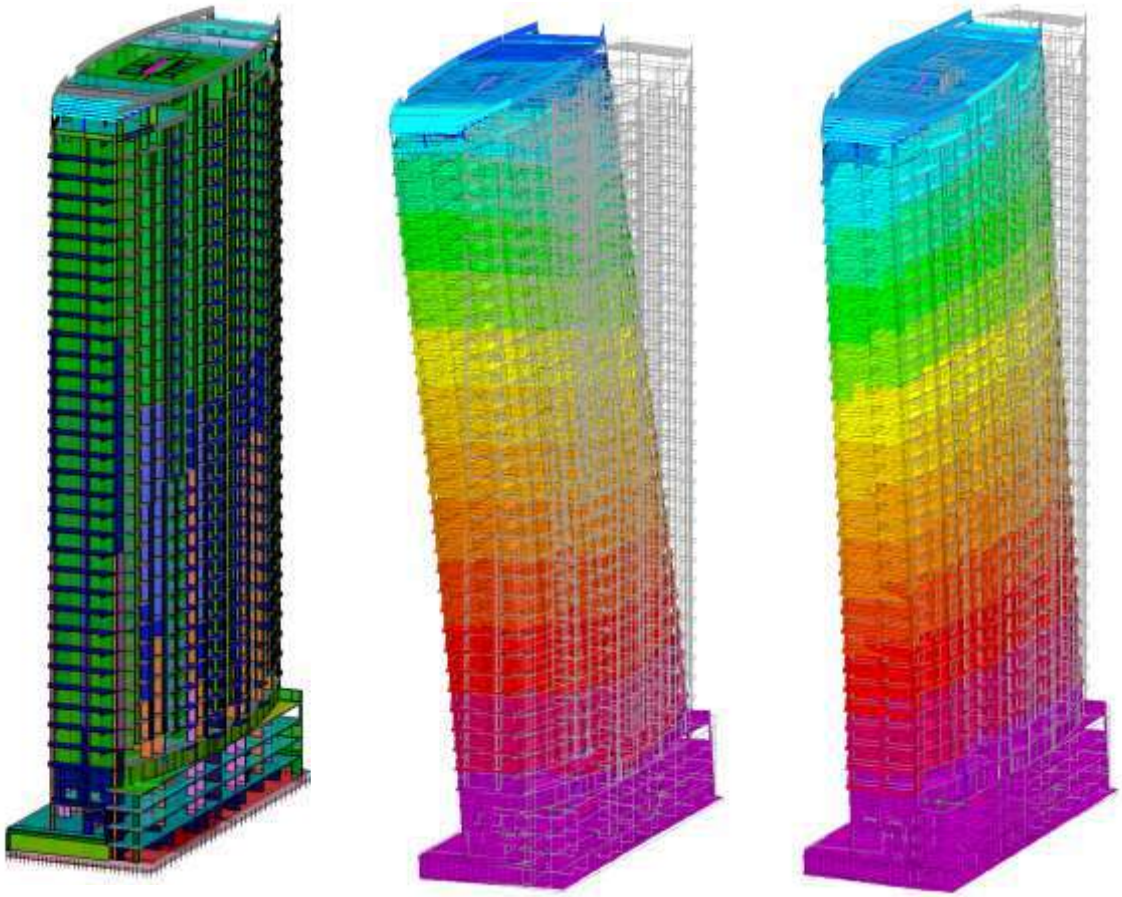
Fig. 10. High-rise building Klingerka: reinforced concrete core

The load-bearing structure of the Klingerka object is comprised of a reinforced concrete skeleton with shear walls and core. The core wall thickness ranges from 600 mm up to 220 mm with a concrete class of C30/37 to C60/75. The walls in the lower part of the core reach a thickness up to 600 mm and are made of C60/75 concrete due to the significant weakening by the openings. The shear walls also form the partitions between apartments, ensuring the acoustic requirements for the partitioning of the apartments. Other partitioning structures are made of plasterboard partitions. The walls of the lift shafts are prefabricated and separated from the walls of the core by dilatation. The ceiling load-bearing structures are made of locally supported reinforced concrete slabs with a thickness of 150, 200, and 250 mm from C30/37 concrete. The floor slab inside the core is separated from the walls of the core for acoustic reasons. At the location of the columns and walls designed from C60/75 concrete there is a designed section with a width equal to 2x the thickness of the slab from C40/50 class concrete. The resistance of the floor slabs against punching shear is designed with the use of shear studs. The building is based on pile-raft foundation. The 1500 mm thick foundation slab from C30/37 concrete is supported by bored piles with diameters of 900 mm and lengths ranging from 11.5 to 19.5 m. The underground floors are designed as a watertight reinforced concrete structure. The thickness of the walls and cross-sections of the columns are designed from high-strength concrete, which, in addition to the more efficient use of the plan view area, also reduces the amount of reinforcement.

The design of the load-bearing systems of these high-rise buildings was determined by the architectural solutions and requirements for the technical equipment of the object. The load-bearing system of the Klingerka object consists of a core and shear walls which also serve as partitioning walls between the apartments. The location of the vertical load-bearing structures was determined by the solution of apartment layouts. In the plan view, a rather varied support for the ceiling load-bearing structures was created, which led to the design of floor slabs with various thicknesses. The graduation of the thicknesses of the floor slabs is designed at the location of the partitioning structures so as to avoid disturbances to the ceilings in the rooms. The disadvantage of the natural strength

distribution is compensated by the higher number of shear walls in the direction perpendicular to the longitudinal axis of the object.

The response of the **Klingerka** object to the wind load was designed in accordance with STN EN 1991 1-4 taking into consideration the results of the calculation of wind effects on the building facades, which was carried out using the computational fluid dynamics (CFD) method by the company SIMULÁCIE BUDOV s.r.o. http://www.simulaciebudov.sk/. The results of the study included the wind pressure coefficients as local winds on the surfaces, not as integral resultants on the floors. The use of such results is practically impossible for the calculation of the wind forces. However, they were useful for estimating of the pressure and suction coefficient. The mechanical resistance of the load-bearing system of the building was verified using a static and dynamic calculation on the basis of a 3D computational model in the ETABS 2016 and Scia Engineer 2016 programs. The first frequency of the flexural vibration of the building has a period of 4.28 s (0.233 Hz) in the direction perpendicular to the main axis of the ellipse. The second frequency with a period of 2.98 s (0.335 Hz) is a flexural vibration of the building in the direction of the main axis of the ellipse.



a) b) c)

Fig. 11. High-rise building Klingerka: a) analytical model b) first frequency f_1=0.233 Hz; c) second frequency f_2=0.335Hz

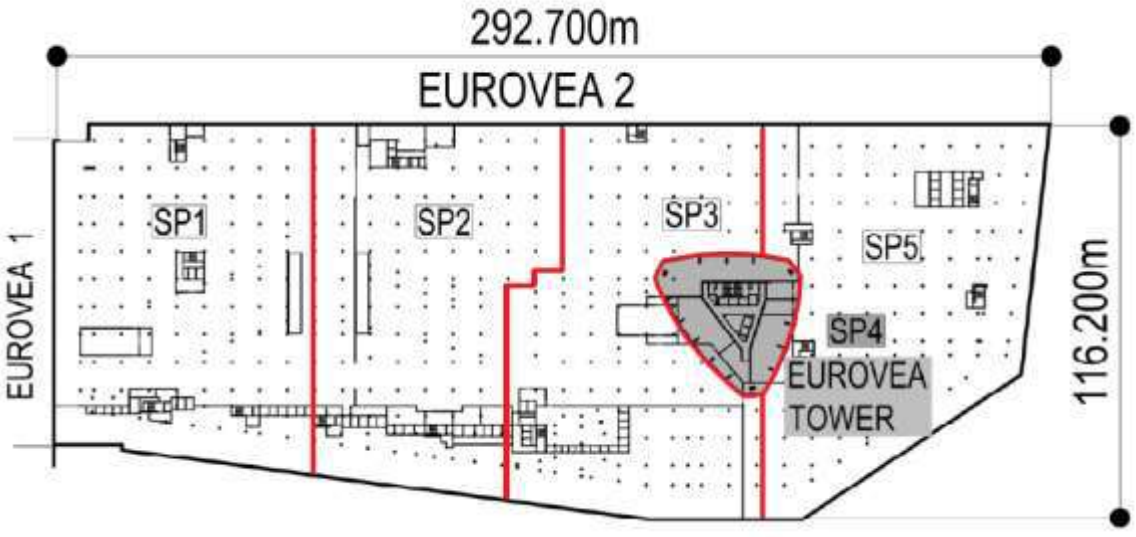


a) b)

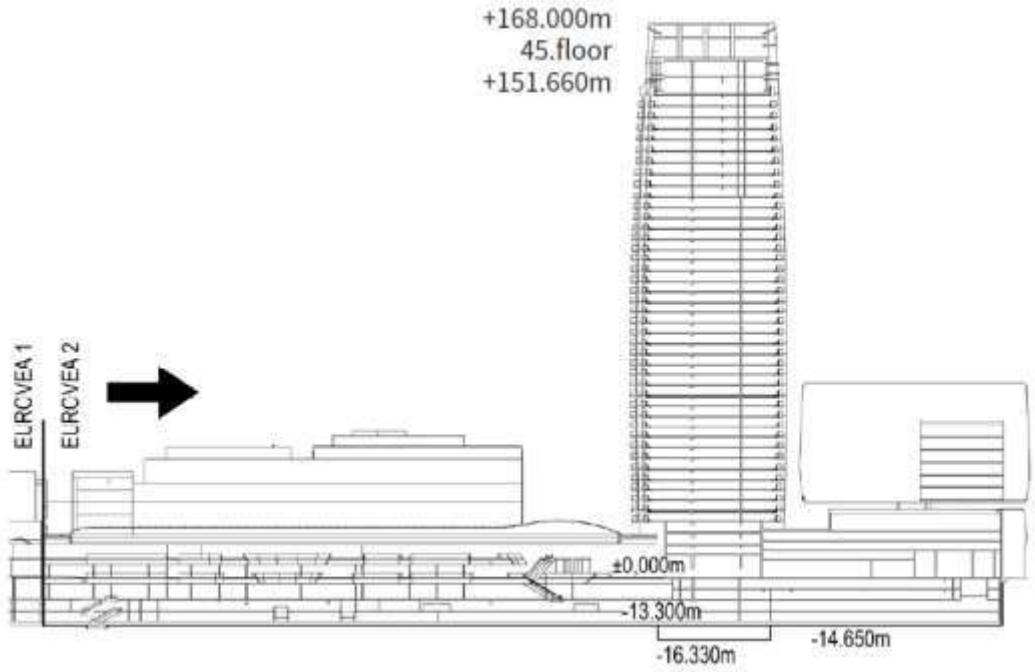
Fig. 12. High-rise building Klingerka: a) general view; b) views of the building

2.4 High-rise building Eurovea Tower

The residential tower EUROVEA 2 is part of the multi-purpose complex EUROVEA 2, which is the follow-up to the Eurovea 1 project. The entire complex is situated on the left-hand bank of the Danube in the wider centre of the capital city. The multi-purpose complex Eurovea 2 with the plan view dimensions of 292.7 x 116.2 m has four underground floors mostly intended designated for parking. A prominent landmark of the complex is the high-rise residential building with 46 above-ground floors with a total height from the ground floor level of 168.0 m. The last residential floor reaches the height of 155.660 m.

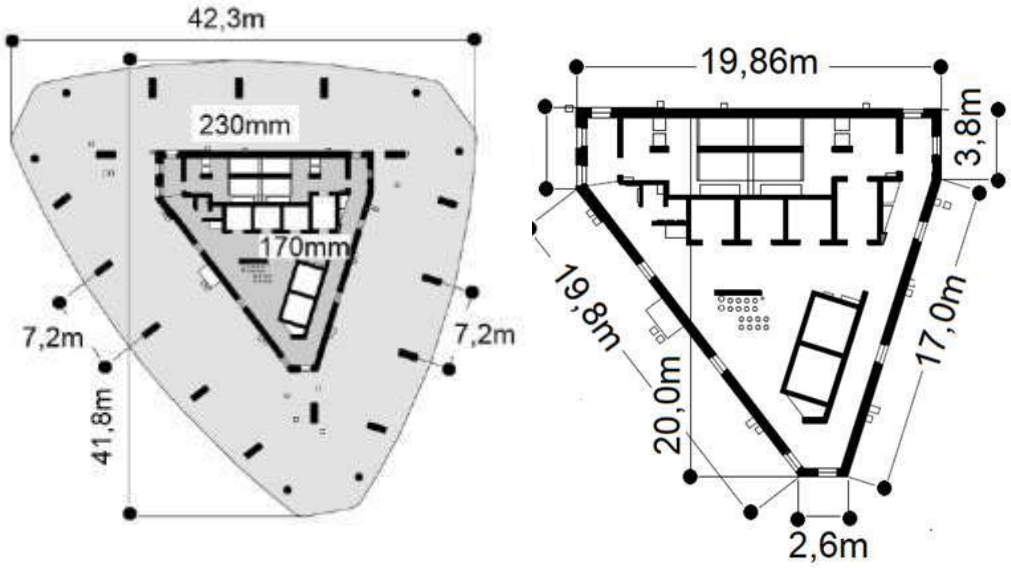


a)



b)

Fig. 13. High-rise building Eurovea Tower: a) division into expansion units; b) longitudinal section



a) b)

Fig. 14. High-rise building Eurovea Tower: a) floor plan of a typical floor; b) reinforced concrete core

The load-bearing structure of **Eurovea Tower** is comprised of a reinforced concrete skeleton with columns and core. The core is of a triangular shape with bevelled edges. The core wall thickness ranges from 250 to 600 mm. Up to the 9th above-ground floor, the walls of the core are 600 to 450 mm thick and made of C60/75 concrete. A 600 mm thick wall is only found in the upper part of the core with doors. The columns with a rectangular cross-section with the dimensions of 900 x 1700 mm are made of C60/75 concrete. The

dimensions of the columns on the 33rd above-ground floors are 500 x 900 mm and made of C40/50 concrete. The corners of the building from the 1st above-ground floor are designed with a V-shaped column which passes in the underground floors into a trapezoidal cross-section 2300-1200 x 1200 from C60/75 concrete. The columns are connected through a beam in the upper part. From the 3rd above-ground floor, the corners of the building include pairs of circular composite columns (six columns) with a diameter from 600 to 550 mm. The columns are reinforced with concreting reinforcement d=40 mm and steel-rolled profile insert HD 320 x 245 from HISTAR material f_y=460 MPa. The reinforcing inserts are joined using bolted joints. The contact of the rolled profiles is designed by a contact of precisely milled-off 50 mm thick face slabs. Columns are made of C60/75 self-compacting concrete.

The area for the Eurovea 2 construction is located on the Danube riverbank. It is comprised of Neogene and Quaternary sediments. The groundwater level is directly related to the water level in the Danube. The surface of the area is located at an altitude of approx. 139.0 meters above sea level. The groundwater level can reach up to 135.0 meters above sea level. The footing bottom of the high-rise building is located at 123.5 meters above sea level, which means that it is 11.5 m below the maximum estimated groundwater level. In order to open the foundation pit with the approximate dimensions of 293 x 117 m, it was necessary to construct an underground diaphragm and supporting wall. Eurovea 2 is adjacent to the previously implemented Eurovea 1 and their interconnection is watertight.

The pile-raft foundation of the entire complex is designed as a single expansion unit watertight concrete structure. The 3m thick foundation slab (under the tower) from C30/37 and C40/50 concrete works together with the bored piles with a diameter of 900 mm and length from 22 to 32 m. A total of 136 piles are located under the tower. Particular attention in the designing of the foundations should have been paid to the mutual settlement of the tower and the adjacent expansion units. Parts of the Eurovea 2 complex near the tower have only 4 underground floors and are strained by the upward hydrostatic pressure. These locations are designed with tension piles to secure the objects against lifting. In order to monitor the settlement of the complex, observation boreholes with installed sliding deformeter were established. The building is already completed. The measured settlements, which are carried out on a continuous basis, are in compliance with the foreseen values.

The foundation slab of the tower was executed by continuous concreting for a period of approx. 60 hrs. The total volume of the concrete is 7,050 m^3, reinforcement of the foundation slab is 1,880 tons. Temperatures in the slab cross-section were monitored during the concreting. These temperatures did not exceed the limit values. The total amount of concrete on the Eurovea Tower high-rise building is 30,400 m3, the total consumption of B550B concrete reinforcement is 5,420 tons.

The load-bearing system of the high-rise buildings Panoráma City and Eurovea Tower is conceptually designed as a system with a reinforcing core and columns. The shape of the reinforcing core is determined by the architectural design. The effects of wind on the triangle-shaped building significantly depend on its direction. Due to the fact that the openings are unevenly distributed in the plan view (especially on the lower floors), the load-bearing system is asymmetrical and, therefore, it was necessary to pay particular attention to the horizontal effects taking into consideration the effects of wind in many directions. In order to determine the effects of wind, a test in the UK RWDI wind tunnel was executed. http://www.rwdi.com/. The test results showed the effects of wind at individual floors for 24 directions. These values were used in the calculation.

In order to analyse the effects of wind, an analysis of the effects was drawn up using the computational fluid dynamics (CFD) method by the company SIMULÁCIE BUDOV s.r.o. http://www.simulaciebudov.sk/. Comparing the results of the two analyses, shear force and tipping moment, we found quite large differences.

The verification of the reliability of the load-bearing systems in terms of their mechanical resistance (ultimate limit state (ULS) and serviceability limit state (SLS)) was carried out using spatial calculation models with the use of two independent computational programs, SCIA Engineer and ETABS. This procedure proved effective for the authors as an independent control, as well as for the use of the strong points of the individual computational programs. The SCIA Engineer program with its SOILIN module was used to design the interaction of the construction with the subbase. The design of the reaction to static and dynamic loads was carried out with the use of the ETABS program. The floor slabs were designed in the SCIA Engineer and SAFE programs.

The results of wind tunnel test were used to design the response to wind effects for Eurovea Tower. The results were also compared to the solution as per STN EN 1991 1-4. The horizontal load-bearing system consists of a reinforced concrete core with a triangular plan view. The mechanical resistance of the load-bearing system of the building was verified using a static and dynamic calculation on the basis of a 3D computational model in the ETABS 2016 and Scia Engineer 2016 programs.

The first frequency of the flexural vibration of Eurovea Tower has a period of 5.471 s (0.182 Hz). The second frequency with a period of 4.647 s (0.215 Hz) is a flexural vibration of the building. The design of the response to seismic loads was carried out using the linear response spectrums method with the design seismic acceleration determined by the geological survey. The response spectrum parameters are determined by the geological survey and according to STN EN 1998-1/NA/Z2 as the reference peak acceleration a_{gr}=0.0675 m/s^2 and subbase category D, subbase coefficient S= 1.5, threshold periods Tb=0.125 s ,Tc=1.0 s Td=3.0 s , object importance coefficient γ_I=1.2 damping 2%.

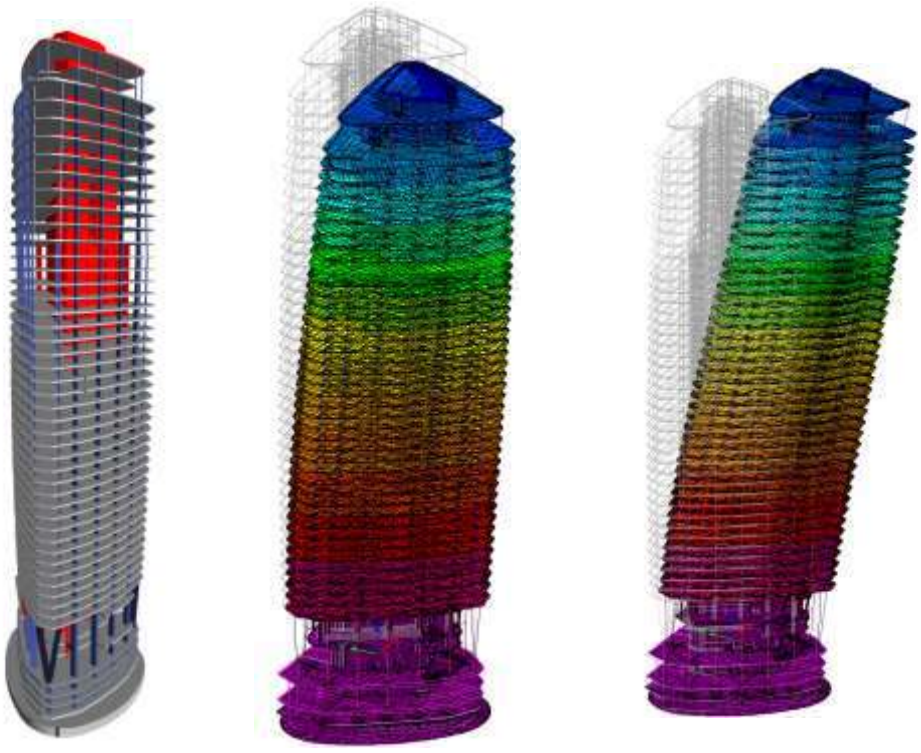


Fig. 15. High-rise building Eurovea Tower: a) analytical model b) first frequency f_1=0,182 Hz; c) second frequency f_2=0,215



a) b)

Fig. 16. High-rise building Eurovea Tower: a) overall view of the construction b) wind shield



Fig. 17. High-rise building Eurovea Tower before completion

3. Insights from the design of high-rise buildings

A significant difference between the design of the horizontal load-bearing system of a high-rise building compared to a low building is the transfer of significantly greater effects from horizontal wind loads and seismicity to the foundation. For wind effects, the wind effects increase with the height of the building, and at the same time the bending or tipping moment at the foundation level increases with the square of the height. There is a difference of about 36 times between a five-storey building and a thirty-storey building. Under seismic loads, the objects have lower natural frequencies, so the accelerations are smaller, but due to the mass can be decisive especially in areas with higher design seismic acceleration (in Slovakia Žilina, Komárno). When assessing the load-bearing system it is not possible to determine only with the magnitude of the shear force or tipping moment which effect is decisive. Different combinations of loading conditions may be decisive for different elements.

High-rise buildings are among the large-scale load-bearing structures with the interaction of vertical and horizontal systems and foundations. The consideration of the

construction process (elastic action, creep and shrinkage) is not linear and has a significant influence on the internal forces in vertical structures from the dead weight of the structure as well as the interaction with the soil. The effects of wind acting in different directions are influenced by the shape of the building and may not be the same for two opposite directions. To obtain the wind effects, a wind tunnel test is appropriate for buildings of atypical floor plans with frequencies below 1.0 Hz. The effects from seismicity depend strongly on the behaviour coefficient (ductility) q. The behaviour coefficient q characterizes the ability of the building load-bearing system to absorb the energy generated in an earthquake. Consideration of cross-sections according to EC2 without taking EC8 into account leads to incorrect results on the hazard side. It is often the case that a q-factor greater than 1.5 is used and the assessment of the cross-sections is carried out without taking EC8 into account, leading to significant errors.

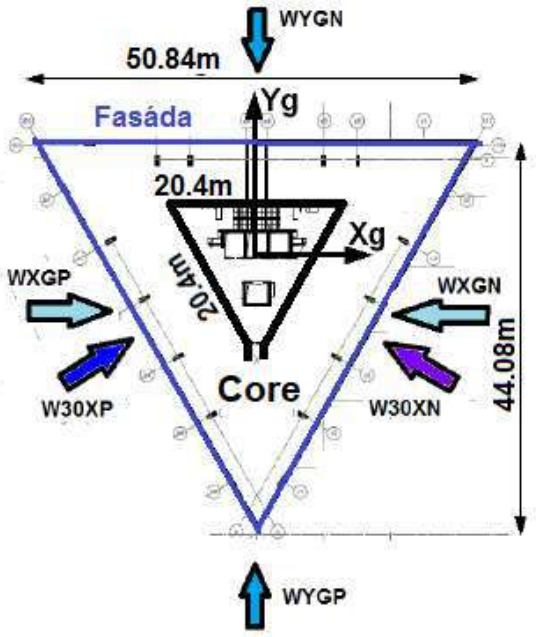


Fig. 18. Example of considering the effect of wind on a building with a triangular floor pla

In high-rise buildings, the influence of the gradual construction of the building is quite pronounced. Taking into account the progress of shrinkage and creep in vertical load-bearing structures influences the behaviour of the load-bearing system. The shortening of columns and vertical load-bearing structures affects the internal forces and deformations of the structures. The response solution with the inclusion of the gradual construction of the structure and the rheological properties of the concrete affects the redistribution of forces from vertical loads and better represents the actual behaviour of the structure.

4. Conclusion

When assessing existing buildings, the main problem is the difference in the standards used in the design at the time of construction and now. Buildings have to be assessed in accordance with the currently valid standards. In terms of assessing mechanical resistance, of the main bearing structure according to currently valid standard we usually find out not enough reliability in mechanical resistance.

From a technological point of view, higher strength materials are currently used in the design of high-rise buildings. There is also a significant difference in the implementation of concrete and steel structures. Reinforced concrete solid structures, foundation slabs, floor slabs are concreted in one working step. Joints of steel structures are bolted with maximum minimization of assembly welds.

Acknowledgement

The authors of the architectural design and general designer of the Klingerka and Eurovea Tower buildings is GFI a.s. https://www.gfi.sk/, to whom we would like to express our gratitude for their trust.

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THE CONCRETE BRIDGES OF THE GREAT BULGARIAN ENGINEER LALO RAYKOV(1923-2023)

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Abstract: *The authors present a short biography of the life and the achievements of the great Bulgarian civil engineer L. Raykov in the field of the concrete bridge design. The paper briefly describes the most interesting examples of deck-stiffened arch bridges designed by L. Raykov which are considered as a pionerring application of this type of structure in Bulgaria. Today these bridges still attract our interest not only with the dimensions of the structure but also with the technology used for their manufacturing and assembling of the bridge components.*

Key words*: concrete bridges, deck-stiffened arch*

1. Introduction

1.1. Early life

Eng. Lalo Raykov was born in 1923 in Sofia, Bulgaria(Fig.1). He completes a secondary education level at Deutsche Schule in Sofia at 1942. L. Raykov received a civil engineering diploma from the State Polytechnic in Sofia in 1950 with honors.



Fig.1. Lalo Raykov

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1.2. Bridge design

Since 1950, L. Raykov worked for the design department "Transproject" of the Ministry of transport. Next nine years he takes part as a designer of many significant bridges: a 3-span, Gerber type railway bridge near Varna (1954); a deck-stiffened arch bridge (Maillard system) at railway Perperek- Kardjali (1957); an arch bridge at railway Sofia-Mezdra (1959) with a longest in Bulgaria span of 80 m, placed in a horizontal curve with a radius of 250 m over a river Iskar; a deck-stiffened arch bridge in Tarnovo over a Yantra river with a span of 60 m. Eng. Raykov is a designer of the first prestressed RC bridge in Bulgaria by using of a prestressing system "Leoba" placed in Pernik. Next years L. Raykov works for design companies "Sofproject", „Kremikovtzi", "Road constructions" and "Injstroy" as a head designer. L. Raykov takes part in the design team of Asparuhov bridge near Varna. He works on the design of piers at a steel part of the bridge and their pile foundations. He also works on the design of the footbridge over Chaya river in Asenovgrad. In 1980 eng. Raykov was designed a bridge over Osam river in Lovech placed near to the famous covered bridge of Master Kolyo Ficheto built in 1874. The bridge is a prestressed frame structure with span of 80 m and a depth of the main girders of 1.25 m. As a head of the design department of "Sofproject" company he takes part in the design of several bridge and overpass structures in Sofia. They include an overpass at Yavorov lane blvd. which is a prestressed continuous beam structure in a horizontal curve; overpass near the National Palace of Culture and the pedestrian overpass bridge at Bulgaria blvd.; overpass structures located at Boyana-Sofia ring-highway interchange.

1.3. Other structures

L. Raykov is a pioneer into design and construction of dry ship's dock structures in Varna. Analysis of these structures is performed by the finite difference method with an application of computer programs. As a head of the design department of the "Sofproject" company he works on a design of 10 prestressed cylindrical water tanks, 40 m high, with volumes of 10 000 m^3 and 15 000 m^3.

1.4. Patents

Eng Raykov is an author of three patents in a bridge construction. The first one considers a new type of bridge piers called "Camerton" (1970). This patent is used in the construction of a set of bridges at first class road Pirdop-Rozino with a maximum pier's height of 72 m and in construction of a road bridge near Rouse with piers 90 m high. The second patent concerns a post tensioning of bridge girders (1976). The method uses RC elements casted on site that at next stage are lifted into their final position by hydraulic jacks. The final stage includes a post-tensioning of the girders which form continuous beams. This method is applied in a construction of a city highway bridge at Sofia and in a road bridge near Sofia airport. The third patent concerns the construction of continuous precasted RC bridges. This method is applied in construction of two road overpass bridges on Vratsa-Boychinovtcy railway.

1.5. Scientific interests

Eng. Raykov is a author of many scientific papers which concern problems of bridge design and construction: nonlinear problems of high bridge piers, prestressing problems of the bridge girders, etc.

2. Summary of the famous bridges designed from L. Raykov

2.1. "Boyana" overpass

The "Boyana" overpass is a 3-span (32 m + 38.50 m + 32 m) prestressed concrete continuous box girder bridge (Fig.2.), which carries the Sofia ring-highway traffic. It is located at Boyana – Sofia ring-highway interchange. The bridge was designed by "Injproject" company with a head designer L. Raykov. The bridge builder was "Injstroy" company. The bridge owner is the Municipality of Sofia. It was opened to traffic in 1974.

The bridge is 102.50 m long and 19.25 m wide. The superstructure is a five-cell box cross-section with a depth of 1.30 m and two 3.60 m long overhangs. The box cells are 2.20 m wide. The walls of the box section in a longitudinal direction are prestressed by 7 tendons for each of them. The support zones of the superstructure are solid concrete structure which is additionally prestressed into a transversal direction by 10 tendons. The bridge piers are single-column hammerhead bents. Dimensions of the column are 7.20x1.25 m at the top, 4.0x0.70 m at the bottom and 9.20 m tall. The pier's footings are with dimensions of 11.40x2.20 m. The bridge abutments are solid concrete structures 4.65 m high. The abutment bearings are steel rollers and pier bearings are neoprene pads [2].



Fig.2. "Boyana" overpass.

2.2. "Camerton" viaduct

The "Camerton" viaducts are located at the Pirdop-Karlovo first class road (Fig. 3). The designers are L. Raykov, K. Tzekov, G. Ganchev, K. Kotsev and E. Milchev. The viaducts were built by "Road construction" company. The owner is Agency "Road Infrastructure". The viaducts are opened to traffic in 1976. The superstructure and the piers of are standardized system which is applied at km 219.641 and km 220.373 with spans of 33.93 m + 5x35.00 m +33.93 m, at km 221.059 with spans of 33.93 m + 7x35.00 m +33.93 m. The main girders of the superstructure are precast prestressed concrete I-beams, 1.80 m deep. The top flange of the beams is 2.70 m wide and 0.15 m thick. The bottom flange is 1.00 m wide and 0.40 m thick. The web thickness is 0.25 m. The prestressing was performed by a "Freyssinet" system. Each of the spans has 4 main girders and 3 diaphragms beams (at the span's ends and at the middle of the span). The main girders are spaced at 3.10 m. The total width of the bridge structure is 12.00 m. The bridge bearings are steel hinge supports. The expansion deck joints are placed on each of the

piers. The bridge piers are two-column bents which height varies from 19.00 m to 70.00 m. The columns have a box cross-section with dimensions of 3.00x3.00 m. The box walls are 0.25 m thick. The upper 12.00 m ends of the columns with the cap beams are split by 50 mm gap. The shape of the pier's columns is similar with the musical tuning fork called "Camerton". This solution allows the upper parts of the columns to bend independently and provides a longitudinal movement of the bridge deck without roller supports. The bridge pier footings are with dimensions of 12.00x10.00x2.50 m. The abutments and wing-walls are monolithic reinforced concrete elements [6]. The concrete used for a superstructure is class C45/55 and for a substructure is class C30/37. In 2005 the viaducts was partially retrofitted and rehabilitated. The rehabilitation includes replacement of the expansion joints, waterproofing of the superstructure, cleaning and lubrication of the hinge bearings. The "Most XXI" was the design company of the rehabilitation [1].



Fig. 3. "Camerton" viaduct.

2.3. "Brussels blvd" overpass

The "Brussels blvd" overpass is longest bridge structure in Bulgaria (Fig. 4). The bridge serves as a connection between the Sofia Airport and the entrance highway of the city. The designers are L. Raykov, M. Minev, H. Konstantinov, M. Genov, V. Yochev and A. Georgiev. The bridge was built by "Moststroy" company and it was opened to traffic in 1983. The owner is the Municipality of Sofia. The overpass consists of two parallel structures separated with a gap of 1.00 m. Part of the structure is placed in a horizontal curve with a radius of 1000.00 m. The superstructure is a 7-span prestressed concrete frame structure with a total length of 2114.00 m. The longest span is 49.00 m, measured at the column's toes. The bridge width is 21.75 m. The bridge superstructure has a 9-cell box cross-section with a depth of 1.20 m and 5.64 m wide. The box cells are 0.56 m wide with overhangs of 2.55 m long. The deck section was prestressed by hydraulic jacks "Stobet-300". The prestressing tendons are 18Ø7 of steel St150/170. The expansion joints are placed at every 98 m. The bridge piers are "V"-shaped bents with inclined columns. The columns have a box cross-section with varying dimensions from 5.65x1.347 m to 4.428x0.6 m. The columns height varies from 7.5 m to 10.00 m. The central part of the bridge at the railway overpass is steel orthotropic girders. The concrete used for the bridge structure is class C45/55 [5].



Fig. 4. “Brussels blvd” overpass.

2.4. “Balaban dere” arch bridge

The “Balaban dere” arch bridge crosses one of the feeders of the “Studen kladenec” reservoir and carries the railway Kardjali- Perperek at km 81.200 (Fig. 5). The bridge was designed by L. Raykov and M. Minev. The builder and owner is the Bulgarian State Railways. It was opened to traffic in 1959. The bridge is a deck-stiffened arch structure (“R. Maillart” bridge system) with a total length of 106.66 m. The arches rise 7.60 m above the ground. This is the first bridge in Bulgaria built using the Maillart bridge system. The bridge deck is a continuous beam structure with spans of 2x5.00 m + 26.00 m +4.33 m + 26.00 m + 4.33 m + 26.00 m + 2x5.00 m (Raykov 1957). It consists of two rectangular concrete girders with dimensions of 1.30x0.50 m and a distance between them of 3.50 m. The deck slab is 0.25 m thick with a total width of 5.79 m. The bridge arch has a rectangular cross-section with dimensions of 4.00x0.30 m. The connection walls between the deck and the arch have dimensions of 4.00x0.20 m. The distance between them is 4.333 m. The walls are rigidly connected to the bridge deck and arch. The expansion joints are placed over the abutments. The bearings are steel rollers. The bridge piers outside of the arches are two column bents. The columns have a rectangular cross-section with dimensions of 0.30x0.70 m; the cap beam is 0.65 m deep. The columns have additional braces with dimensions of 0.50x0.30 m. The tallest pier rises 8.30 m above the ground. The bridge abutments are frame structures. The arches have footings with dimensions of 4.50x6.80 m and 1.70 m high bedded on solid rocks. The footings of the bridge piers have dimensions of 1.50x2.20 m and 1.50 m high. The concrete used for the bridge structure is class C20/25; the reinforcing steel – with yield strengths of 235 MPa and 390 MPa [3, 4].

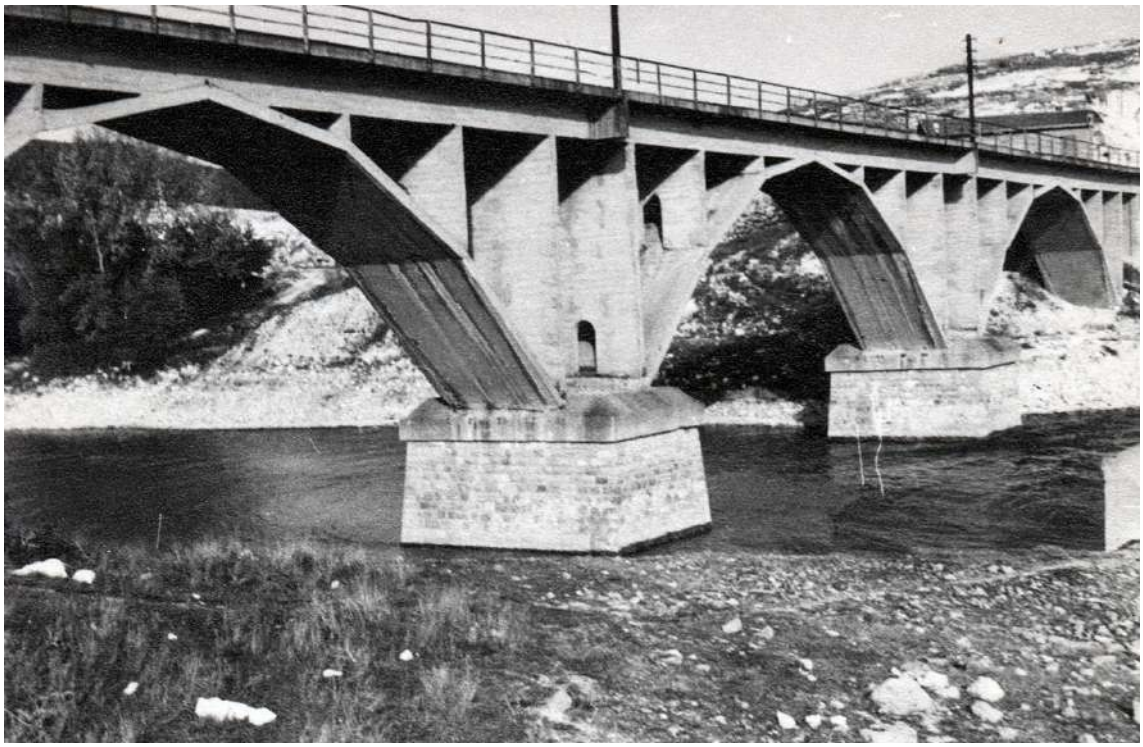


Fig. 5. “Balaban dere” arch bridge. (Courtesy of M. Minev. With permission.)

2.5. “Opletnya” arch bridge

The “Opletnya” arch bridge as shown in Fig. 6, crosses the gorges of the Iskar River near the Opletnya railway station and carries the railway Sofia-Mezdra at km 54.400. The bridge was designed by L. Raykov, H. Popov, K. Tzekov, P. Chavov and A. Ganev. The bridge builder and owner is the Bulgarian State Railways and it was opened to traffic in 1962. The bridge is a deck-stiffened arch structure (“R. Maillart” bridge system) with a total length of 212.00 m. The bridge is a continuous reinforced concrete box girder structure stiffened by a polygonal reinforced concrete arch with a span of 80.00 m. The arch crown rises 14.40 m above the gorge. The bridge is placed in a horizontal curve with a radius of 275.00 m. The magnitude of the centrifugal forces leads to adopt the nontraditional solution of the bridge structure. The arch structure is split into two parts at the arch base like a swallow tail. The arch is 4.50 m wide and thickness varies from 0.75 m to 0.80 m. The superstructure box cross-section has dimensions of 1.65x4.50 m. The box walls have thicknesses of 0.60 m; the box slabs are 0.25 m thick. The total width of the deck is 6.00 m. The connection walls between the deck and the arch have dimensions of 4.50x0.20 m. The distance between the walls is 8.00 m and they also are split at the lower ends. Outside of the arch the box deck have additional 4 spans of 12 m. The remaining parts of the bridge (three and four spans of 12.00 m) are continuous beams with a rectangular cross-section. The expansion joints are placed on the abutments and at the ends of the box deck. The bridge piers are two column bents. The tallest pier rises 16.00 m above the ground. The concrete used for the bridge structure is class C30/37; the reinforcing steel – with yield strengths of 235 MPa and 390 MPa. During the construction of the bridge arch the scaffolding was collapsed. The reasons of the accident were the exceeding of the load-carrying capacity of the scaffolding elements due to the unsymmetrical concrete laying.



Fig. 6. “Opletnya” arch bridge. (Courtesy of D. Gocheva. With permission.)

2.6.“Vratza” overpass

The structure of this bridge was patented in 1988. The patent concerns the construction of continuous precastedRC bridge girders. The bridge piers consist of “Lotus”-shaped columns and cap beams with longitudinal overhangs(Fig.2.7.).



Fig. 7. “Vratza” overpass (Courtesy of D. Gocheva. With permission.)

3. Conclusion

Indeed eng. Lalo Raykov was not only a pioneer of Bulgarian bridge engineering and a brilliant scientist but also a creative, powerful and highly motivated structural engineer. His colleagues which are currently leading engineers in Bulgaria still remember him as an eminent leading, dynamic and highly respected person.

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CONSTRUCTION OF THE SLOVAK NATIONAL UPRISING (SNP) BRIDGE IN BRATISLAVA OVER DANUBE

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***Abstract**: Collection of more than 300 rare photographs and documents from the period of SNP Bridge construction (1967-1972). Photographs and drawings are arranged chronologically and in the way showing sequences of the bridge construction steps.*

***Key words**: Cable-stayed bridge, SNP Bridge construction, collection of rare photographs.*

1. Introduction

The Slovak National Uprising (SNP) Bridge is the second permanent bridge over the Danube River in Bratislava opened 50 years ago in the 29th August 1972.

Design and details of the SNP Bridge calculation are provided in [1-3]. Loading tests are described in [4]. Measurements and evaluation of imperfections are published in [5, 6]. Results of shear lag investigation are presented in [7]. Possible bridge shortening is investigated in [8]. Bridge inspection, assessment and strengthening are referenced in [9-12]. Retrospective information is presented in [13]. A lot of information about the complete history of the SNP Bridge, its planning including political maneuvering, discussions between architects and structural engineers, competition between several proposals (concrete and steel one), final decision and description of the winning proposal for the SNP Bridge may be found in [14, 15]. Popular papers about SNP Bridge were published in the journal Eurostav [16-19]. The SNP Bridge (called New Bridge in the period 1993-2012) is described also in [20].

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The following are the main sources of the photos and drawings presented in this paper:
a) [1]. These photographs and drawings are referred as: [1];
b) Photographs of Department of Metal and Timber Structures, Faculty of Civil Engineering, Slovak University of Technology in Bratislava (in Slovak: Katedra kovových a drevených konštrukcií, Stavebná fakulta, Slovenská technická univerzita v Bratislave. Abbreviation: KKDK SvF STU). These photographs are referred as: (Photo KKDK SvF STU). Almost all of these photographs were taken by the lady photographer of KKDK SvF STU Mrs. Milada Maľcevová-Brucknerová. Only some of photos are stamped on the reverse side with her name. Such photographs are referred as: (Photo Milada Maľcevová – Brucknerová, KKDK SvF STU);
c) [21]. These photographs were taken by Juraj Bončo and they are referred as: (Photo Juraj Bončo). They are very valuable because Juraj Bončo provides the date in which photo was taken;
d) [22]. These photos were taken by Igor Scholtz (Fig. 2b) and they are referred as: (Photo Igor Scholtz).

Photo captions contain symbols: a) BS which means that photo shows something on Bratislava Side (on the left bank of the Danube river), and b) PS = Petržalka Side (on the right bank of the Danube river). Abbreviation KBKM (in Slovak: Katedra betónových konštrukcií a mostov) means Department of concrete structures and bridges.

Information about the teams from the Faculty of Civil Engineering, STU in Bratislava, who participated in the design of SNP Bridge may be found in [23]. A collection of photographs of demolished buildings during the period of the SNP Bridge construction is presented in [24].

The SNP Bridge is the second permanent bridge over Danube River in Bratislava. The bridge consists of a flyover on the right bank of the Danube river (on Petržalka side) with 3 spans (3 x 30 m), a steel cable stayed bridge with 3 spans (74,8 m + 303 m + 54 m = 431,8 m) and the flyover on the left bank of the Danube river (on Bratislava side) with 5 spans (5 x 30 m). The flyovers are built from monolithic prestressed concrete with uniform height 3,6 m of the structure. The steel cable-stayed bridge has a single backward inclined steel tower and a single cable plane. The asymmetrical position of the inclined 84,6 m high A-shaped tower, crowned by a circular restaurant with 32 m in diameter, creates a natural balance to the famous Bratislava Castle and the St. Martin Dome. The restaurant UFO (previously called Bystrica; capacity 120 people) and the public observation deck on its roof offer a nice view of Bratislava.

The SNP Bridge is the only bridge registered in the World Federation of Great Towers, where the key criterion is to have a public observation deck. When it was constructed in 1969, the bridge, with main span 303 m, was the world record holder in the category of cable-stayed bridges. The bridge was the fourth-longest cable-stayed bridge span when it opened to traffic on August 29, 1972. It was the winner of the competition Structure of the Twentieth Century in Slovakia, organized in 2001. The original name of the bridge was Slovak National Uprising Bridge (in Slovak: Most Slovenského Národného Povstania). It was renamed, like other Bratislava Danube bridges, after the collapse of the communist regime in the early 1990s. The name "New Bridge" was applied only during 1993-2012. Before and after this period the name is SNP Bridge.

The general designer was Dopravoprojekt, Inc., Bratislava; the general contractor was Doprastav Bratislava; the complete design solution was by Project-construction office VŽKG n.p. (Vítkovice Iron Works Klement Gottwald, national enterprise), branch Bratislava in cooperation with the Department of Metal and Timber Structures, Faculty of Civil Engineering, Slovak University of Technology in Bratislava, Slovakia; fabrication of the steel structure was by Vítkovice Iron Works; the bridge was erected by Hutní Montáže,

Ostrava n.p.; ČSPD (Czechoslovak Danube River Navigation company in Bratislava) was in charge of the floating; and loading tests were performed by TSUS (Building Testing and Research Institute) in Bratislava in cooperation with FCE STU, VÚIS Bratislava (Research Institute of Civil Engineering Works in Bratislava) and Geodetic Institute Bratislava.

Three cables support the 4,6 m high steel orthotropic box girder, dividing the main span into four segments of 51,5 m + 70,2 m + 82,6 m + 98,7 m = 303 m. The main span is 80.2% of the total length of two stayed spans (74,8 m + 303 m). The reason for the major span being longer than usual is that the bridge has a single back stay anchored to the abutment rather than several back stays distributed along the side span. The navigation opening was prescribed to be 10 m × 180 m. The original design of the bridge support system was improved by using a triangular steel structure located between two tower legs to support the girder. This arrangement produced only vertical loading (165 MN) on the ground foundation because the sum of the horizontal reaction of the girder and the horizontal reactions of the tower legs became zero inside of the concrete foundation block.

The bridge superstructure consists of a steel orthotropic two-cell box girder supported by a single plane fan cable system. Nevertheless, along the length 86 m + 54 m = 140 m, the cross section was originally a single-cell box steel girder because the middle web of the box girder was omitted from the point of the last, most-inclined cable anchorage. In 1991 the bridge was strengthened by adding a Warren-type truss as a middle web to make a two-cell box girder along the whole length of the bridge [9-12]. The six-meter spacing of the "nodes'' and the diagonal area of 120 cm^2 provide fictitious wall shear thickness t_s = 2,73 mm. Adding a truss middle web along the length 140 m has a positive influence by increasing the effective breadth when taking into account shear lag phenomenon caused by variable loading at the support in the form of a framed pendulum wall on the Bratislava side. Cables in one vertical plane along the bridge middle axis are used. The bridge has four traffic lanes. There are two pedestrian walkways of 3 m on both sides of the bottom flange. The steel box girder has a width of 21 m. The orthotropic deck plate is mainly 12 mm thick and increased to 16 mm, 20 mm, 25 mm, and 30 mm at the cable anchorages. The bottom flange plate thickness varies between 12 mm, 14 mm, 16 mm, 18 mm, 20 mm, and 22 mm, and the three vertical web plates are 12 mm thick. Cross frames are spaced at 3 m. The system of longitudinal stiffeners is as follows: closed trapezoidal ribs with 600 mm spacing made of 6 mm and 8 mm thick plates are used in the bridge deck, while the three vertical web plates (spacing 6,3 m, height 4,6 m) and the bottom flange of the box girder are stiffened by L-profiles.

The tower has two hollow legs. There is a lift (delivered by the Austrian company Sowitsch) in the left leg when looking from the Bratislava side and there is an emergency staircase with 430 steps in the right leg. St 52 steel with yield stress 360 MPa and ultimate tensile strength 510 MPa was used for all structures. The total steel used for the superstructure was 6 500 tons, including 1 600 tons for the tower and 620 tons for the cables. The tower legs are inclined 17° from the vertical plane and fixed to the concrete block foundation. The fixed support of the box girder is located between the tower legs and attached to the concrete block foundation 7 m × 20,5 m × 40,5 m. The concrete block is supported by 56 piles. The load-carrying capacity of the piles is between 10,2 MN and 12,5 MN. The anchoring block was constructed in an open pit protected against water infiltration by steel sheet piling, which reached the clay layer. It is built of 12 000 m^3 concrete transmitting a tension force of 129 MN by its own weight. The foundation slab of the pylon has dimensions 20 m x 40,5 m x 7 m. It rests on piles and transfers the vertical component of the reaction equal to 169,4 kN. The underground walls and piles were realised by Vodní stavby, n.p., Praha (Water structures, Prague).

The steel superstructure was fully welded and fabricated in the factory into seven sections. The first tower section was erected from 6 m length parts with the help of slanting

temporary supports. Next a 133,2 m portion of the box girder was completed on the scaffolding in the first field on the right bank and pushed over the river with a 53 m cantilever. Then this part of the box girder was laid down on bearings and stayed by cables I and II. Sections 4, 5, and 6 (lengths from 72 m to 78 m), with weights approximately 800 kN, were mounted on the right river bank 300 m downwards. They were floated to the axis of bridge, lifted, supported by a temporary pier, welded to the previous piece of the girder, and stayed by cables. During the box girder erection 3 intermediate temporary piers (A, B, and C) were used. The seventh section (the last part of box girder) was completed on the left river bank on scaffolding. Consequent vertical movements caused by removing the temporary piers and controlled by telescopic lifting jacks located on permanent supports achieved the desired level of stresses in the cables and box girder to be achieved.

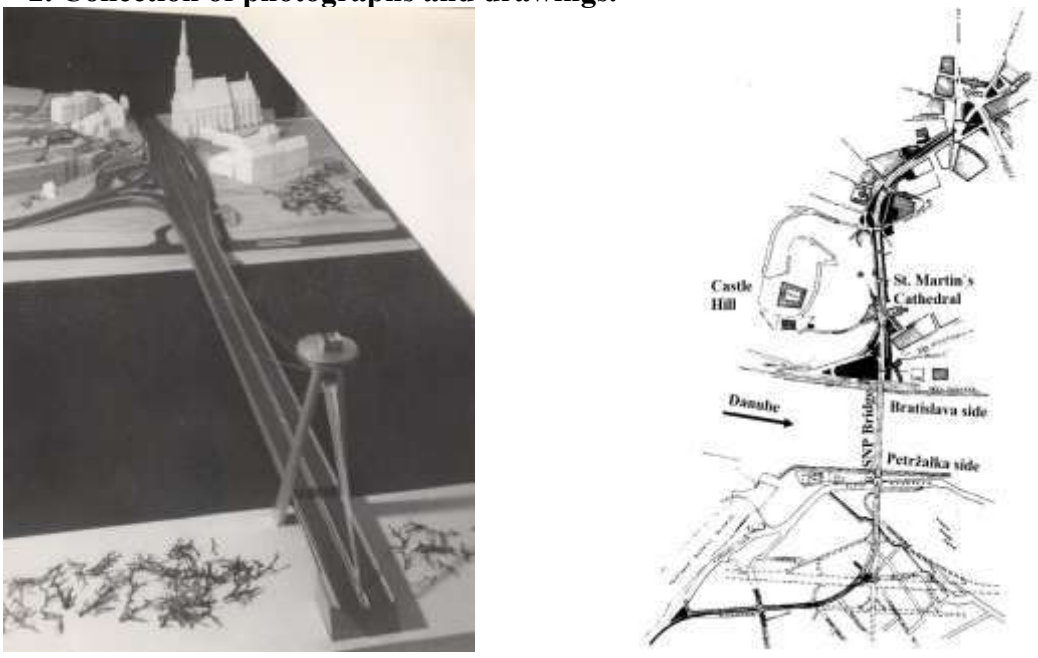
The bridge box-girder segments were 72 m - 78 m long with weight approximately 800 tons. Field-welding segment No.6 and obtaining the vertical deformation 836 mm of the temporary support C, produced negative bending moments at the anchorage cable VI, which influenced positively the bridge box-girder against positive bending moments caused by variable loading. The bridge box-girder segment No.7 was assembled on the scaffold on the left bank of the Danube river (on Bratislava side). The intentional gap was finally closed by the special bridge box-girder segment approximately 12 m long (so called adaptor, Passstück in German). After closing, the box girder segment was dropped 537 mm on the support located on Bratislava bank. As a result of this very sophisticated and complicated erection procedure, the thickness of the box-girder flanges changes little although the box-girder height is constant.

Static and dynamic loading tests by moving vehicles and jet motors impulses were performed during July and August 1972 and achieved 78% of the total loading specified by the design codes. Good agreement between theoretical and test results was found [4]. See also Baláž, Chladný [9-12].

The bridge has been inspected and maintained in accordance with code ON 73 62221, once every three years. The shape of the bridge is verified twice a year by surveying. The owner of the bridge is the city of Bratislava.

Construction costs were estimated at 344 millions Kčs.

2. Collection of photographs and drawings.



a) b)

Fig. 1 SNP Bridge: a) Model; b) Location.



a) b)

Fig. 2. PS. a) Ceremonial opening of SNP Bridge construction. Ing. František Barbírek (on the right), Deputy chairman of the SNR (Slovak National Council) and Ing. arch. Milan Hladký, CSc. (on the left), Chairman of MsNV. Inserting the charter into the foundation stone of the structure. 6.12.1967. (Photo Igor Scholtz); b) TV broadcasting. 29.7.1969. From the left: Ing. Igor Scholtz (1928-2013), Head of the investor department for the construction of the Danube bridge, MSK; Prof. Ing. arch. Jozef Lacko, (1917-1978); Dipl. technician Imrich Sládkovič, chief construction manager, Doprastav; TV reporter PhDr. J. Koščo (1929-2010). (Photo: Juraj Bončo).



Fig. 3. BS. Information table: General contractor: DOPRASTAV, n.p., Bratislava; Supplier of steel structures and erection: VŽKG, n.p., Ostrava, Hutní montáže, n.p. Ostrava; Realization of structure: DOPRASTAV, n.p., Bratislava; Head of construction, main site manager: Dipl. Technician: Imrich Sládkovič; Begin of construction: November 1967; End of construction: August 1972; General designer: DOPRAVOPROJEKT Bratislava, Ing. Ľudovít Mego; Designer of main structure: Slovenská vysoká škola technická (Slovak University of Technology) in Bratislava: Prof. Dr. Arpád Tesár, DrSc., Prof. Ing. arch. Jozef Lacko; Central investor: ZS – KNV Bratislava; Direct investor: Mestská správa komunikácií v Bratislave, Ing. Karol Štrba. 6.4.1969. (Photo Juraj Bončo).



Fig. 4. SNP Bridge over the Danube river in Bratislava. (Photo Ivan Baláž).



BURLIVÝ ROZVOJ CESTNEJ DOPRAVY SI VYŽIADAL PREMOSTIŤ DUNAJ V BRATISLAVE, HLAVNOM MESTE SLOVENSKA, NOVÝM MOSTOM MEDZI PETRŽALKOU A CENTROM BRATISLAVY. DOTERAJŠIE SPOJENIE CEZ PROVIZÓRNY MOST, VYBUDOVANÝ V ROKU 1946 ČERVENOU ARMÁDOU NA MIESTE VOJNOU ZNIČENÉHO MOSTA, SA SVOJOU KAPACITOU BLÍŽI NAPLNENIU. V SÚLADE SO SMERNÝM ÚZEMNÝM PLÁNOM BRATISLAVY SME SA ROZHODLI POSTAVIŤ MOST V OBLASTI RYBNÉHO NÁMESTIA V HISTORICKY CENNEJ OBLASTI MESTA TAK, ABY TÁTO OBLASŤ PO DOKONČENÍ STAVBY PREMOSTENIA EŠTE VIAC VYNIKLA. NOVÉ PREMOSTENIE UMOŽNÍ RÝCHLU A BEZPEČNÚ VNÚTROMESTSKÚ DOPRAVU A PO DLHÚ DOBU BUDE SLÚŽIŤ I MEDZINÁRODNEJ DOPRAVE PO TRASE E 15.
NOVÝ MOST V BRATISLAVE BUDE PRETÍNAŤ DUNAJ V RIEČNOM KILOMETRI 1869,1 A BUDE NAJVÄČŠÍM OBJEKTOM PREMOSTENIA, KTORÉHO CELKOVÁ DĹŽKA OD NAPOJENIA NA VIEDENSKÚ CESTU V PETRŽALKE PO VÝSTE-

V DOBE KONANIA SLÁVNOSTNÉHO AKTU ZAPOČATIA STAVBY

hlavný predstaviteľ Československej socialistickej republiky
ANTONÍN NOVOTNÝ

predseda vlády Československej socialistickej republiky
JOZEF LENÁRT

predseda Slovenskej národnej rady
MICHAL CHUDÍK

poslanec NR, z predsedov Slovenskej plánovacej komisie
Ing. FRANTIŠEK BARBÍREK

predseda Západoslovenského krajského národného výboru
Ing. ALBERT KOŠTÁL

predseda hlavného národného výboru hlavného mesta Bratislavy
Ing. arch. MILAN HLADKÝ C. Sc.

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Prof. Ing. Dr. JOZEF TROKAN

a členmi autorských kolektívov
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Prof. Ing. arch. JOZEF LACKO
Doc. Ing. JOZEF ZVARA

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Dopravoprojekt Bratislava, ktorého riaditeľom bol
Ing. JAROMÍR ŠEBESTA

hlavným inžinierom projektu
Ing. ĽUDEVÍT MEGO

Generálnym dodávateľom je Dopravný, stavebný podnik hlavného mesta Bratislavy, ktorého riaditeľom bol
Ing. JÁN SEDLÁČEK

Ocelovú nosnú konštrukciu
ŽKOVICKÉ ŽELEZIARNE KLEMENTA GOTTWALDA, NÁRODNÝ PODNIK OSTRAVA



ZAKLADACIA LISTINA NOVÉHO MOSTA CEZ DUNAJ V BRATISLAVE

MCMLXVII

NIE NA SUCHOM MÝTE V BRATISLAVE BUDE MERAŤ 2,5 KILOMETRA. HLAVNÝ NOSNÍK MOSTA O CELKOVEJ DĹŽKE 432 METROV, O VÁHE VIAC AKO 7000 TON, VYTVORENÝ Z OCEĽOVÝCH PLECHOV, MÁ ĽAŽAŤ NA ŠTYROCH PODPORÁCH A VYTVÁRAŤ TRI MOSTNÉ POLIA. STREDNÉ POLE O ROZPÄTÍ 304 METROV BUDE ZAVESENÉ NA TROCH PÁROCH KÁBLOV Z OCELE, PREVEDENÝCH CEZ ŠIKMÝ PYLÓN A ZAKOTVENÝCH V KOTEVNOM BLOKU O VÁHE TAKMER 25 000 TON. HROTY PILOT ZÁKLADOVEJ KONŠTRUKCIE POD PYLÓNOM BUDÚ SIAHAŤ DO HĹBKY 33 METROV POD TERÉNOM A VRCHOL PYLÓNA SA MÁ TÝČIŤ DO VÝŠKY TAKMER 90 m NAD TERÉNOM, VO VÝŠKE ASI 80 m NAD TERÉNOM BUDE NA PYLÓNE KAVIAREŇ A NAD ŇOU VYHLIADKOVÁ PLOŠINA. HLAVNÝ MOSTNÝ OBJEKT, PRIEAHLÉ ESTAKÁDY Z PREDPÄTÉHO BETÓNU A TECHNOLOGICKÉ POSTUPY STAVBY BUDÚ NADĽHO REPREZENTOVAŤ SCHOPNOSTI A ODVAHU TÝCH, ČO MOST NAVRHLI AKO I TÝCH, ČO MOST VYBUDUJÚ.

O STAVBU SA ZVLÁŠŤ ZASLÚŽILI

Vedúci odboru dopravy a spojov Slovenskej plánovacej komisie
Ing. JÁN SLANINKA

poverený Západoslovenského krajského národného výboru
Dr. Ing. ĽUDOVÍT VAŠKOVIČ

predseda dopravnej komisie Mestského národného výboru hlavného mesta Bratislavy
Ing. RUDOLF SUROVÝ

Generálnym riaditeľom stavby je Západoslovenský krajský národný výbor, kde vedúcim odboru dopravy bol
JÁN LABANC

Priamym riaditeľom stavby je Mestský správny komunálnych služieb, ktorej riaditeľom bol
Ing. KAROL SYRBA

vedúcim útvaru výstavby premostenia a priamym riaditeľom
Ing. IGOR SCHOLTZ

Ocelovej nosnej konštrukcie
HUTNÉ MONTÁŽE, NÁRODNÝ PODNIK, OSTRAVA

Základnú výstroj zaistila
VODNÉ STAVBY, NÁRODNÝ PODNIK, PRAHA

ĎAKUJEME TIEŽ VEĽMI MNOHÝM TU NEMENOVANÝM PRACOVNÍKOM, Z RÔZNYCH INŠTITÚCIÍ, KTORÍ SA PODIEĽALI SVOJOU PRÁCOU NA PRÍPRAVE TEJTO VEĽKEJ STAVBY.

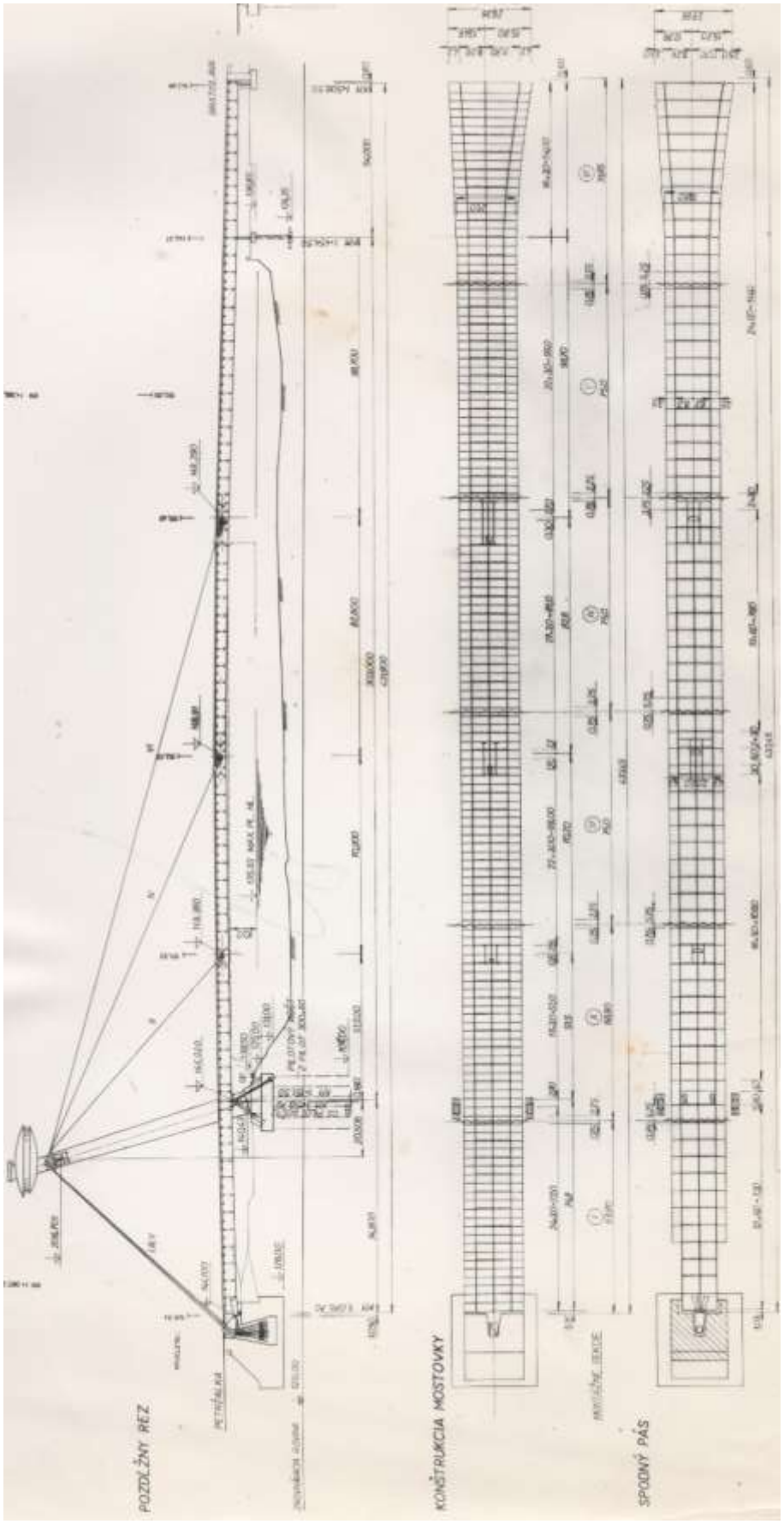
VERÍME, ŽE STAVBA NOVÉHO PREMOSTENIA DUNAJA V BRATISLAVE PRISPEJE K ROZKVETU NAŠEJ DRAHEJ VLASTI A NÁŠHO MESTA.

DANÉ V DEŇ SLÁVNOSTNÉHO ZALOŽENIA STAVBY
DECEMBER 1967

M. Hladký

Ing. arch. MILAN HLADKÝ CSc.
predseda MNV

Fig. 5. Charter of SNP Bridge over the Danube river in Bratislava. 1967. [15].
Prof. Trokan is on Fig. 137b, Prof. Tesár on Fig. 77, 96a, 98a, 137a, Assoc. Prof. Zvara on Fig. 137b, Prof. Lacko and Ing. Scholtz are on Fig. 2b, Ing. arch. Hladký is on Fig. 2a.



Longitudinal section – Deck structure – Bottom flange structure

Fig. 6. SNP Bridge in Bratislava over the Danube river. Spans: 74,8 m + 303 m + 54 m. (Elaborated by Ing. Zoltán Agócs, KKDK SvF STU).



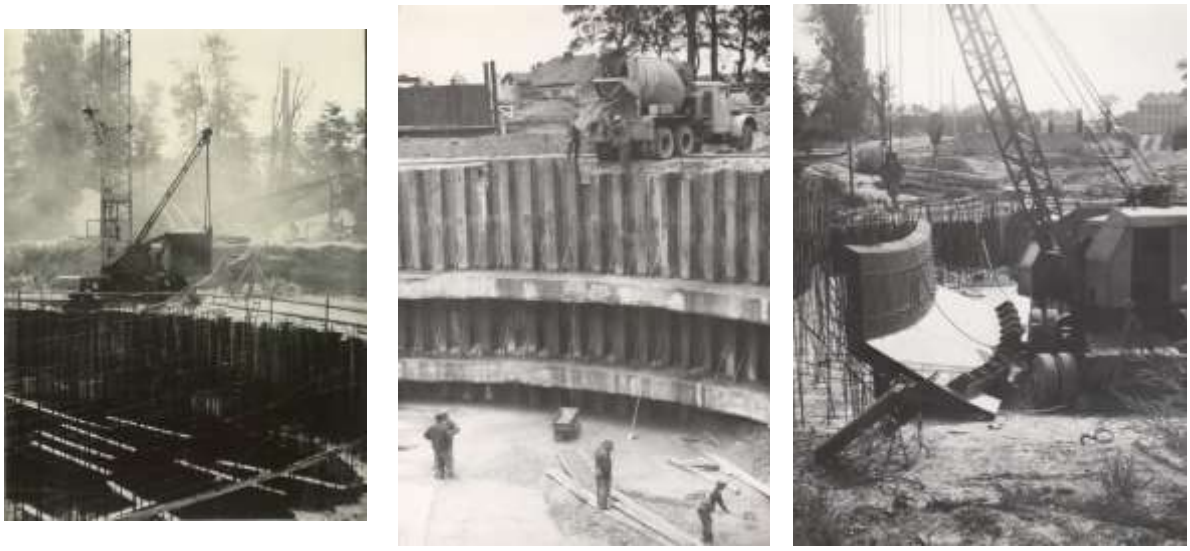
a) b)

Fig. 7. PS. a) and b) Some of vehicles used during construction of SNP Bridge. (Photo Igor Scholtz).



a) b) c)

Fig. 8. PS. a), b) and c): Armature in the pit for the anchorage block. (Photo Igor Scholtz).



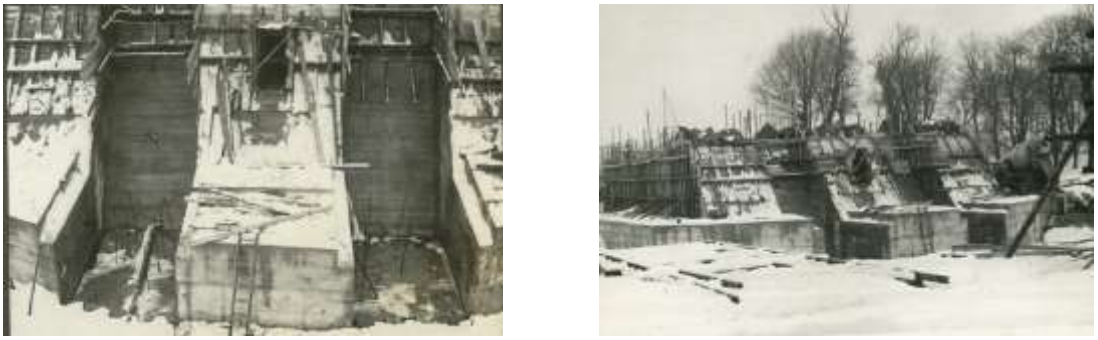
a) b) c)

Fig. 9. PS. a) Armature in the pit for the anchorage block. (Photo Igor Scholtz); b) and c): Pit, enclosure (ø 34 m) and foundation for anchorage block. (Photo Milada Maľcevová – Brucknerová, KKDK SvF STU).



a) b)

Fig. 10. PS. a) Foundation for the anchorage chamber. (Photo Igor Scholtz); b) Pit for anchorage block. (Photo Milada Maľcevová – Brucknerová, KKDK SvF STU).



a) b)

Fig. 11. PS. View from Petržalka on the: a) tunnel leading inside anchorage chamber and on anchorage chamber shafts for lines of engineering networks, b) anchorage block. 4.12.1968. (Photo Igor Scholtz).



a) b) c)

Fig. 12 PS. a) Armature to the guide walls of grab (0,6 m x 3m). (Photo Igor Scholtz); b) Measuring of pilot pit. (Photo Milada Maľcevová – Brucknerová, KKDK SvF STU); c) Excavation of the pits for pilots. (Photo Milada Maľcevová – Brucknerová, KKDK SvF STU).



a) b) c)

Fig. 13. PS. a) Excavation of the pits for pilots. (Photo Milada Maľcevová – Brucknerová, KKDK SvF STU); b) Loading circle for measurement of depth and verticality of pilot. (Photo Igor Scholtz); c) Test pilot No.25 preparation. (Photo Milada Maľcevová – Brucknerová, KKDK SvF STU).

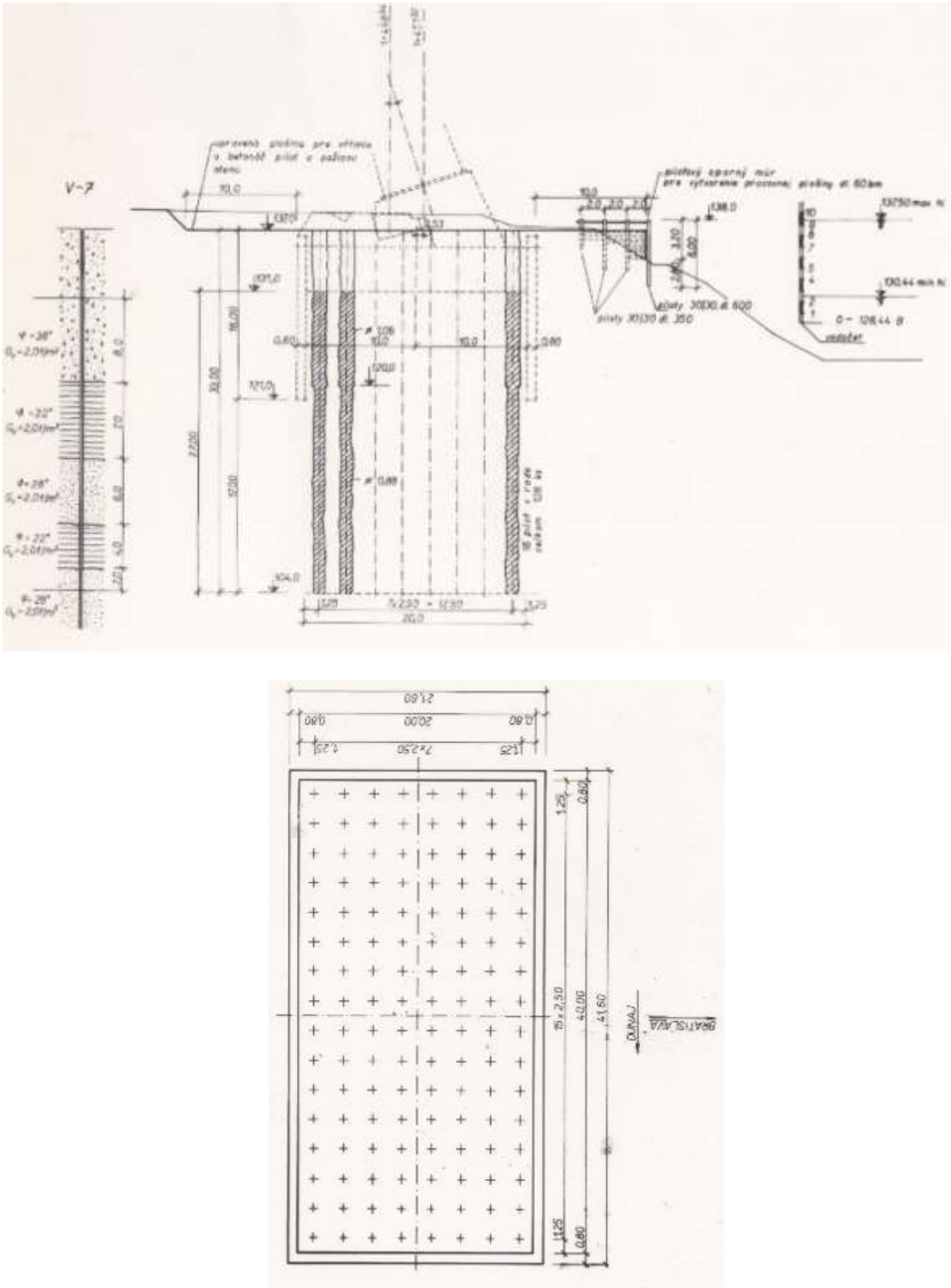


Fig. 14. PS. Proposal for pylon foundation on pilots.
(Elaborated by Assoc. Prof. Jozef Zvara, KBKM SvF STU).

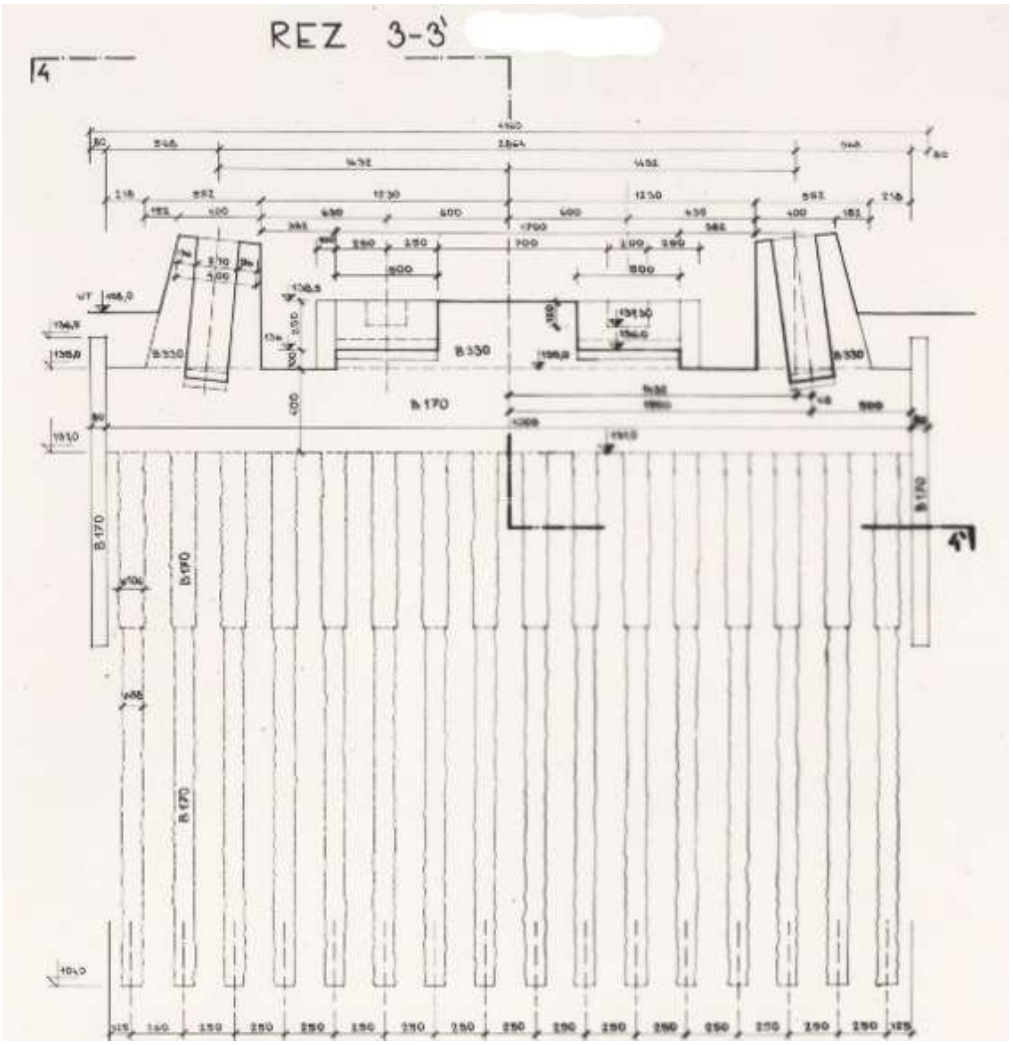


Fig. 15. PS. Proposal for pylon foundation on pilots.
(Elaborated by Ing. Štefan Baďura, KBKM SvF STU).

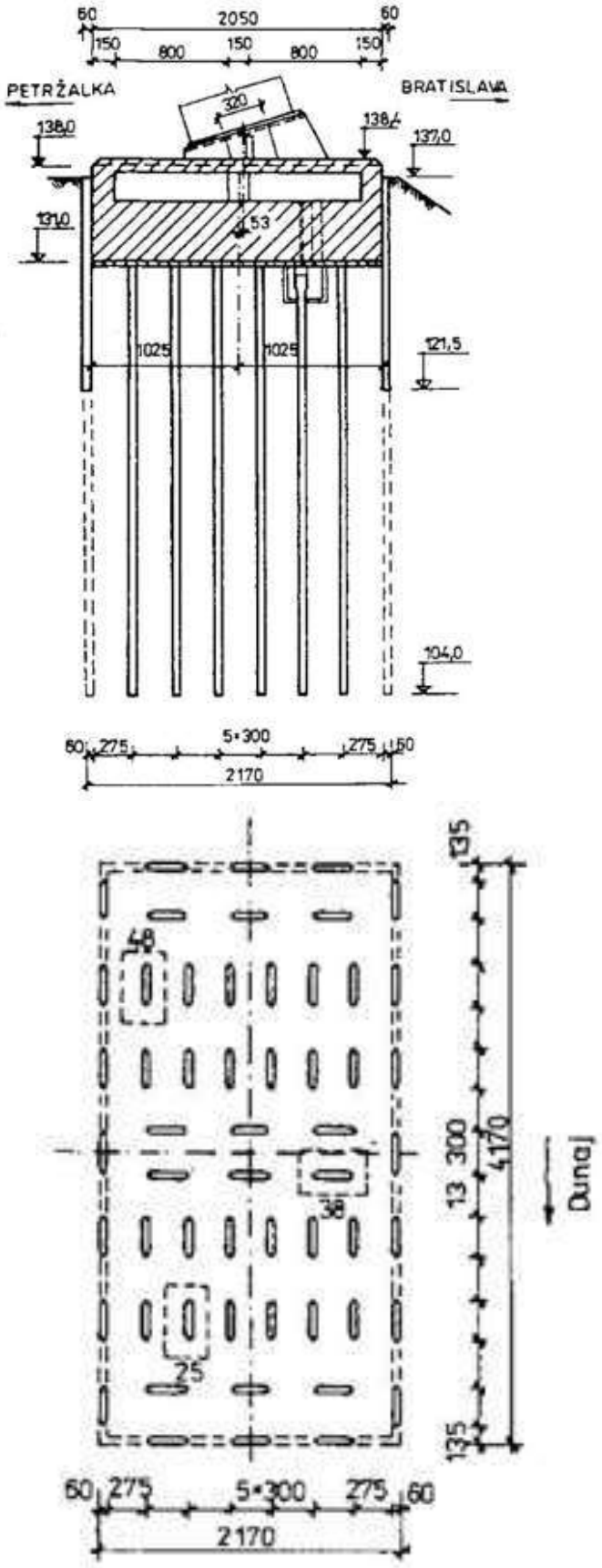


Fig. 16. Realised pylon foundation on pilot gtill.



a) b) c)

Fig. 17. PS. a) and b): Armature of test pilot No.25; c) Electrical cables for test pilot No.25. (Photo Milada Maľcevová – Brucknerová, KKDK SvF STU).



a) b) c)

Fig. 18 PS. a) Electrical cables for test pilot No.25. Assoc. Prof. Ing. Dušan Majdúch, CSc., (KBKM), is third from the right. (Photo Milada Maľcevová – Brucknerová, KKDK SvF STU); b) and c): Installation of the part of the fixed support of the bridge box-girder. (Photo Igor Scholtz).



a) b)

Fig. 19. PS. a) Pit for anchorage block in the foreground and pit for pylon foundation in the background; b) Armature for pylon foundation. (Photo Igor Scholtz).

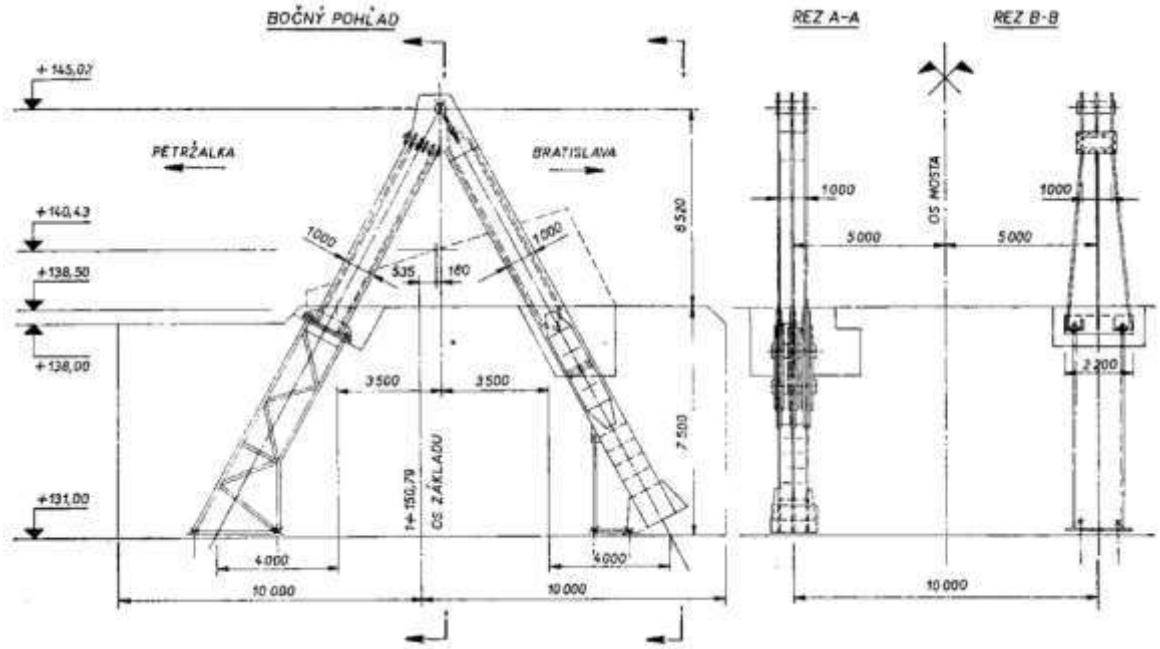
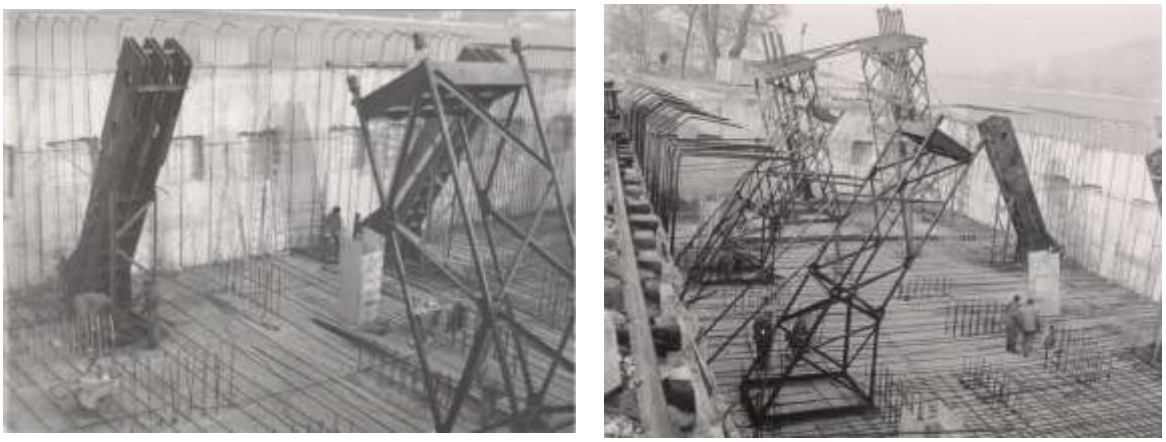


Fig. 20. PS. Steel structure of the fixed support of the bridge box-girder. (Elaborated by Assoc. Prof. Pavel Dutko, CSc., KKDK SvF STU). [1].



a) b)

Fig. 21. PS. a) Part of the fixed support of the bridge box-girder between armatures of the foundations of the left and right leg of pylon; b) Foundation slab of pylon and armature of pylon foundation. (Photo Igor Scholtz).



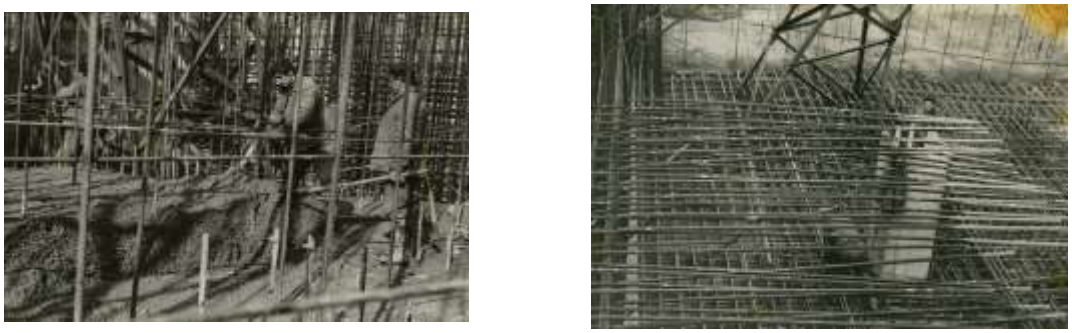
a) b)

Fig. 22. PS. a) and b): Foundation slab of pylon, armature of pylon foundation and part of the fixed support of the bridge box-girder. (Photo KKDK SvF).



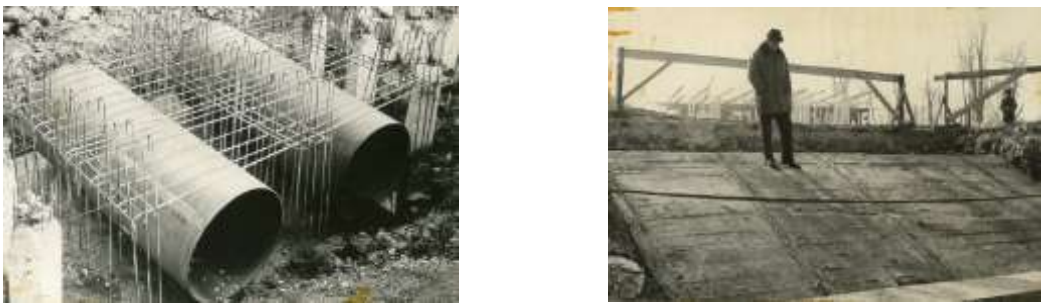
a) b) c)

Fig. 23. PS. a) Armature of pylon foundation; b) Parts of fixed support of the bridge box-girder; c) Base of temporary support of pylon. (Photo Igor Scholtz).



a) b)

Fig. 24. PS. a) and b) Armature and concreting of pylon foundation. (Photo Igor Scholtz).



a) b)

Fig. 25. BS. a) Water pipe protectors; PS b) Foundation of temporary support of pylon; (Photo Igor Scholtz).



a) b)

Fig. 26. PS. a) Armature of pylon foundation;
b) Flyover on the right bank of Danube river. 6.4.1969. (Photo Juraj Bončo).

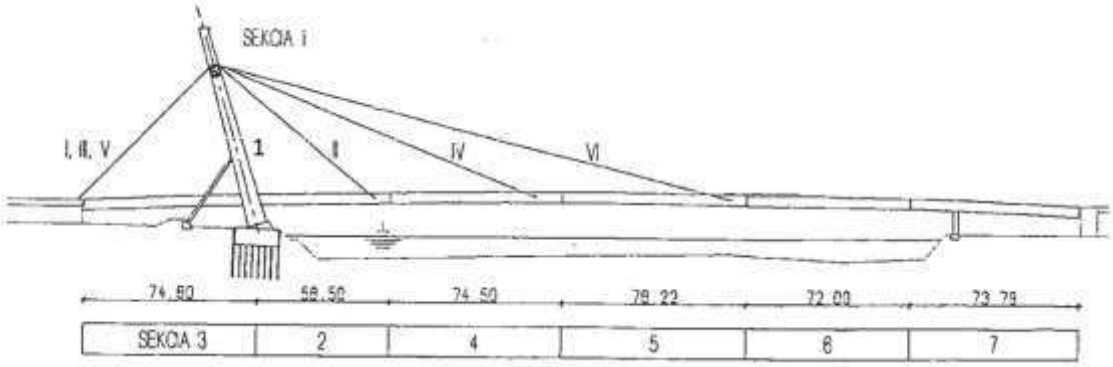
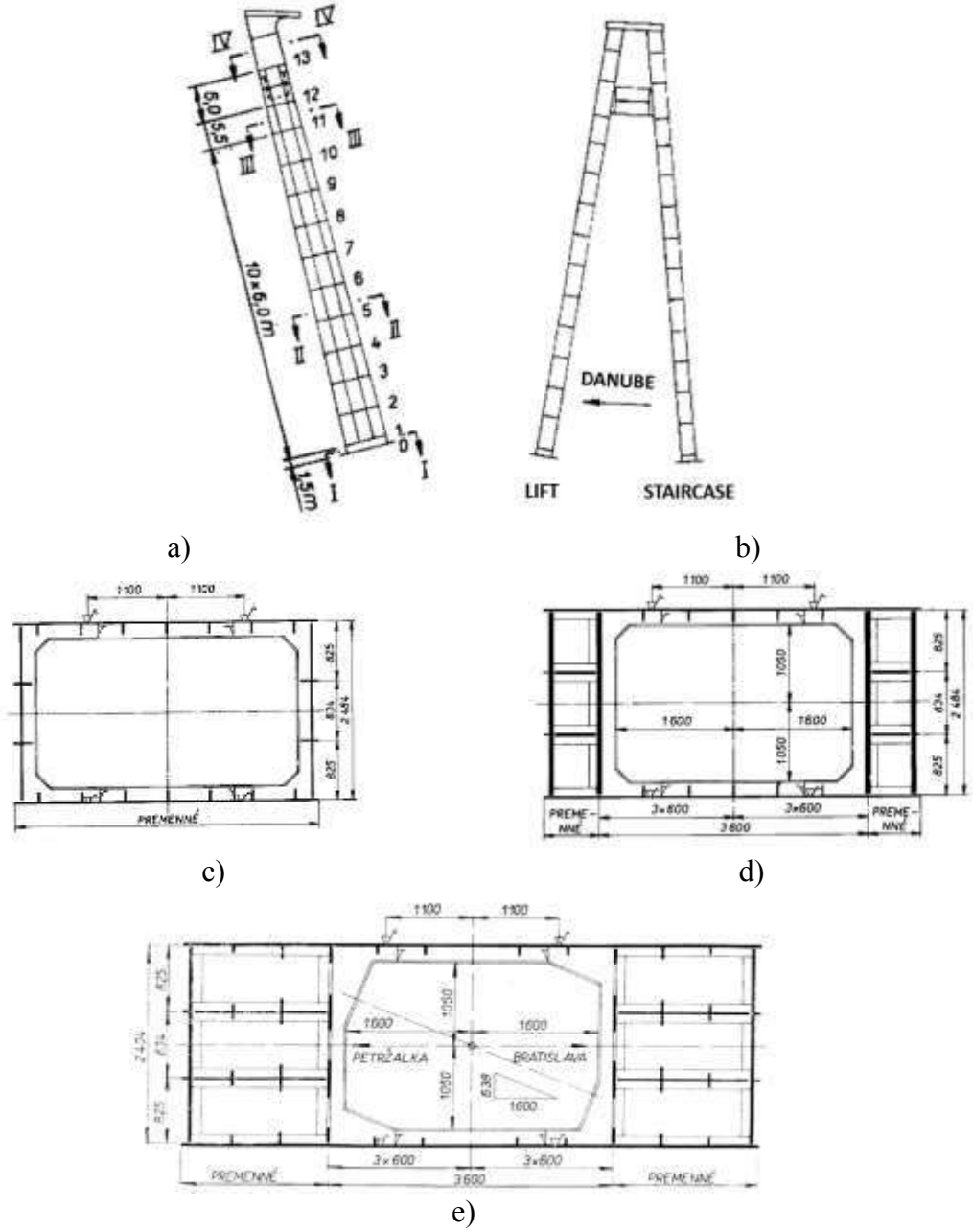


Fig. 27. Cables I, III, V and cables II, IV, VI. Chronological numbering of construction bridge segments: segment No.1 (pylon section), segments No.2-7 (bridge box-girder segments).



a) b) c) d) e)

Fig. 28. PS. PS. a) Pylon sections; b) Pylon legs Pylon cross-sections: c) IV-IV; b) III-III; d) II-II. (Elaborated by Assoc. Prof. Ing. Pavel Dutko, CSc., KKDK SvF STU). [1].

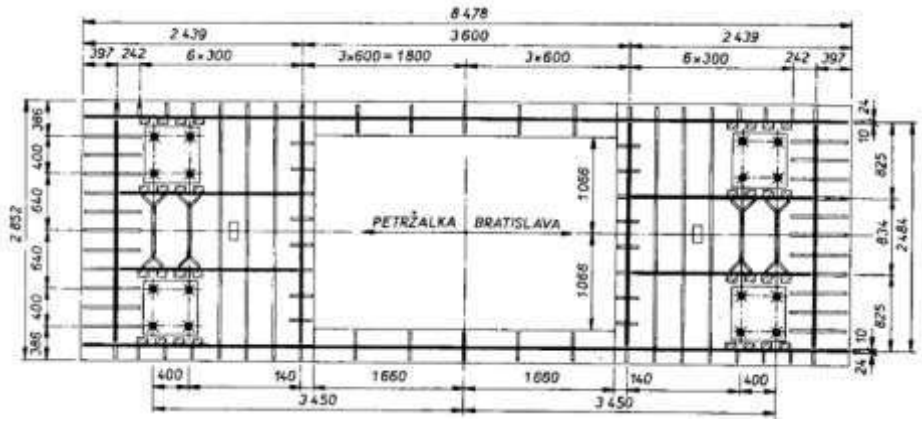
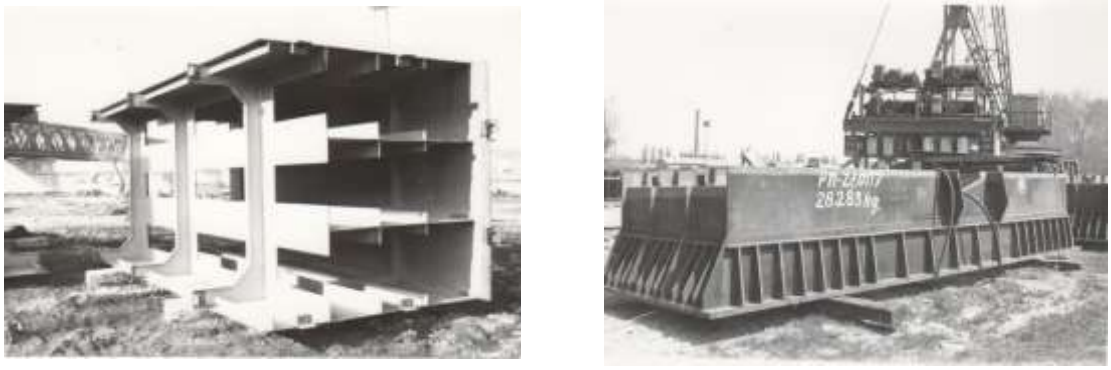


Fig. 29. PS. Pylon cross-sections I-I.
(Elaborated by Assoc. Prof. Ing. Pavel Dutko, CSc., KKDK SvF STU). [1].



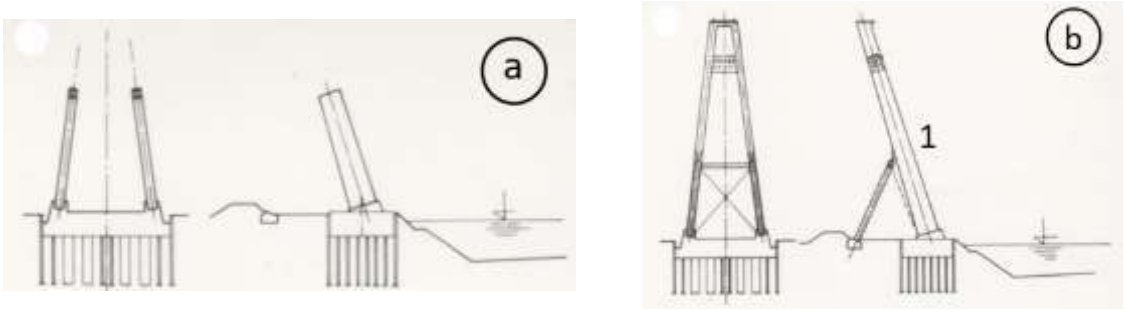
a) b)

Fig. 30. PS. a) Elements of pylon; b) Base of pylon. (Photo KKDK SvF).



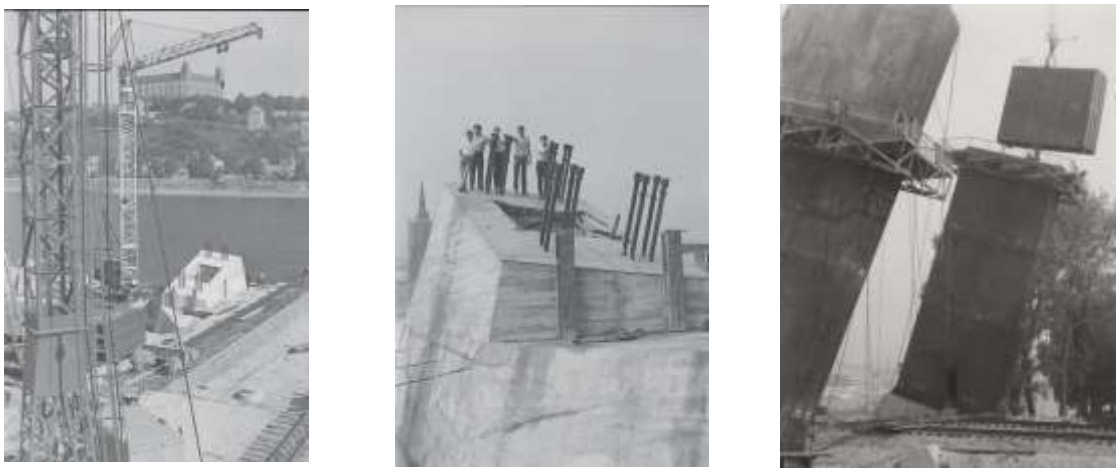
a) b)

Fig. 31. PS. a) and b) Elements of pylon. 29.7.1969. (Photo Juraj Bončo).



a) b)

Fig. 32. PS. Construction of pylon (segment No.1): a) Phase a; b) Phase b.
(KKDK SvF STU). [1].



a) b) c)

Fig. 33. PS. a) and b) Foundation of pylon. 29.7.1969. (Photo Juraj Bončo);
c) Construction of pylon. Phase a. (Photo KKDK SvF STU).



a) b)

Fig. 34. a) Checking of pylon legs in production hall; b) Construction of pylon. Phase a. (Photo KKDK SvF STU).



a) b)

Fig. 35. PS. a) Grinding the cracks in transverse beam of pylon; b) Transverse beam of pylon with cracks. Legs of temporary support of pylon. (Photo Igor Scholtz).



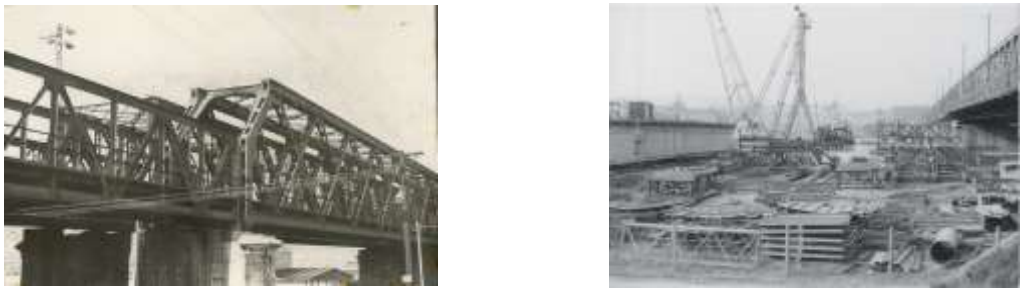
a) b)

Fig. 36. PS. Phase a. a) Construction of pylon. Crane BK 1000. (Photo Igor Scholtz);
b) Construction of pylon. (Photo Dnes 24).



a) b) c)

Fig. 37. PS. a) and b): Construction of pylon. Crane BK 1000; c) Upper parts of the pylon for entrance in the restaurant. (Photo Igor Scholtz).



a) b)

Fig. 38. PS. a) Old Red Army Bridge where parts of the bridge were unloaded from wagons (Photo Igor Scholtz); b) Site near Red Army Bridge where the parts of bridge box-girder were prepared for launching onto pontoons. (Photo KKDK SvF STU).



a) b)

Fig. 39. PS. a) Site near Red Army Bridge. Parts of bridge girder;
b) Transverse beam of pylon at pylon leg. (Photo Igor Scholtz).



a) b)

Fig. 40. PS. Site near Red Army Bridge: a) Elements of cable saddle; b) Cranes. (Photo Igor Scholtz).



a) b) c)

Fig. 41. PS. Construction of pylon: a) and b). (Photo Igor Scholtz);
c) 12.4.1970. (Photo Juraj Bončo).



a) b)

Fig. 42. PS. a) Construction of pylon. 19.4.1970. (Photo Juraj Bončo); 2
b) Cables saddle. (Photo Igor Scholtz).



a) b)

Fig. 43. PS. Construction of pylon: a) 14.6.1970. (Photo Juraj Bončo);
b) (Photo KKDK, SvF).



a) b)

Fig. 44. PS. Anchorage block. a) (Photo KKDK SvF STU); b) Later in phase d. (Fig. 64). (Photo Igor Scholtz).



a) b)

Fig. 45. a) BS. Construction of flyover on the left bank of Danube. (Photo Igor Scholtz); b) Site near Red Army Bridge. Store of bridge components and assembling of bridge box-girder segments. (Photo KKDK SvF STU).



a) b)

Fig. 46. BS. Construction of flyover on the left bank of Danube: a) 14.6.1970; b) 21.3.1971. (Photo Juraj Bončo).



a) b)

Fig. 47. BS. Construction of flyover on the left bank of Danube: a) 21.3.1971. (Photo Juraj Bončo); b) (Photo KKDK SvF STU).

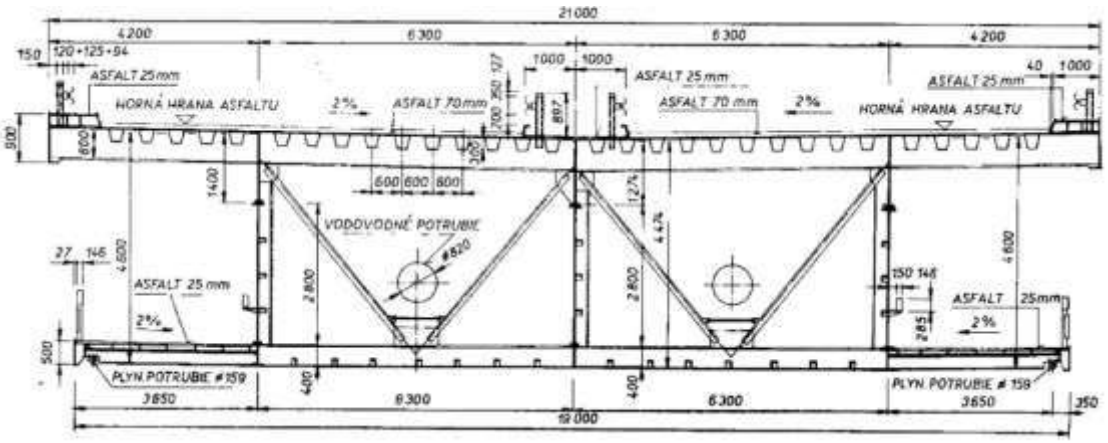
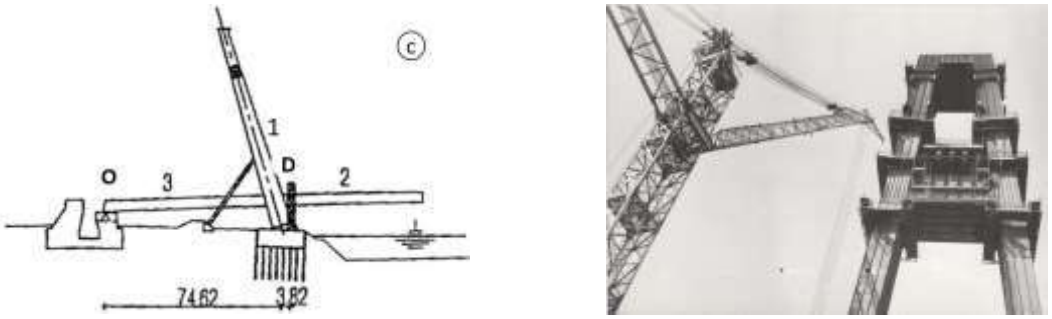


Fig. 48. PS. a) and b). Bridge box-girder cross-section. (KKDK SvF STU).



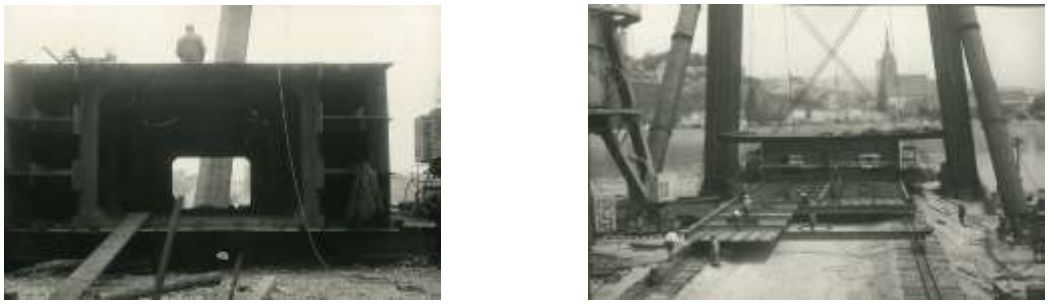
a) b)

Fig. 49. PS. a) and b). Bridge box-girder assembling. (Photo KKDK SvF STU).



a) b)

Fig. 50. PS. a) Construction of bridge. Phase c. Assembling and launching of the bridge box-girder segments No.2 and 3. Placing them on temporary supports O and D. The weight of segments No.2 and 3 was approximately 1400 tons and cantilever parts of the bridge box-girder was 53 m. (KKDK SvF [1]); b) Head of the pylon (Photo KKDK SvF STU).



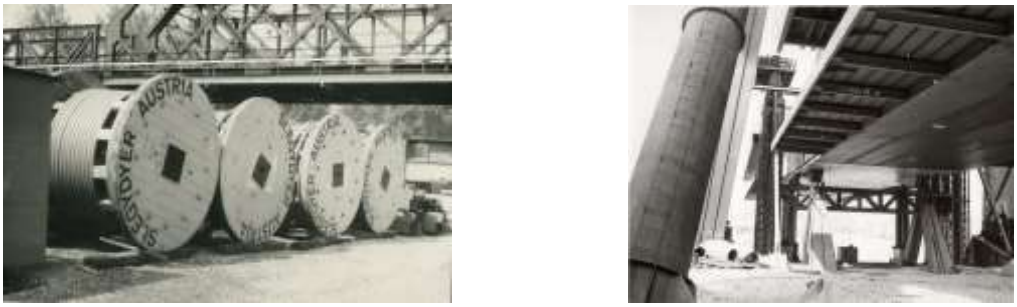
a) b)

Fig. 51. PS. a) and b) Preparation for bridge box-girder launching in phase c. (Photo Igor Scholtz).



a) b)

Fig. 52. PS. a) Welding of bridge box-girder; b) Bridge box-girder assembling. (Photo Igor Scholtz).



a) b)

Fig. 53. PS. Site near Red Army Bridge: a) Cables. (Photo Igor Scholtz); b) Bridge construction. Phase c. (Photo KKDK SvF STU).



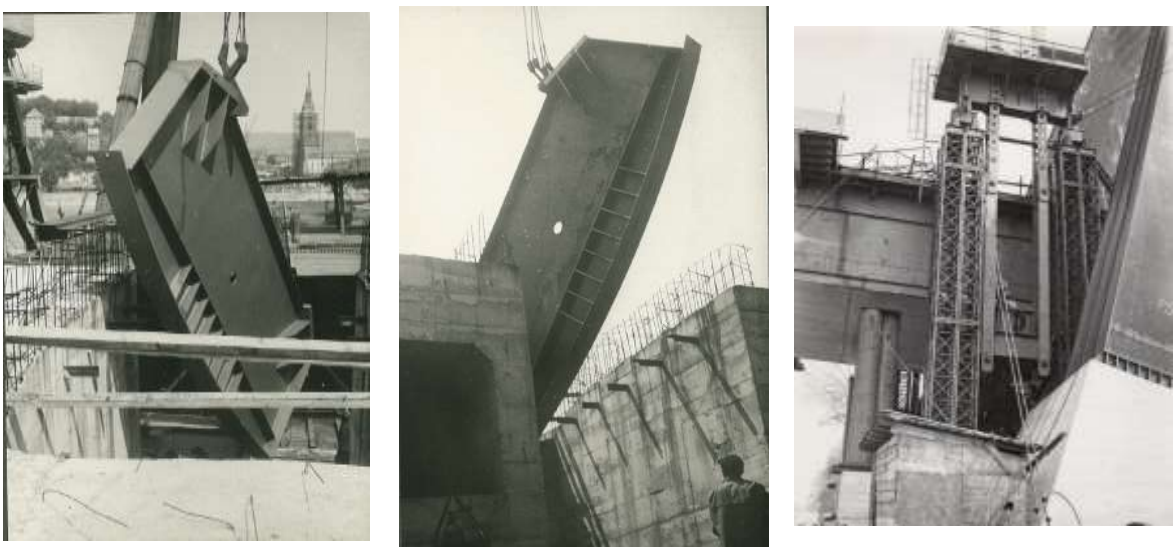
a) b) c)

Fig. 54. PS. a) Site near Red Army Bridge. Store of bridge components and assembling of bridge box-girder segments. Cables; b) Preparation of bridge box-girder for launching in phase c. (Photo Igor Scholtz); c) (Photo KKDK SvF STU).



a) b)

Fig. 55. PS. a) and b) Construction of bridge in phase c (Fig. 50). (Photo Igor Scholtz).



a) b) c)

Fig. 56. PS. a) and b). Anchorage block. Installation of anchorage element into anchorage chamber. (Photo Igor Scholtz); c) Bridge box-girder construction. (Photo KKDK SvF STU).

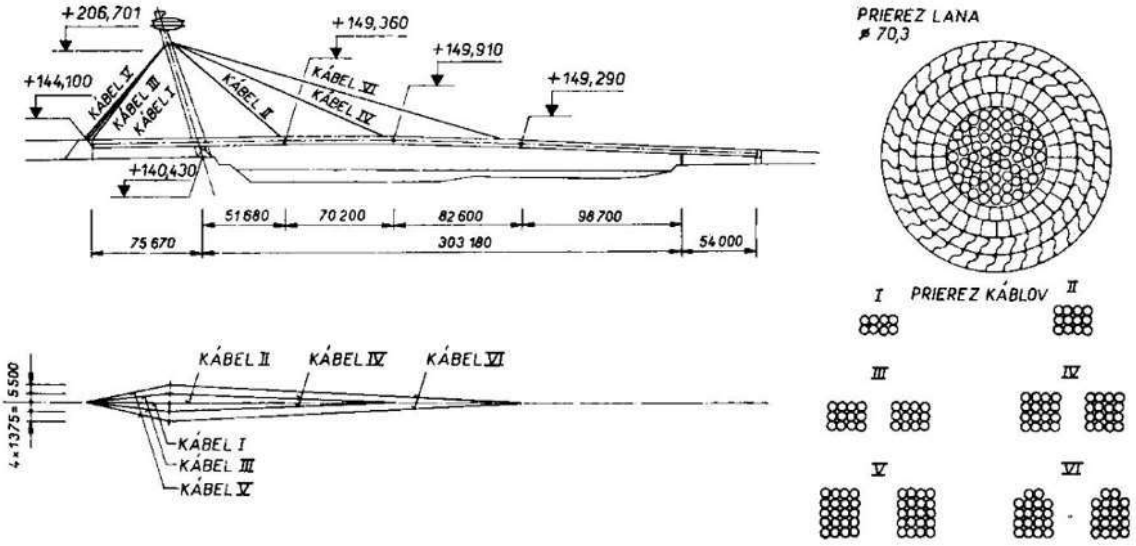
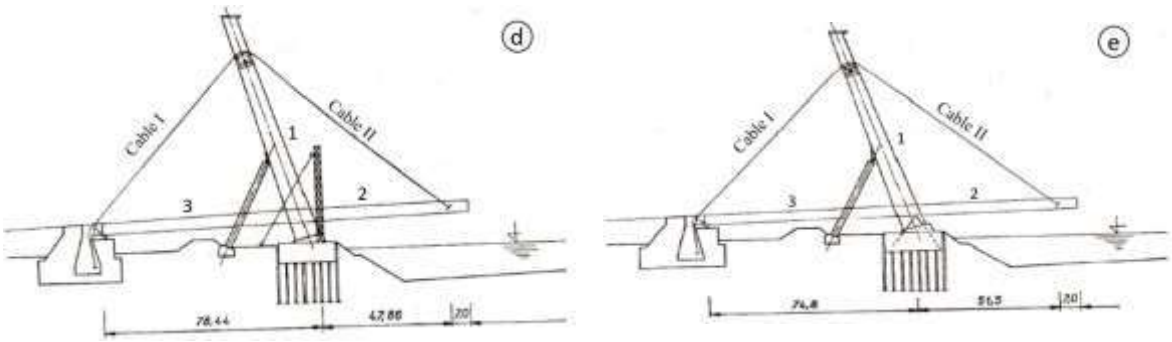


Fig. 57. Arrangement of cables I – VI. [1].



a) b)

Fig. 58. PS. Construction of bridge: a) Phase d; b) Phase e. [1].



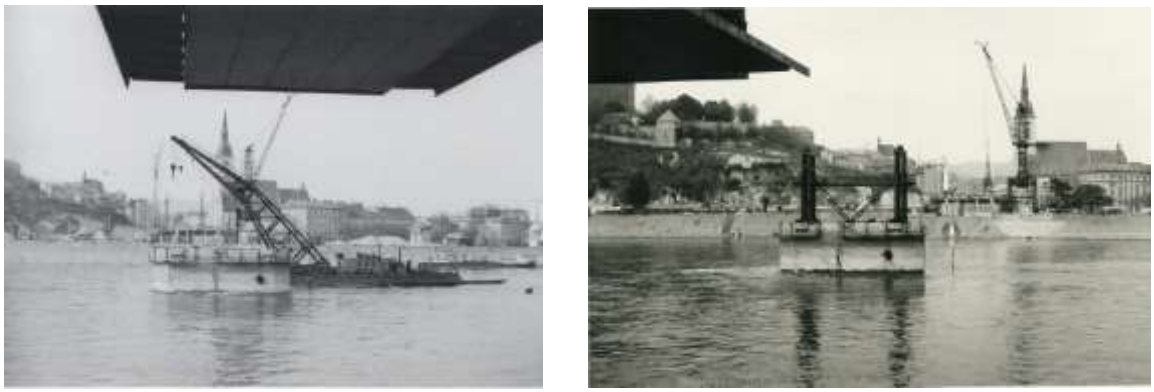
a) b) c)

Fig. 59. PS. Bridge construction. Phase d. Preparation of installation of:
a) Cable II; b) Cable I and II; c) Cable I. (Photo KKDK SvF).



a) b) c)

Fig. 60. PS. Phase d. In the foreground preparation of installation of:
a) Cable II; b) Preparation of installation of cable I and cable II;
c) Preparation of installation of cable I and installation of cable II. (Photo Igor Scholtz).



a) b)

Fig. 61. PS. a) Temporary bridge girder support A (phase f on Fig. 92). 10.4.1971.
(Photo Juraj Bončo); b) Temporary support A. (Photo Igor Scholtz).



a) b)

Fig. 62. PS. Phase d. Preparation of installation of:
a) cable II; b) cable II. Temporary support A. (Photo KKDK SvF STU).



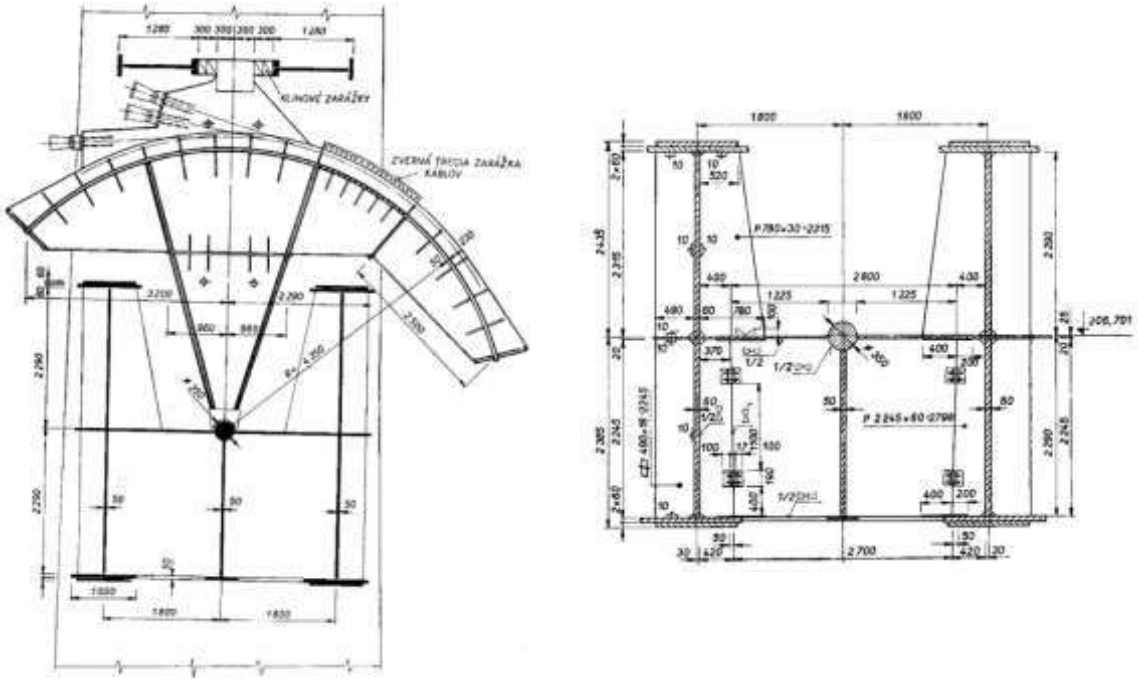
a) b)

Fig. 63. PS. a) and b) Bridge construction. Phase d. Preparation of installation of cable II.
(Photo KKDK SvF STU).



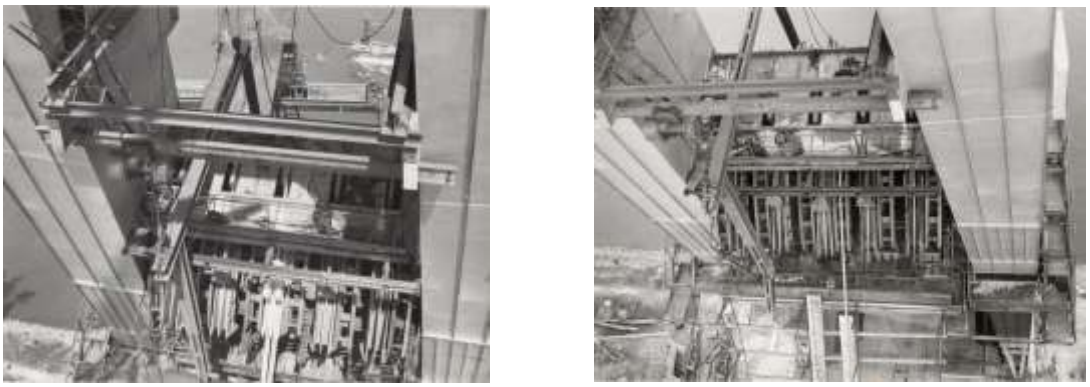
a) b)

Fig. 64. PS. a) Beam carrying cables saddle. b) Installation of cable I in phase d.
(Photo KKDK SvF STU).



a) b)

Fig. 65. PS. a) Cables saddle (KKDK SvF STU);
b) Transverse beam of pylon carrying cables saddle.
(Elaborated by Ing. Robert Fodor, KKDK SvF STU). [1].



a) b)

Fig. 66. PS. a) and b). Beam carrying cables saddle. Installation of cable I in phase d (see Fig. 58a). (Photo KKDK SvF STU).



a) b)

Fig. 67. PS. a) and b) Bridge construction. Phase d. Installation of cable I.
(Photo KKDK SvF STU).



a)



b)

Fig. 68. PS. a) and b) Bridge construction. Phase d. Installation of cable I. (Photo KKDK SvF STU).



a)



b)

Fig. 69. PS. a) and b) Bridge construction. Phase d. Installation of cable I. (Photo KKDK SvF STU).

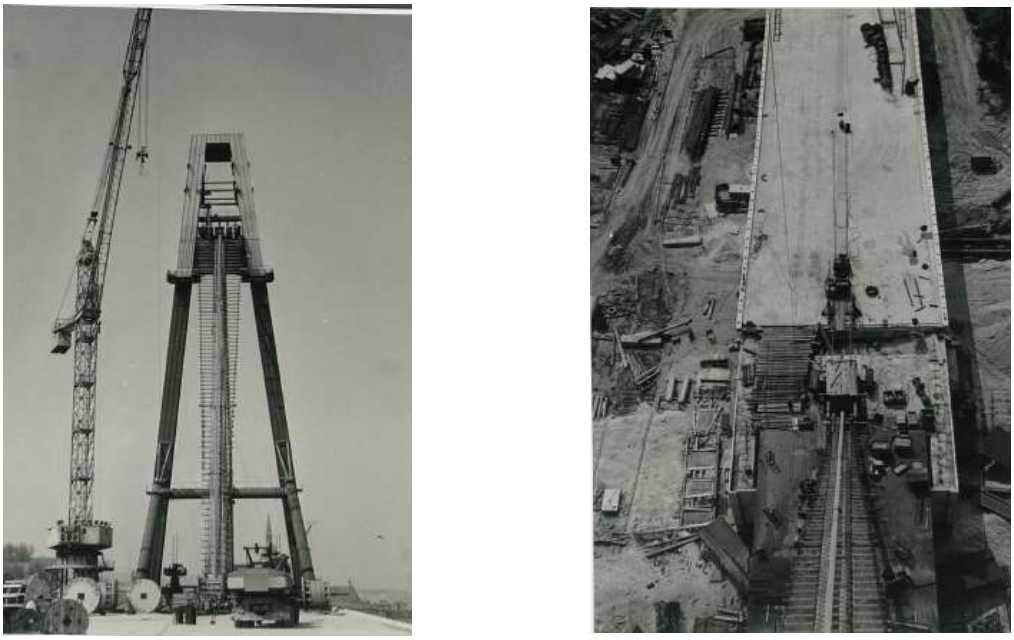


a)



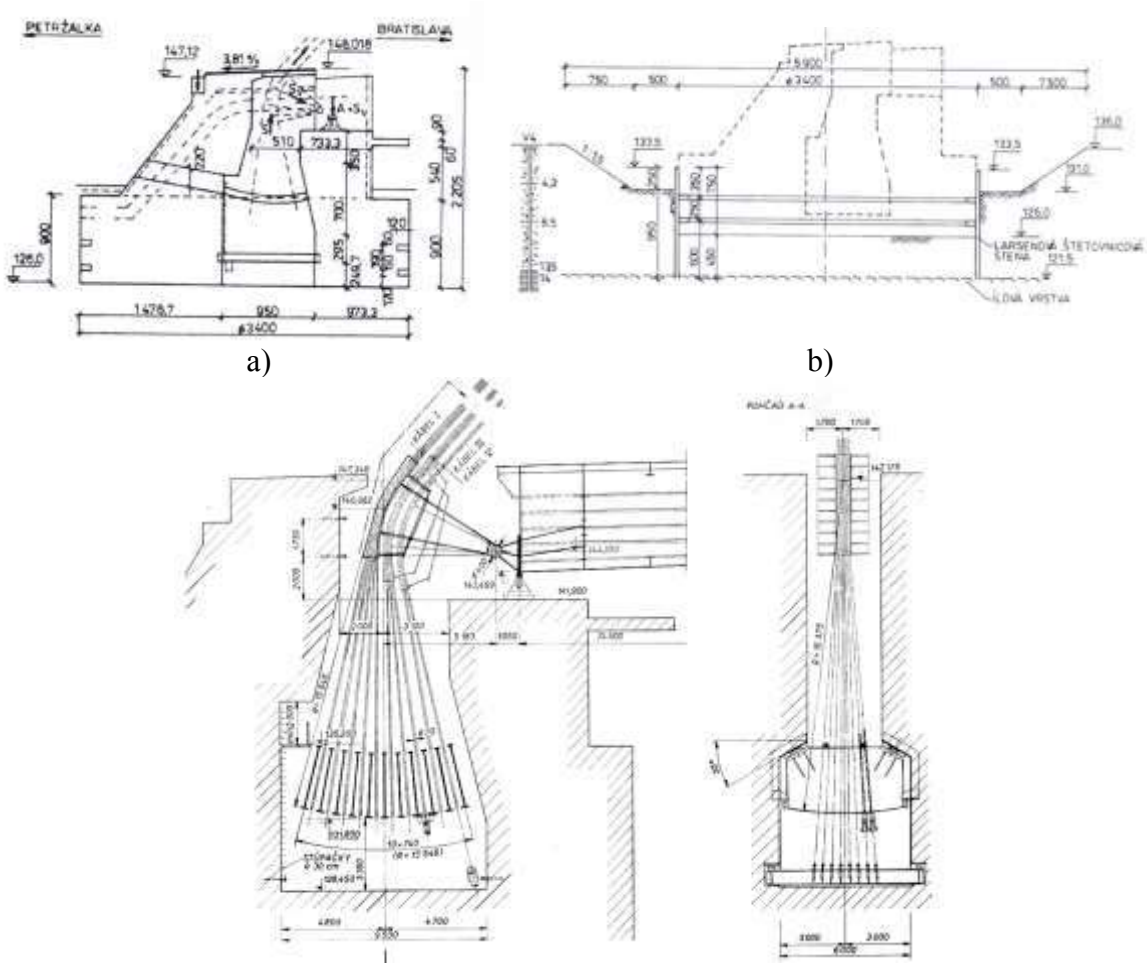
b)

Fig. 70. PS. Bridge construction. Phase d. Installation of:
a) cable I (Photo Igor Scholtz); b) cable II. (Photo KKDK SvF STU).



a) b)

Fig. 71. PS. a) and b) Bridge construction. Phase d. Installation of cable I. (Photo Igor Scholtz).



a) b)

c)

Fig. 72. PS. a) and b) Anchorage block; c) Anchorage of the cables in anchorage chamber and end support of bridge box-girder. (Elaborated by Ing. Jozef Lapos and Ing. Juraj Virčík, KKDK SvF STU).



a) b) c)

Fig. 73. PS. a) Detail of bridge box-girder end support. (Photo KKDK SvF STU);
b) Anchorage of cables inside of anchorage chamber. (Photo Igor Scholtz);
c) Installation of cable I. (Photo KKDK SvF STU).



a) b)

Fig. 74. PS. a) End support of bridge box-girder. (Photo KKDK SvF STU).
b) Anchorage of cables inside of anchorage chamber. (Photo Igor Scholtz).



a) b) c)

Fig. 75. PS. a) and b). Phase d. Installation of cable II. The temporary support A;
c) Installation of cable I. (Photo Igor Scholtz).



a) b)

Fig. 76. PS. Site near Red Army Bridge: a) (Photo KKDK SvF STU); b) Structure for lifting and launching of bridge box-girder segments. (Photo Igor Scholtz).



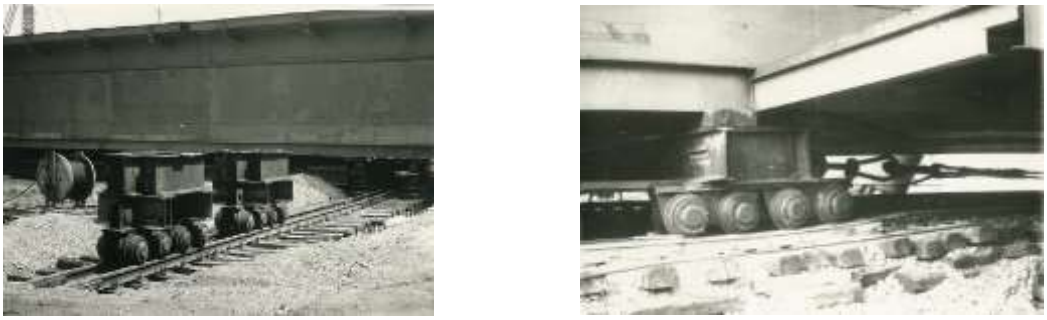
a) b)

Fig. 77. First launching of bridge box-girder segment by Prof. Dr. Ing. Arpád Tesár, DrSc. (Photo KKDK SvF STU).



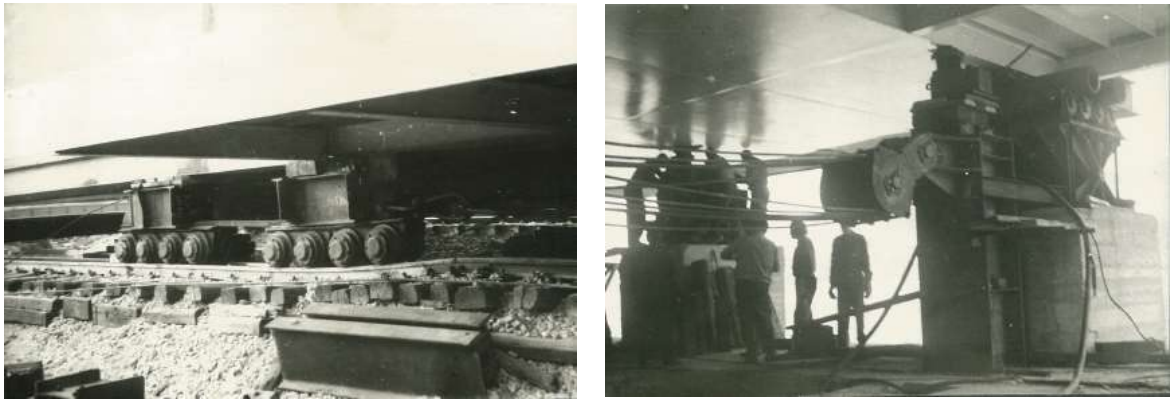
a) b)

Fig. 78. PS. Site near Red Army Bridge: a) Assembling of bridge box-girder segments; c) Preparation for bridge box-girder launching. (Photo Igor Scholtz).



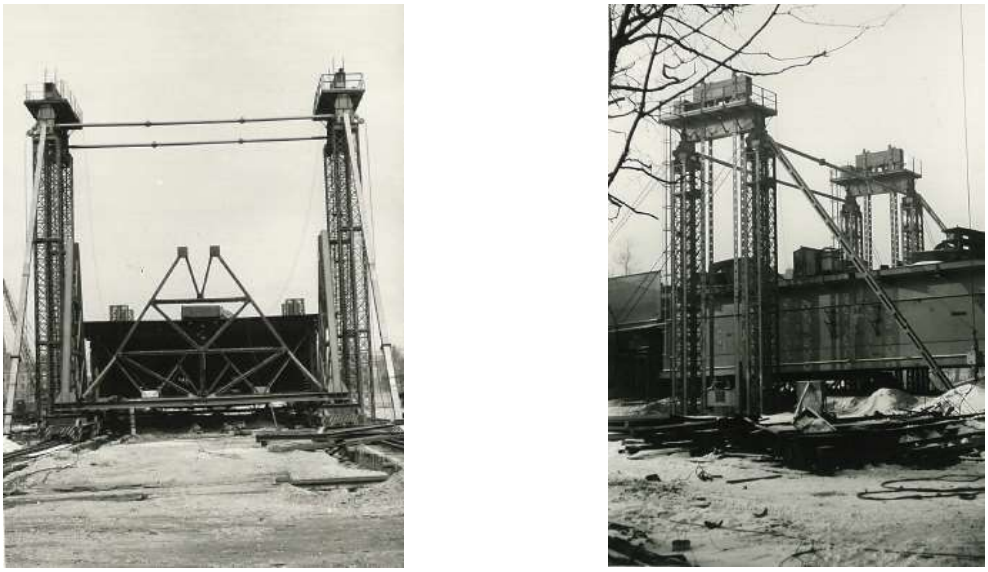
a) b)

Fig. 79. PS. a) and b). Site near Red Army Bridge. Mechanisms for bridge box-girder launching. (Photo Igor Scholtz).



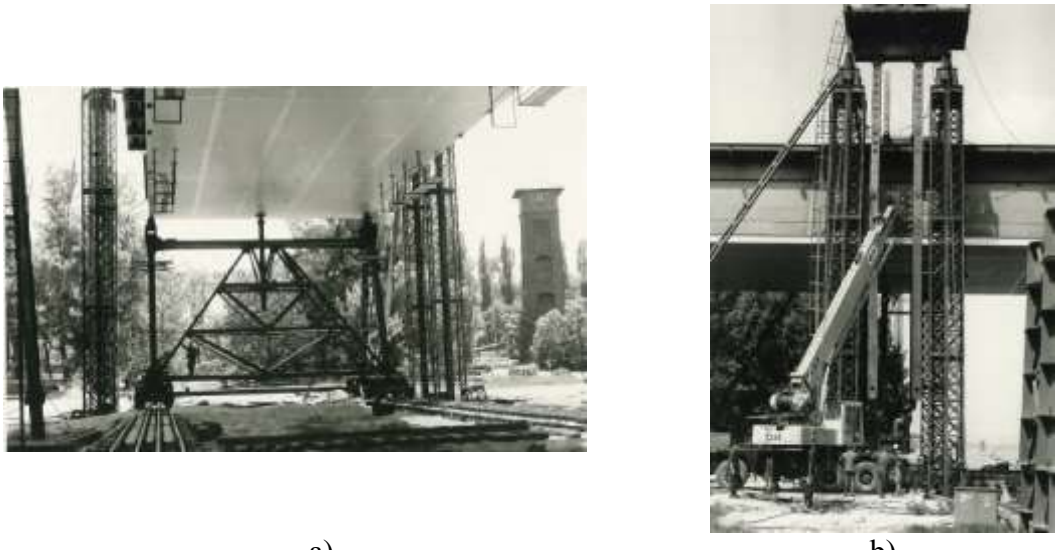
a) b)

Fig. 80. PS. a) and b). Site near Red Army Bridge. Mechanisms for bridge box-girder launching. (Photo Igor Scholtz).



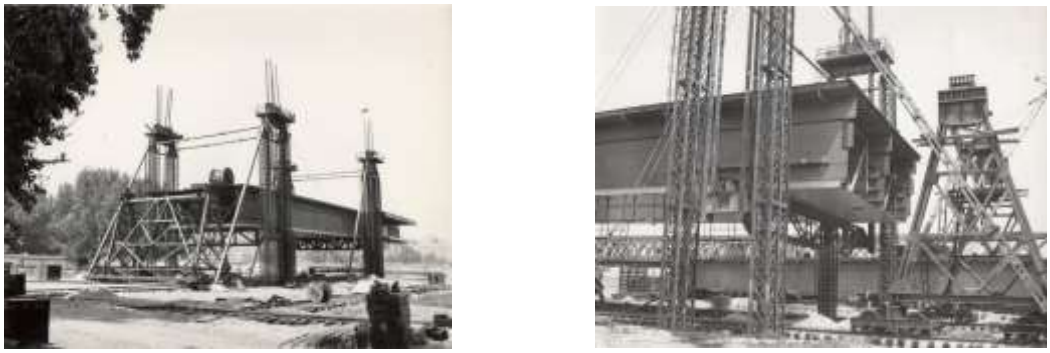
a) b)

Fig. 81. PS. a) and b). Site near Red Army Bridge. Preparation for lifting of bridge box-girder. (Photo Igor Scholtz).



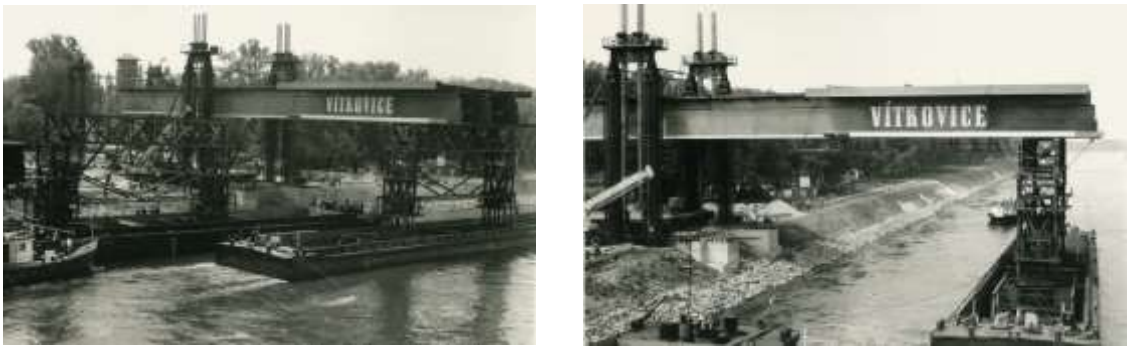
a) b)

Fig. 82. PS. a) and b). Site near Red Army Bridge. Lifted bridge box-girder segment prepared for launching. (Photo Igor Scholtz).



a) b)

Fig. 83. PS. Site near Red Army Bridge. Lifted bridge box-girder segment prepared for launching: a) (Photo Igor Scholtz); b) (Photo KKDK SvF STU).



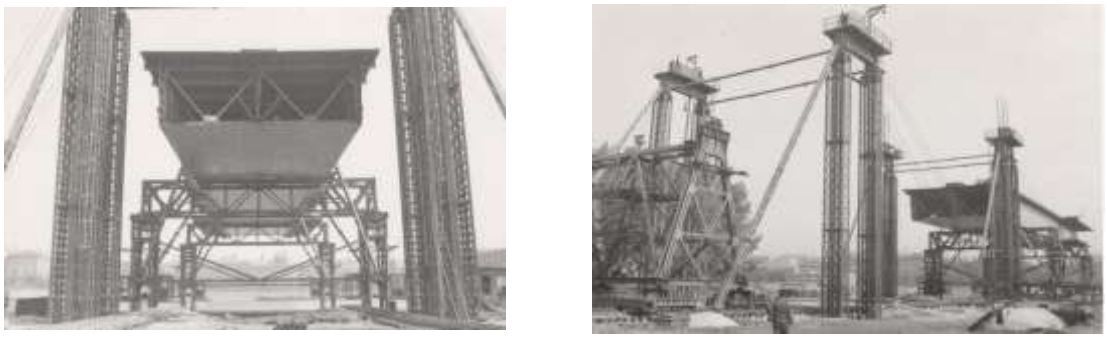
a) b)

Fig. 84. PS. a) and b). Site near Red Army Bridge. Launching of bridge box-girder segment (title VÍTKOVICE). (Photo Igor Scholtz).



a) b)

Fig. 85. PS. a) and b). Site near Red Army Bridge. Launching of bridge box-girder segment (title VÍTKOVICE). (Photo Igor Scholtz).



a) b)

Fig. 86. PS. a) and b). Site near Red Army Bridge. Bridge box-girder segment (title VÍTKOVICE) put on pontoons after launching. (Photo KKDK SvF STU).



a) b)

Fig. 87. a) and b). Transport of bridge box-girder segment (title VÍTKOVICE) on pontoons. (Photo KKDK SvF).

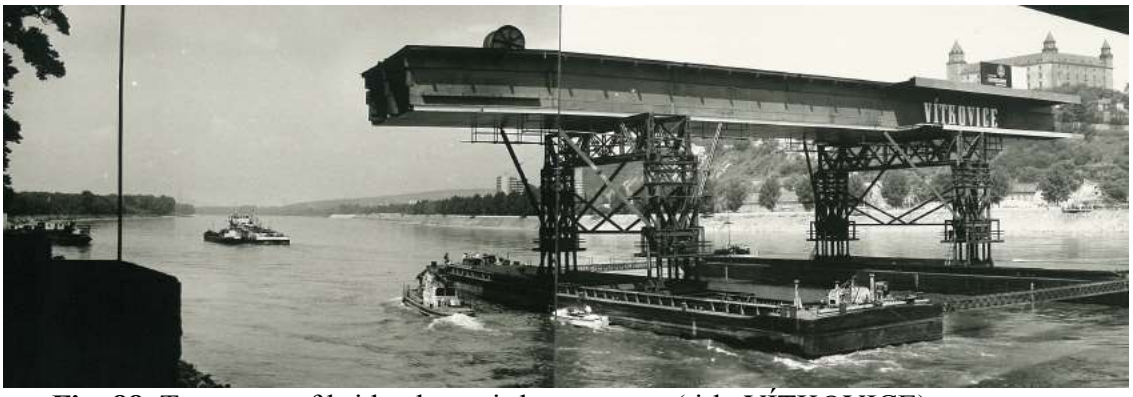
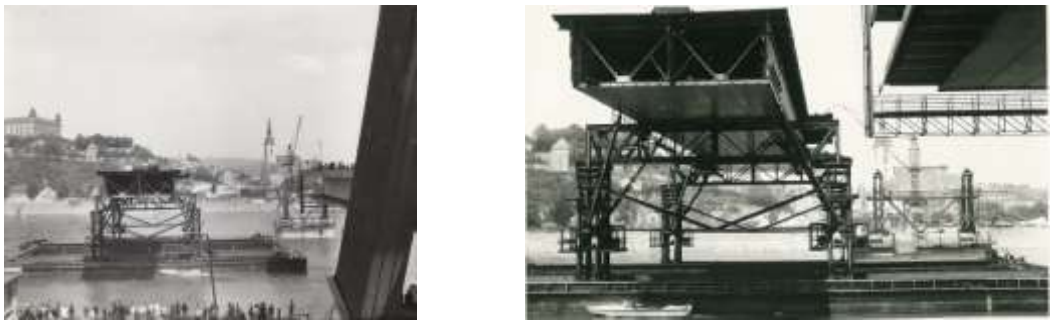


Fig. 88. Transport of bridge box-girder segment (title VÍTKOVICE) on pontoons. (Photo Igor Scholtz).



a) b)

Fig. 89. a) and b) Transport of temporary support B on pontoons. Accident with temporary support B. Winter 1970. (Photo Igor Scholtz).



a) b)

Fig. 90. PS. Preparation of the bridge box-girder segment No.4 (title VÍTKOVICE) for its attachment to the segment No.2: a) (Photo KKDK SvF STU); (Photo Igor Scholtz).



a) b)

Fig. 91. a) and b). Transport of temporary support B on pontoons. Accident with temporary support B. Winter 1970. (Photo Igor Scholtz).

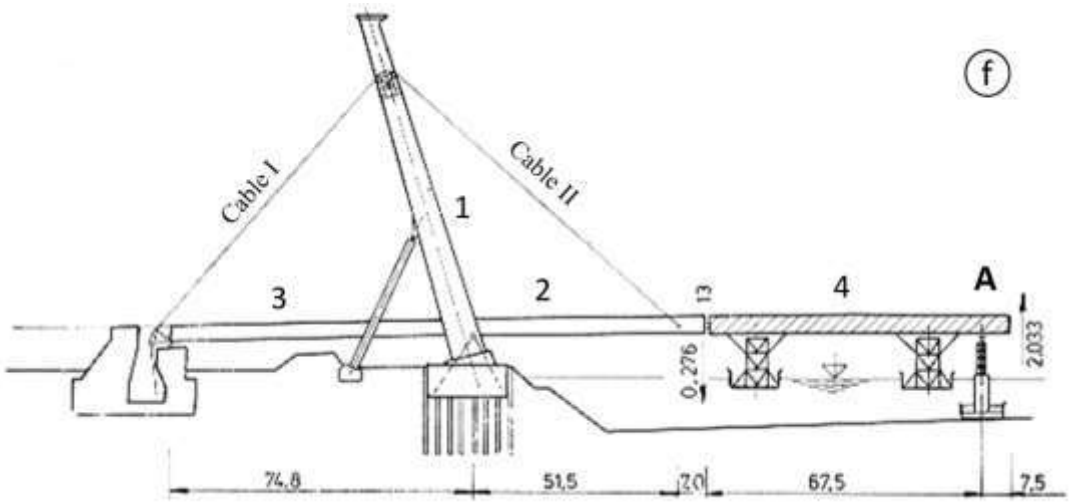


Fig. 92. PS. Construction of bridge. Bridge box-girder segment No.4 (title VÍTKOVICE) attached to the segment No.2 and supported by temporary support A. Phase f. [1].

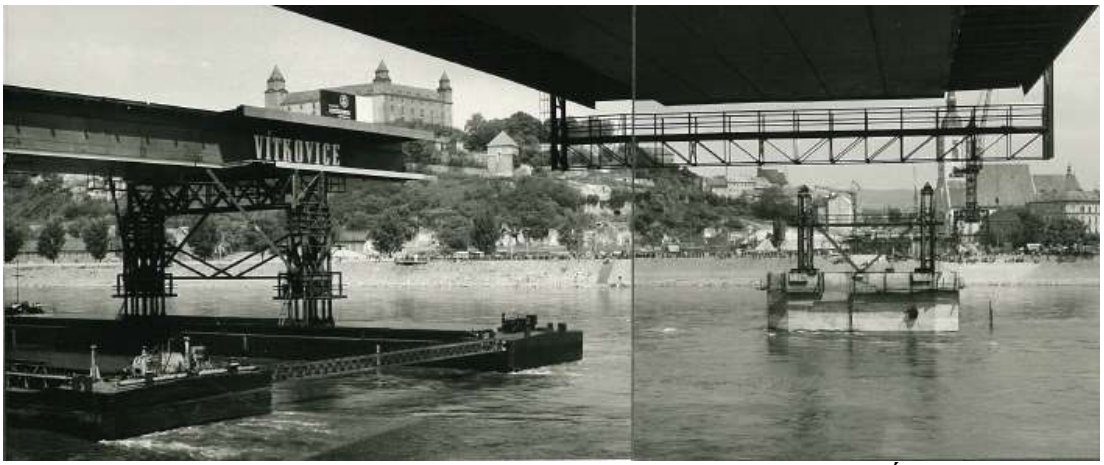


Fig. 93. PS. Preparation of the bridge box-girder segment No.4 (title VÍTKOVICE) for its attachment to the segment No.2: (Photo Igor Scholtz).



a) b)

Fig. 94. PS. a) and b). Preparation of the bridge box-girder segment No.4(title VÍTKOVICE) for its attachment to the segment No.2. Cable II. (Photo Igor Scholtz).



a) b)

Fig. 95. PS. Preparation of the bridge box-girder segment No.4 (title VÍTKOVICE) for its attachment to the segment No.2. Members of KKDK SvF STU: a) Mr. Karol Droxel, craftsman; b) Designers of SNP Bridge, from the left: Ing. Ferdinand Draškovič, Assoc. Prof. Ing. Pavel Dutko, CSc., Mr. Karol Droxel, Ing. Eugen Chladný, CSc. (Photo KKDK SvF STU).



a) b)

Fig. 96. PS. Preparation of the bridge box-girder segment No.4 (title VÍTKOVICE) for its attachment to the segment No.2. Members of KKDK SvF STU, designers of SNP Bridge: a) 2nd from the left Assoc. Prof. Ing. Vladimír Vořišek, CSc., Ing. Eugen Chladný, CSc., Prof. Dr. Arpád Tesár, DrSc.; b) 3rd from the left: Assoc. Prof. Ing. Vladimír Vořišek, CSc. (Photo KKDK SvF STU).



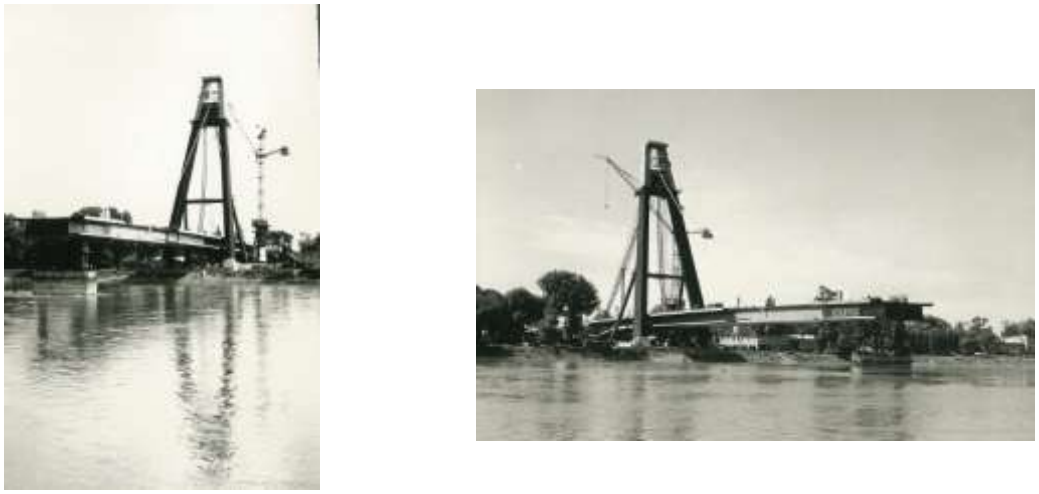
a) b)

Fig. 97. PS. Preparation of the bridge box-girder segment No.4 (title VÍTKOVICE) for its attachment to the segment No.2. Members of KKDK SvF STU, designers of SNP Bridge: a) Ing. Jozef Lapos, Ing. Jozef Vajda; b) Mr. Alexander Šalga, electrical technician.



a) b)

Fig. 98. PS. Preparation of the bridge box-girder segment No.4 (title VÍTKOVICE) for its attachment to the segment No.2. Member of KKDK SvF STU:
a) Prof. Dr. Ing. Arpád Tesár, DrSc.; b) Unknown people. (Photo KKDK SvF STU).



a) b)

Fig. 99. PS. a) and b) The bridge box-girder segment No.4 (title VÍTKOVICE) attached to the first segment No.2 and supported by temporary support A. Cable II.
(Photo Igor Scholtz).



a) b)

Fig. 100. PS. The bridge box-girder segment No.4 (title VÍTKOVICE) attached to the first segment No.2 and supported by temporary support A. Cable II.:
a) 6.6.1971. (Photo Juraj Bončo); b) (Photo KKDK SvF STU).

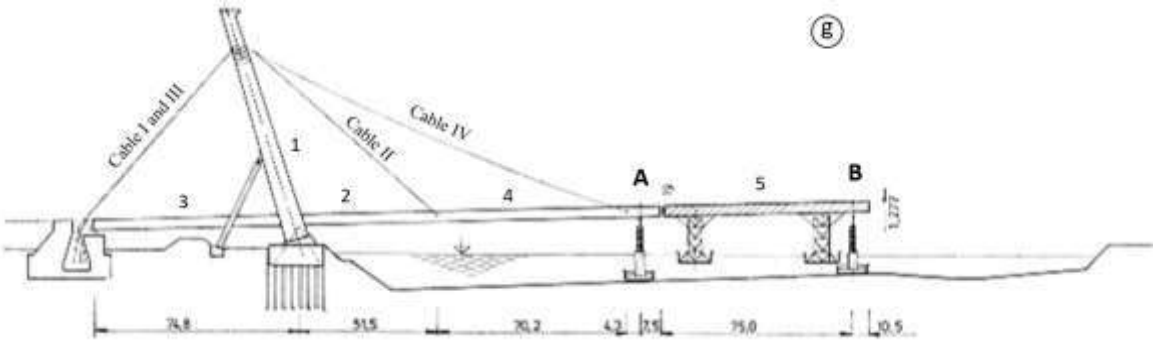


Fig. 101. Construction of bridge. The bridge box-girder segment No.5 (title HUTNÍ MONTÁŽE) attached to the segment No.4 (title VÍTKOVIC) and supported by temporary support B. Phase g. [1].



a) b)

Fig. 102. PS. The bridge box-girder segment No.4 (title VÍTKOVICE) attached to the segment No.2 and supported by temporary support A. Temporary support B. Cables II and IV: a) (Photo Igor Scholtz); b) 10.6.1971. (Phtoto Juraj Bončo).



Fig. 103. PS. The bridge box-girder segment No.5 (title HUTNÍ MONTÁŽE) prepared for launching. (Photo Igor Scholtz).



a)



b)

Fig. 104. a) Catamaran type vessel was designed by František Páleš (source: František Páleš [30]; b) PS. Connection of the bridge box-girder segment No.5 on vessel (title HUTNÍ MONTÁŽE) to the segment No. 4 (title VÍTKOVICE). (Photo Igor Scholtz).



a)



b)

Fig. 105. Connection of the bridge box-girder segment No.5 (title HUTNÍ MONTÁŽE) and segment No.4 (title VÍTKOVICE): a) (Photo KKDK SvF STU);
b) (Photo Igor Scholtz).



a)



b)

Fig. 106. PS. a) and b) End of cable II. (Photo Igor Scholtz).

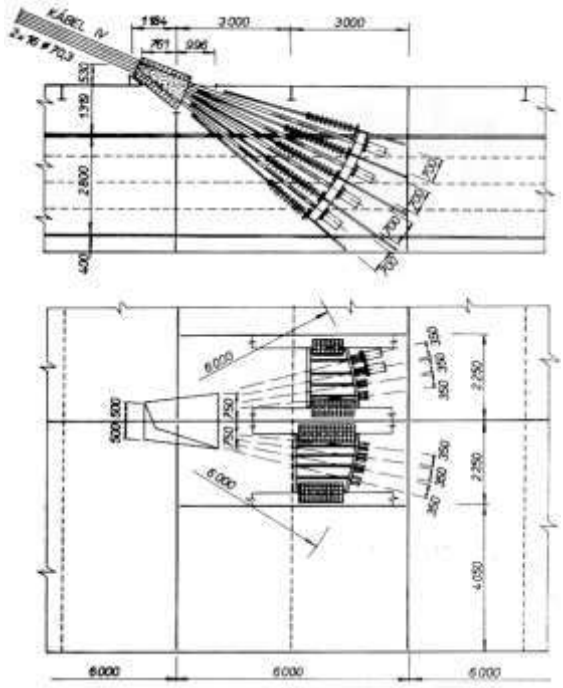


Fig. 107. Attachment of cable IV to the bridge box-girder segment No.5. (KKDK SvF STU).



a) b)

Fig. 108. PS. a) and b). Installation of cables II and IV. (Photo KKDK SvF STU).



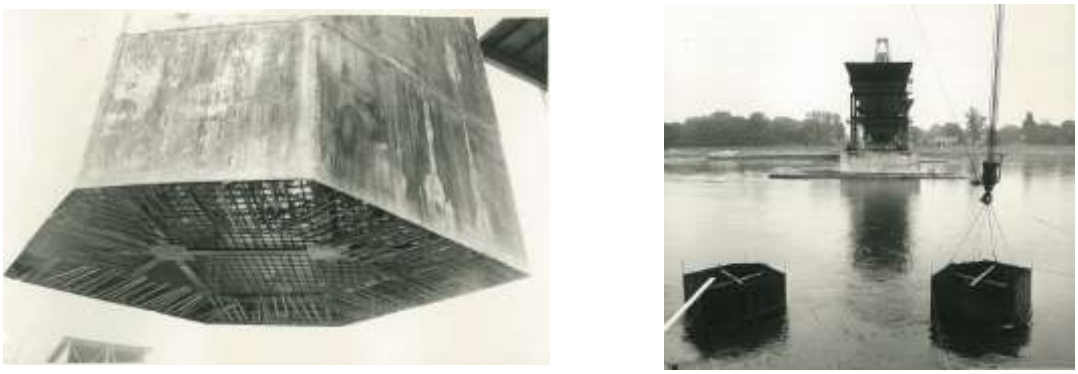
a) b)

Fig. 109. PS. a) Lead (plumbum) elements for installation inside cables; b) Installation of cable IV. (Photo Igor Scholtz).



a) b)

Fig. 110. PS. a) Installation of cable VI. (Photo Igor Scholtz); b) Bridge box-girder segment No.5 (title HUTNÍ MONTÁŽE) attached to the segment No.4 (title VÍTKOVICE) and supported by temporary support B. All three cables (II, IV and VI) are attached to the bridge box-girder. 29.8.1971. (Photo Juraj Bončo).



a) b)

Fig. 111. a) and b). Building of temporary support C. (Photo Igor Scholtz).



a) b)

Fig. 112. a) and b). Construction of temporary support C. (Photo Igor Scholtz).



a) b)

Fig. 113. PS. a) Asphalt surface of the road; b) Construction of structure of the restaurant on the top of pylon. Cables II, IV and VI in the left. (Photo Igor Scholtz).

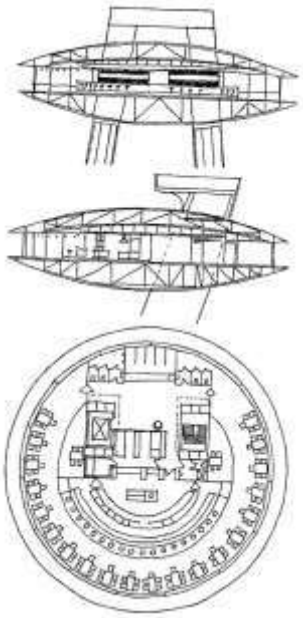


Fig. 114. Steel structure carrying restaurant. Architectural design: Prof. Ing. arch. Jozef Lacko, CSc. (*19.I.1907 – †15.VI.1989), Ing. arch. Ladislav Kušnír, PhD. (*16.VII.1929 – †25.XII.2010), Ing. arch. Ivan Slameň (*8.VI.1932 – †5.VIII.2006). Design of final steel structure: VŽKG n.p., branch Bratislava.
Original drawing: Ing. Ivar Plašil, KKDK SvF STU.



a) b)

Fig. 115. PS. Construction of structure of the restaurant on the top of pylon. a) Cables I, III and V on the left, cables II, IV and VI on the right; b) Cables II, IV and VI on the left. (Photo Igor Scholtz).

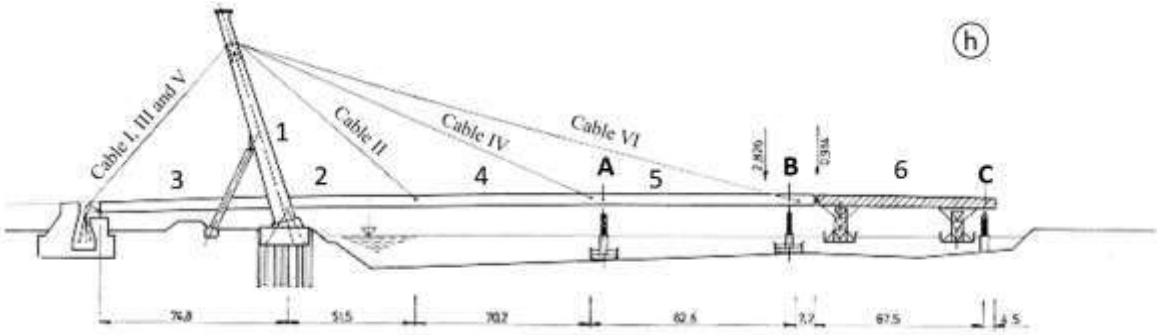
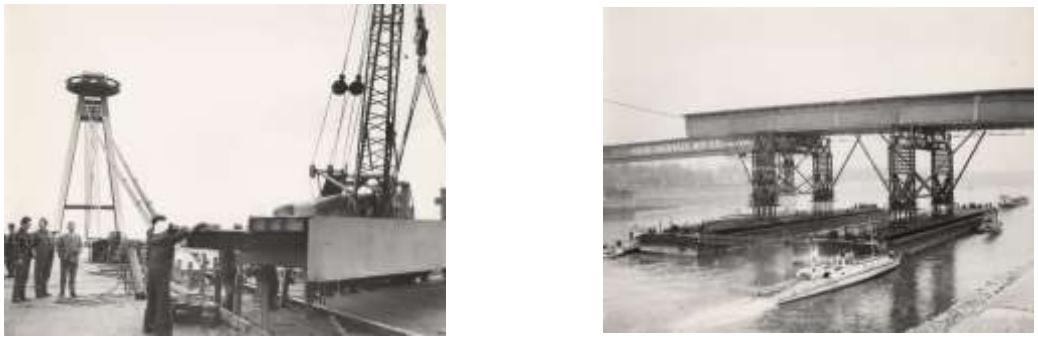


Fig. 116. Construction of bridge. Phase h. [1].



a) b)

Fig. 117. BS. a) Cable VI; b) Transport of the bridge box-girder segment No.6 on pontoons. a) (Photo KKDK SvF STU); b) (Photo Igor Scholtz).



a) b)

Fig. 118. BS. a) and b). Preparation of the bridge box-girder segment No.6 to the segment No.5 (title HUTNÍ MONTÁŽE). (Photo KKDK SvF STU).



a) b)

Fig. 119. BS. a) and b). Preparation of foundation for steel pendel piers wall (Fig. 122).



a) b)

Fig. 120. BS. Construction of flyover on the left bank of the Danube river; b) Bridge box-girder segment No.7 over the left bank of the Danube river. (Photo Igor Scholtz).





a) b)

Fig. 121. BS. a) and b) Foundation of steel pendulum piers wall (Fig. 122) on the left bank of the Danube river. (Photo Igor Scholtz).





a) b)

Fig. 122. BS. a) SNP Bridge after completion (Photo Ivan Baláž);
b) Steel pendulum piers wall. (Photo Igor Scholtz).





a) b)

Fig. 123. Bridge box-girder segment No.7 over the left bank of the Danube river:
a) (Photo KKDK SvF STU); b) (Photo Igor Scholtz).

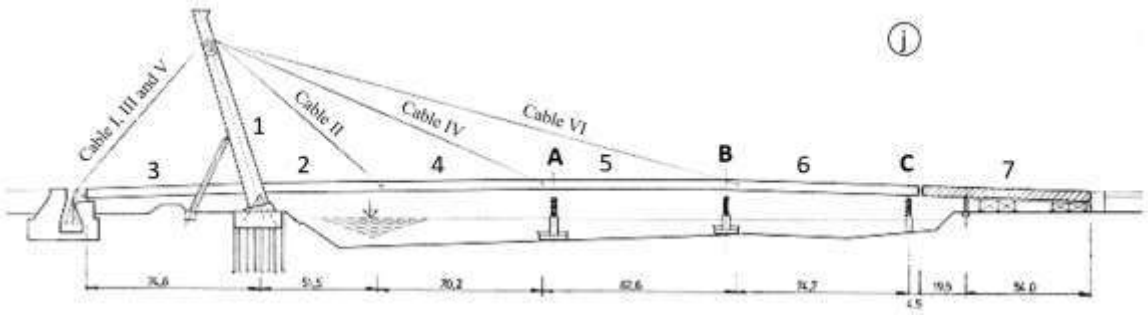


Fig. 124. Construction of bridge. Phase j. [1].



a) b)

Fig. 125. BS. a) and b). Bridge box-girder segment No.6 attached to the segment No. 5 (title HUTNÍ MONTÁŽE) and supported by temporary support C. (Photo KKDK SvF STU).



a) b)

Fig. 126. PS. a) and b). Painting of the pylon. (Photo KKDK SvF STU).



a) b)

Fig. 127. a) Cables II, IV and VI. a) (Photo Igor Scholtz); b) (Photo KKDK SvF STU).



a) b)

Fig. 128. The pylon of the SNP Bridge. (Photo Ing. Ladislav Bača).



Fig. 129. Bridge box-girder segment No.6 (left) and the segment No. 7 (right). (Photo Ing. Ladislav Bača).



a) b)

Fig. 130. a) Installation of tree – symbol of the bridge box-girder completion;
a) Ing. Eugen Chladný, CSc. on the left, member of KKDK SvF. (Photo KKDK SvF STU).



a) b)

Fig. 131. a) and b). Loading test of SNP Bridge. July – August 1972. [7]. Maximum deflection was 744 mm (92,7 % of calculated value).



a) b)

Fig. 132. a) Loading test of the flyover on the right bank of the Danube river. (Photo Dnes 24); b) SNP Bridge is complete except restaurant on the top of pylon.
(Photo KKDK SvF STU).



a) b)

Fig. 133. SNP Bridge: a) 26.8.1972; b) 3.9.1972. (Photo Juraj Bončo).



a) b)

Fig. 134. Staromestská street next. View towards city center: a) 29.8.1971; b) 20.6.2007. (Photo Juraj Bončo).



a) b)

Fig. 135. Staromestská street. View towards SNP Bridge: a) 29.8.1971; b) 24.6.2007. (Photo Juraj Bončo).



a) b)

Fig. 136. Staromestská street. View towards SNP Bridge: a) 13.5.1973; b) 29.4.2005. (Photo Juraj Bončo).

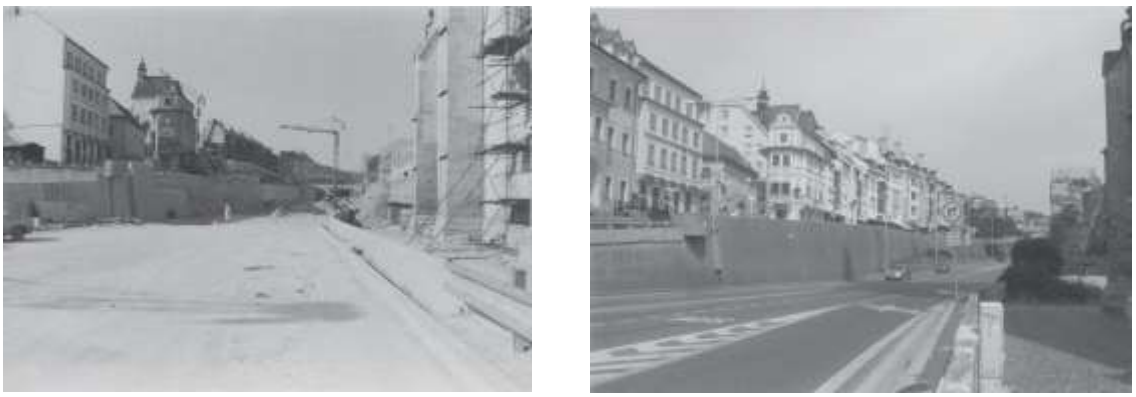


Fig. 137. a) Staromestská street beside St. Martin`s Cathedral (left), 6.4.1973;
b) View towards city center, 21.6.2007. (Photo Juraj Bončo).



Fig. 138. Three young engineers who also took part in design of SNP Bridge. From the left Jozef Lapos, Zoltán Agócs and Ivar Plašil. First two became later Professors at STU in Bratislava. Ivar Plašil emigrated in 1968 in Germany and worked some time at University of Stuttgart. (Rare photo from 1966, courtesy of Jozef Lapos).



Fig. 139. From the left: Ing. Zoltán Agócs, CSc. (*24.IV.1938 – † 14.II.2018), Prof. Dr. Arpád Tesár, DrSc. (*1.II.1919 – † 15.VI.1989), Ing. Jarmila Mesiariková, CSc., members of KKDK, in discussion with future engineers after defending their master diploma thesis about importance of so called "Sitzfleisch" (in German, it means perseverance in work of designer); 1973.



Fig. 140. Conference about SNP Bridge. No.1 Prof. Ing. Eugen Chladný, CSc. (*1. VIII. 1928 – †7.VII.2020) and No.2 Ing. Ján Bustin, CSc. (*17.II.1926 – †11.II.2017);
The branch of VŽKG in Bratislava worked out the final project of the steel construction and workshop drawings. For example the large amount of workshop drawings of the bridge box-girder in 15 thick volumes were prepared by the design staff of construction branch VŽKG in Bratislava. These drawings were signed by Ing. Jozef Šubr, Head of designers and Ing. Ján Bustín, CSc., doyen of Slovak designers and recognized by everyone as top expert.



Fig. 141. From the left: Ing. Ivan Baláž, CSc., (KKDK), Assoc. Prof. Ing. Jozef Zvara (*17.VII.1925 – † 29.II.2004), (KBKM), Prof. Dr. techn. Jozef Trokan (3.VI.1919 – † 28. V.2002), (KBKM), former Chancellor of STU and Minister for Civil Engineering of Czechoslovak government in 1968, Ing. Ferdinand Draškovič, CSc., (KKDK), in discussion about SNP Bridge at conference in Kočovce mansion. 1982.
The names Tesár, Zvara and Trokan are given in the document on Fig. 5.



Fig. 142. SNP Bridge in Bratislava over the Danube river. (Photo Ivan Baláž).

3. Authors

Authors are teachers of Slovak and foreign students at Faculty of Civil Engineering STU in Bratislava and they inform students about important historical structures and their designers. Students are interested also in other actual topics like those treated in [25-29].



Fig. 143. SNP Bridge in Bratislava over the Danube river. (Photo Ivan Baláž).

4. Conclusions

The paper presents collection of 300 photograph and drawings done in the period of SNP Bridge construction (1967-1972). Photographs and drawings are arranged chronologically in the way which enables to follow construction steps of SNP Bridge in detail.

Acknowledgement

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BRIEF OVERVIEW OF THE BULGARIAN EXPERIENCE IN STRENGTHENING AND RESTORATION OF HISTORICAL BUILDINGS IN BULGARIA

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Abstract: *The report attempts to give a brief overview of the Bulgarian experience on strengthening and restoration of old buildings, some of which have been declared cultural monuments in Bulgaria, and onto which modern construction technologies have been applied to increase their life-cycle including installing and tensioning of anchor cables, applying shotcrete, and other modern approaches.*

Key words*: reinforcement, restoration, brickwork, anchors, shotcrete, reconstruction, reinforced concrete, reinforced anti-seismic brick zone*

1. Strengthening the structure of the Central Military Club in Sofia[1]

1.1. Introduction

A contemporary approach to the structural strengthening of **a building** considered cultural heritage (**one** that had suffered local damage caused by modern urbanization) was applied by an experienced team of specialists [1]. By tensioning the brick walls by insetting steel anchors and filling-and-sealing cracks with chemical solutions, the imposing edifice of the Central Military Club in Sofia (a loadbearing brick-masonry construction at all sides) was entirely strengthened, whereby the existing cracks were filled-and-sealed. The purpose of such a development was to improve the performance of the supporting structure of the Military Club (Fig. 1a, b), significantly deteriorated after digging a tunnel for the subway line right underneath by using TBM heavy machinery [1]. As a result of this big-scale operation cracks were formed in the loadbearing supporting brick structure of the building - from 1 to 5-6 cm (Fig.2a, b).

Upon analysis made by the designers of the project concerning the strengthening of the building structure, one of the many conclusions but rather in favour of the **subway contractor** – the **Japanese company "TAISEI"**, was that **the seismic securing** of this building (which under any other circumstances would have been done properly) would be rather labor-intensive and may involve quite a serious **structural intervention**, which would turn extremely expensive –

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nearly comparable to the cost of a new building. For this very reason, for **buildings that are listed immovable cultural heritage**, some loop-holes from the regulatory requirements regarding seismic safety are sought after, as in this paper **they** are solely embodied and presented through the constructive measures undertaken with the only purpose of just filling-and-sealing the cracks in the masonry work, thence improving the overall behavior of the structure.

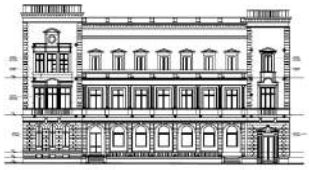
а)  б) 

Fig. 1a, 1b. Location of anchor plates on the façade - view from 1a "Tsar Osvoboditel" Blvd., General view of the building-1b).

Fig. 2a, 2b, 2c. General view of portico and main entrance to CMC-2a, Crack across the vault-2b, Crack in the facade of the building-2c

1.2. General overview

The foundations of the building are made of load-bearing stone masonry using lime mortar, and the vertical supporting structure is of brick-courses tied with lime-mortar mixture. From the discoveries made when studying the interfloor structures, it was seen that floor-slabs were made of wooden beams with dimensions of 30(32)/34cm. The aim was to create pressure across the plane of the interfloor horizontal structural members (the joists), so that when horizontal earthquake forces occur, that may potentially separate the walls from the horizontal structural members, the latter is counterbalanced, and the whole building works as one common element. The strengthening of the building was achieved by installing and tensioning 51 pieces of anchors of the MINOVA company, inserted/placed as follows: In the floor structure of the 2nd floor - x20, x3 of which Φ32mm and x17 - Φ25. In 3rd floor – 20 pieces, of which 6 are Φ32 and 14 are Φ25. In the roof slab – 11 anchors, of which 4 are Φ32 and 7 are Φ25 (Fig. 3).

1.3. The way the strengthening was performed

1.3.1. Inserting/placing anchors in single floor level

In the premises above the western tower and those facing Rakovski Str., on the timber beam structure planks are nailed, OSB wooden boards are layered, and finally finished with parquet, as **thickness** reaches 9cm. **Within such thickness**, anchors are placed on the upper

surface of the supporting beams: 2.1 and 2.2 in the floor structure of the second floor, and 3.1 and 3.2 in the floor structure of the third floor (Fig.3).

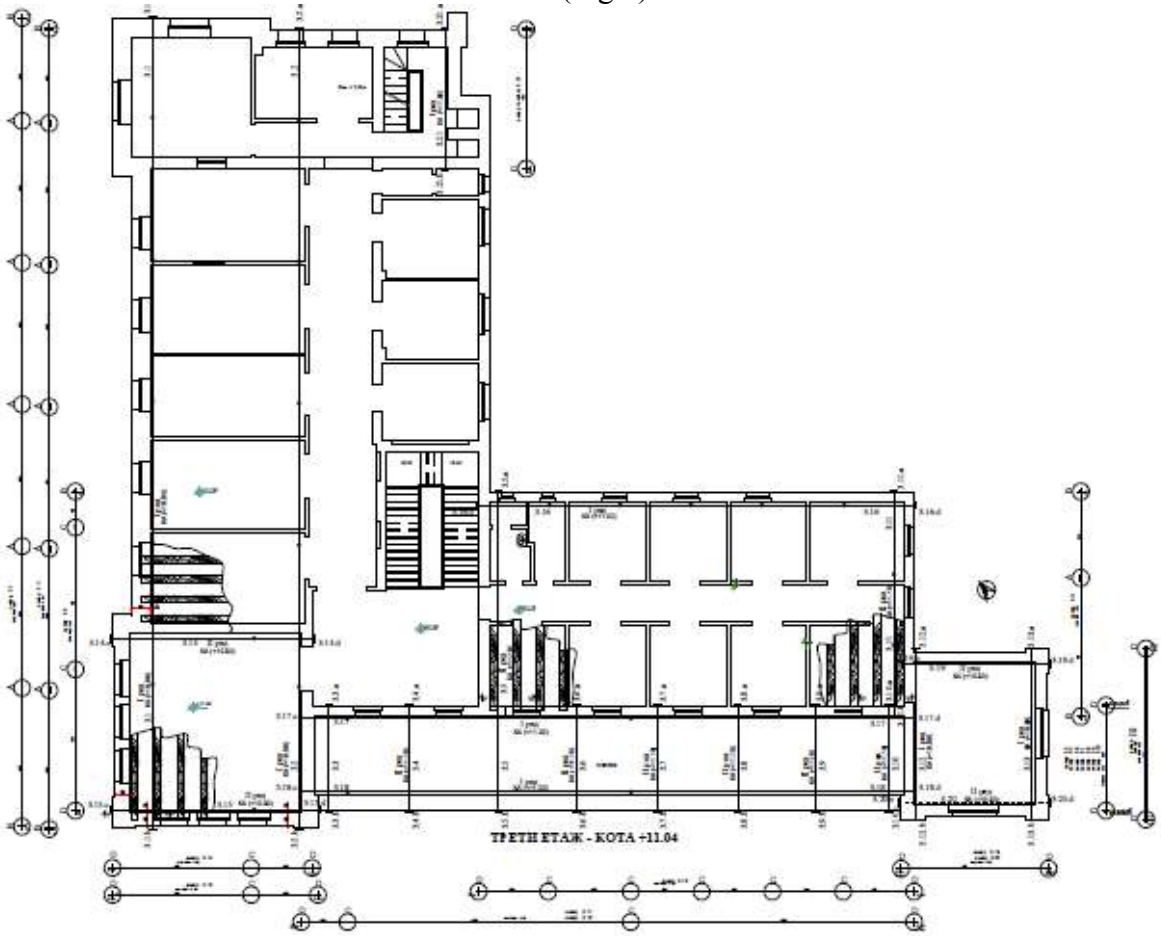
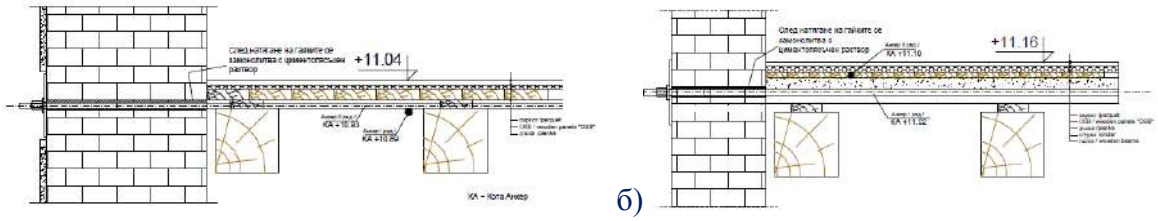


Fig. 3. Location/plotting of anchors across the floor plan of level 3

Anchors get installed first, indicated as follows: - I row, a diameter of **ϕ32mm** type SAS500. Onto them, in a perpendicular direction, 2.15 and 2.16 (on 2nd floor slab) get inset; anchors 3.14 and 3.15 are inset in 3rd floor, within the same thickness specified above. Anchors 3.16, 3.17, 3.18 are fixed at 3rd floor-area, anchors 2.3, 2.4, 2.5, 2.6, 2.7, 2.8, 2.9, and 2.10 of 2nd floor are installed on 1st row (of **ϕ25mm** diameter, type **SAS500**). Anchors 3.3, 3.4, 3.5, 3.6, 3.7, 3.8, 3.9 and 3.10 are inset across the third floor, and anchors 2.17, 2.18 and 2.19 of the second floor area are installed on 2nd row (of **ϕ25mm** diameter, type **SAS500)** (Fig.3a, 3b)).



а) б)

Fig. 3a, 3b. Detail for placement of I and II row anchors at elevation +11.04-3a); Detail for insetting anchors in I and II rows, at elevation +11,16-3b)

1.3.2. Anchoring the anchors

The anchors are fixed by using two types of anchor plates in the brick walls: first type - steel plates of 10 mm thickness - type Вст.3сп4 (old BG codes) of 350/350mm for anchors with a diameter φ32, and 300/300mm for anchors of φ25 diameter. Plates from the SAS500 system are

placed in front of them, measuring 120/120mm and 20mm thickness for φ32 anchors, and 70/70mm with 10mm thickness for anchors of φ25mm diameter. When splicing the anchors, a single muff and two counter nuts are used (Fig. 4a, b).

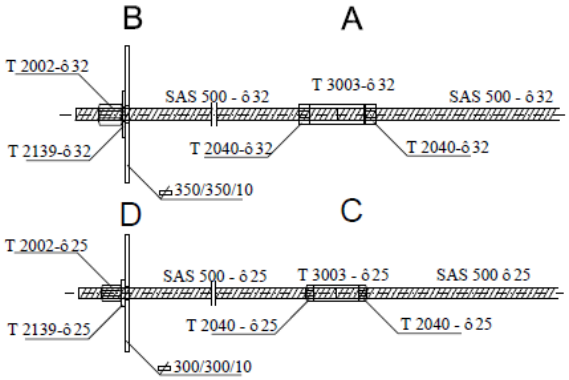


Fig. 4a, b. Detail for splicing different diameter anchors - φ 25mm and φ 32mm

1.3.3. Working about anchor holes and anchor heads

The cavity made upon drilling the brick-wall between the hole and the anchor gets injected after the anchor has been fixed/tightened with a cement-sand solution 1:1, from the inside of the building to the outside. All anchor heads, anchor plates and anchor nuts are pre-hot galvanized and after tensioning get coated with two coats of paint compatible with the galvanized finishing. In addition to the anchor plates of MINOVA, steel plates of 350/350/10mm are installed for anchor types SAS 500 φ32mm, and 300/300/10 for anchors of φ25mm dimension. The contact plane between these plates and the brick wall gets leveled/finished with a thin layer of cement-sand solution 1:1.

1.3.4. Stress process - sequence

Tensioning of anchors is carried out in a strict sequence, as tensioning force is not applied all at once, but in separate stages. In each stage, the anchors in/at all floor levels are successively tensioned by applying force of about 10kN. The tension executed is repeated in a series of four single stages, so that an end-tension force of 40kN for each anchor type is achieved. ***Of utmost importance is a mandatory control over the tensioning process.***

1.4. Seismic study

Pursuant to Art. 23 of Ordinance 2/23.07.2007 for the design of buildings and facilities in earthquake regions, it is necessary to check the effect of a horizontal computational seismic impact acting perpendicular to their plane and equal to 0,6.C.Kc.Q (local seismic standards of 1987). In the study, only a checkout was made as to whether the emerging earthquake force would cause the brick wall to fail holding the main goal to determine the dimensions of the anchoring plate.

1.5. Repairing/fixing the cracks in the walls

All cracks in the brick-walls were treated and filled with materials/compounds/solutions from the MINOVA company - BEVEDOL VF-BEVEDAN system, observing the system's recipes and the details indicated below (Fig. 5).

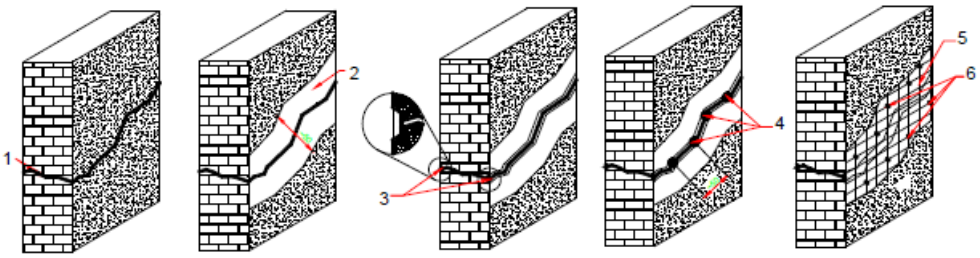


Fig. 5. Stages of an internal wall crack-treatment and recovery

Processing sequence is as follows:
- a crack of width > 10 mm; removal of plaster and lining in an area with a width of 50 cm; crack processing; packer; reinforcing mesh 10/10 cm; reinforcement - anchoring net - N8 (N10) of steel AIII (B420); injected solution; shotcrete or plastering.

2. Strengthening of the load-bearing brick-walls of "Crystal" building – Sofia; process of reinforcement and applying shotcrete[2]

2.1. Description of the built building and reconstruction

Reconstruction of buildings of load-bearing brickwork structure that have exhausted/weakened their loadbearing capacity appears a persisting issue. Cracks that develop over time threaten the safety of such buildings, thence of the inhabitants. Reinforcement of the load-bearing walls and applying concrete by spraying high-strength shotcrete mixture allows for additional loadbearing not only of new vertical loads, but also of other horizontal loads caused by an earthquake. In this case, the supporting structure of the building consists of three parts: northern, southern and central **[2]**.

2.1.1. Southern Part

The loadbearing supporting RC structure of the building in its southern part was erected a monolithic one - a column-and-beam structure over three-story rise: basement, ground floor, and first floor. The new part is separated from the old one by a construction joint of 2cm, as it rests on reinforced concrete columns that run alongside the edge of the structure joint, as well as on the eastern and southern facades, and on the inside. At basement level, the southern enclosing wall is laid entirely underground, as it appears a monolithic RC unit. RC follows up the height of the ground floor in the form of an under-window-sill frame. RC slab above ground floor, in 2nd and 3rd interior zones opposite the interior staircase, is observed in poor condition. There were many holes, whereat concrete had eroded, and for the lack of concrete finishing, reinforcement had got heavily corroded. During the process of pouring concrete over zone 3, the upper reinforcement above the beams was damaged, resulting in continuous cracks formed along the contour/edge of the slab with its surrounding beams. Such condition necessitated for concrete to be removed in these spots while preserving the upper and lower reinforcement inside the area, at a distance of at least 120cm from the inner edges of the beams. Such reinforcement is needed as a top reinforcement for the new fields/areas, and the bottom one for splicing between the old and new bars across the particular zone. The thickness of the new plate is to be 12cm as it may rest on 10cm slits onto the beams following the contour of the area. Considering designing a "Japanese" garden on the first floor, an opening of the roof area in the fourth zone located at the western end of the building is to be developed. The upper and lower reinforcement elements

must also be kept at a minimum of 100cm adjacent to the opening, so that they get inset and concreted in the reinforced concrete boards (as a rim around the new opening) of 25/40cm, completed above the slab. These boards and the inset reinforcement, including the one preserved in the area, may create a way to absorb the negative moments above the contour beams of the field, which had created the loads of the destroyed zone. Condition of vertical and other horizontal structural elements is good. There are no visible deformations or damages in them. The degree of importance of the building is not overrated following Ordinance N2of 23/07/2007 that covers Structural Design of Buildings and Facilities in Earthquake Areas, Art. 6 (2) – the seismic **safety assessment** is positive. The mass of the structure does not change either.

Nevertheless, on the southern facade within first and third zones, at the ground floor and on the first floor, the existing brick walls were torn down, as they were built anew of single solid bricks using **lime-cement mixture** brand M 2.5m, thus creating reinforced anti-seismic brick zone, encompassed with reinforced concrete beams and columns. **When** laying the brick-courses of the new masonry, both the horizontal and vertical joints in-between every single brick, and the bricks, as well as the contour RC elements had to be filled with that mixture. In every 2 rows, in the horizontal mortar joints, reinforcing bars of 2N10 of steel class A-III were inset. The reinforcing bars are to be anchored horizontally - 15 cm inside the columns encompassing the reinforced anti-seismic brick zone. The brick masonry should run transversely along the length of the wall, without a longitudinal vertical mortar-joint. Thus, the loadbearing capacity of the brick wall increases by 20%. The compressive strength of the single solid bricks should be 10MPa. The last horizontal row shall be wedged/anchored to the RC beam above it, by using oak (wooden) wedges placed on the outside and inside of the wall, at 100cm maximum. The brick wall fencing the inner courtyard on the east side of the new restrooms would be rebuilt. The western outer wall would be strengthened with a reinforcement mesh and shotcreted with class B30 shotcrete. (Fig. 6).



Fig. 6. Western outer wall

K28 and K29 columns at the height of the first floor get enhanced with angular steel-profiles of 100/100/10mm following the structural details, as all the edges get plastered beforehand with cement-lime mortar mixture for a tight fitting of the metal profile to the reinforced concrete columns.

2.1.2. Northern Part

The northern part is also built in a monolithic way, but as a skeletal-non-beam structure in two levels; Ground floor is a colonnade of 15 columns and floor-level one. The slab above the first floor is a beamless one, yet pierced by 6 monolithic RC domes. As the **clear structural height** in the floor is 240 cm, which is insufficient and in many places the slab is cracked by diagonal cracks running toward the middle row of columns, the same alongside the domes gets

demolished and replaced by a new smooth reinforced concrete slab, raised by 50 cm, and the clear structural height of the floor becomes 290 cm. In order to absorb seismic forces aided and supported by the columns only, it is necessary to strengthen the colonnade in the ground floor with a 15 cm thick reinforced concrete jacket(layer), on all three sides of the columns, as in the first floor the old columns are destroyed and in their place new ones with dimensions equal to those reinforced with a jacket are built. The roof plate between K7, K8 and K12, K13 columns is cut to shape a glass atrium.

2.1.3. Central (old) part

The construction in this part of the building is built in 6 floors of supporting brick walls with a thickness of 44 cm - 1 and 1/2 brick units. The bricks are a single-unit, solid, hand - pressed. The brick masonry withstands both the vertical and horizontal loads exerted. In many places, the walls were pierced with openings through which the air conditioning of the reconstructed building in the 1970s run. In this part, two staircases connecting the ground floor with the first floor were built. Concerning the future functions of the building, part of the door openings on the first floor and ground floor would be sealed following the design project, whereas in other places it will be necessary to cut-open new doors and openings for ventilation that require reinforcement of these openings following the details of the structural design. The basement of the building does not change. The foundations of the building remain unchanged as they have served their function efficiently (Fig. 7).

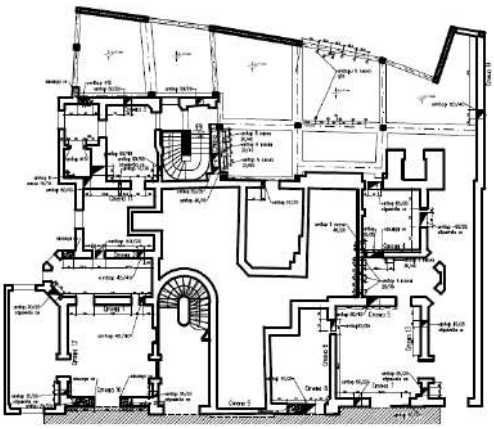


Fig. 7. Plan of the walls and openings of elevation -0,10

Inter-storey floor structures are made of metal beams, perpendicular to which are mounted metal bracing elements that act as reinforcement. The bracing is placed on the top of the beams, but between the beams - at their bottom part. Space between beams is filled with material imitating concrete with very low strength, consisting of cinders, sand and quicklime. Condition of the metal beams is good, but surface corrosion has developed on the lower belts when revealed, thence be cleaned with wire brushes. Beams surface is to be primed and then coated with two layers of paint to prevent future corrosion. Condition of parts of the vertical structural elements is good, but due to the drilling of air-conditioning and heating openings, their load-bearing capacity declined. Some of the walls got cracked by vertical and diagonal cracks within the height of 1st floor. Reinforcing all openings of the walls and the walls is a must. They are welded with each other, as the horizontal and vertical bars of the enhanced skeleton of the walls get welded onto them. In wall 2, a new door opening is cut out, as the same is reinforced with metal corner profiles specified in the wall reinforcement 2 (Fig. 8).

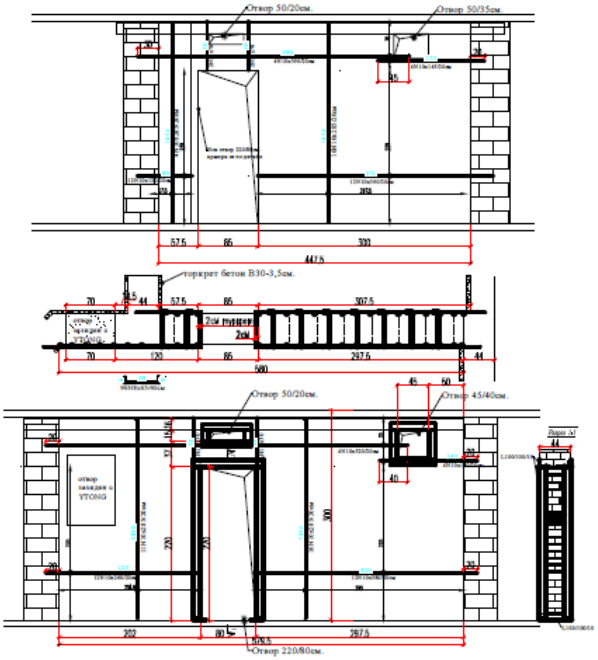


Fig. 8. Wall enhancement 2 - bilateral

All reinforced vertical walls are shotcreted unilaterally or bilaterally with shotcrete class B30, following the technical drawings.

2.2. Conclusions

This paper shows a rational and economical way of strengthening tall buildings of stone or brick masonry loadbearing walls. By strengthening the walls on the ground floor and first floor higher reliability has been achieved, both in terms of absorbing vertical and horizontal winds and seismic impacts. The reinforced structure of the building withstood the earthquake of 22/05/2012, without any problems.

3. The Museum of Contemporary Art in Sofia - modern view on reconstruction of buildings listed our cultural and historical heritage[3]

3.1. Introduction

In a cultural monument building built of load-bearing masonry walls and inter-storey structures made of wooden beams, new reinforced concrete structure was implanted addressing the contemporary structural requirements [3]. The stairwell built outside the contour of the building is attached to the new reinforced concrete structure by tensed anchors. The existing building was completed in the first quarter of 20th century as a three-storey rise, (Fig.9). It is part of our cultural and historical heritage. The foundation is stone masonry following strip foundations type. The construction of the building is done by encompassing load-bearing masonry of single-unit hand-pressed solid-brick course, as each brick is of the following dimension - 29/13/7cm. Along the longitudinal facades, at a distance of about 400cm, brick pilasters with dimensions of 59/43cm - (2 bricks)/(1 and 1/2 bricks) are completed. The function

of pilasters but from a constructive point of view is to perform the role of masonry columns with greater rigidity and to give stability to the building. On the transverse facades, such same pilasters are placed but at a greater distance. The pilasters are completed along the entire height of the building across the three floor levels. The external longitudinal facade brick-walls are load-bearing, and the transverse facade walls are self-supporting ones. The internal longitudinal brick-masonry walls of single brick thickness are load-bearing, and the transverse ones of the same thickness are self-supporting. Internal brick walls with thickness of 1/2 brick are partitioning.



Fig. 9. Museum façade

In the interior, two longitudinal brick-walls with thickness of a single-unit brick, and transverse walls with thickness of 1 and 1/2 brick are completed. Within the height of the building, the facade walls continue in the second and third floors following the same course; as the internal longitudinal and transverse walls have a partial change, illustrated when photographing the building. In the northern part of the building there is a stairwell with a flight of cantilevered stone steps and landings, as the surrounding walls are erected a load-bearing brickwork. The bearing, self-supporting and partition brick-walls are in very good condition, of no cracks or defects. This shows that the structure would normally tolerate the vertical loads, as well as the horizontal impacts that have occurred, so far. It lacks the necessary RC elements, columns, vertical and horizontal reinforced masonry zones that may possibly absorb the stress caused by 9-degree earthquake on the scale of the MSC, under Ordinance 2 for the design of buildings and facilities in earthquake areas.

3.2. Determining the compressive and tensile strength by subjecting the bricks with which the existing masonry structure of the building was built to a bending test

To study the strength qualities of the bricks, 8 bricks were removed from the building. They were tested in laboratory conditions to determine their tensile strength in bending and their compressive strength.

From the experimental studies conducted, it was found that the compressive strength is not particularly high: Rt = 3.3 MPa, and the tensile strength in bending is Rt,t = 1.23 MPa. Therefore, bricks cannot be included in the calculations of load-bearing brick walls absorbing seismic forces, because according to Ordinance 2 the minimum strength of bricks must be 10 MPa. Such matter prompts at the need of new reinforced concrete elements, columns, walls/bands and slabs to be implemented when reconstructing the building to absorb the seismic efforts.

3.3. Establishing a method of building the inter-storey floor structure and the type of material to be used

Upon uncovering the inter-storey floor structure, it was discovered that floor was made of a series of timber beams laid, of about 20/38cm, placed/embedded at about 59 cm distance, resting on one end on the facade walls and on the opposite end on the internal longitudinal loadbearing bricks-walls. From the sections exposed, it was found out that the condition of the wood is good, but the new reconstruction would create a greater load that such timber-beam floor is not able to bear. This fact necessitates the implementation of a reinforced concrete slab in a future reconstruction, which will also play the role of a horizontal diaphragm, distributing the efforts between the new vertical reinforced concrete diaphragms.

3.4. Assessment of the condition of the supporting structure

The above-stated suggests that the condition of the structure is very good, but it is not able to bear new, additional loads caused by the new function of the building, thus the strength of the masonry bricks does not meet the minimum requirements to withstand compression under Ordinance 2 covering the Design of Buildings and Facilities in Earthquake Areas. Thus, it necessitates (in future reconstruction) to switch to the implementing of new reinforced concrete slabs, which will also act as horizontal diaphragms, distributing the efforts between the new, vertical reinforced concrete diaphragms and columns.

3.5. In-building a new supporting structure

Inside the existing building, a new supporting reinforced concrete structure is built, which consists of 18 reinforced concrete columns, 6 reinforced concrete diaphragms, of which 2 in the longitudinal direction and 4 in the transverse direction of the building (Fig.10).

In the middle and alongside the contour/edges of the building, 5 continuous beams are built, stepping on the columns and diaphragms. A new reinforced concrete slab rests on top of the beams and diaphragms. All wooden beams and the timber floor structure are preserved as part of the future interior. The reinforcement of the slab rests on the façade/elevation beams, whereas in the middle of the building onto the center beam. The longitudinal wall is pierced as dowels are formed for the lower and upper reinforcement to run through them, upon considering a continuously reinforced slab to be laid. The existing timber-beam floor of 38cm is inlaid into the new reinforced concrete beams, so that a continuous reinforced concrete part of 50 cm remains underneath; total height reaches 114cm (Fig.11).

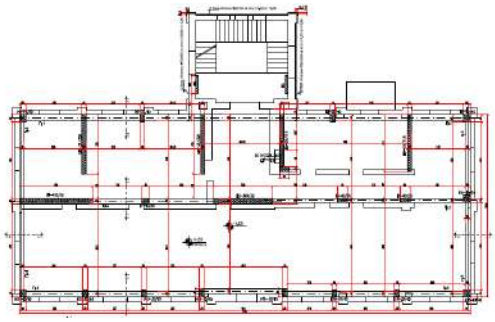


Fig. 10. Formwork plan of floor plate at elevation +4.65



Fig. 11. Details

Their width along the outer contours of the building is 30 cm, and in the middle of the building - 25 cm. Their transverse reinforcement-stirrups follow the height of the entire beam, encasing the horizontal wooden floor beams on either side. The same inter-storey floor structure is repeated above the slab on 2nd floor. Above floor 3, a grid of beams is made onto which the new reinforced concrete slab rests and following the slope of the roof, 2 reverse longitudinal beams (Gr 19 and Gr 20) in the middle third of the opening; 2 transverse beams (Gr 21 and Gr 22). An aluminum structure with overhead lighting is implemented over the four beams. The new columns and brackets are inlaid in the strip foundations, executed on the façade elevations at the inside of the building, and in the middle, on both sides of the existing stone strip foundations, as embedded to 10 cm width and a depth of 30 cm.

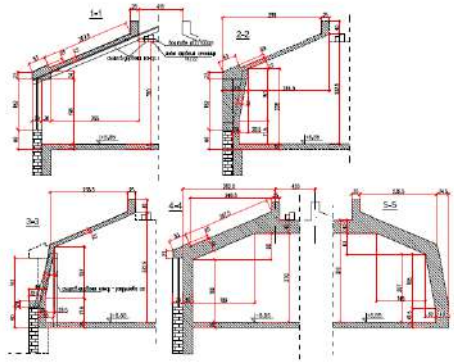


Fig.12. Cross-sections of roof structure

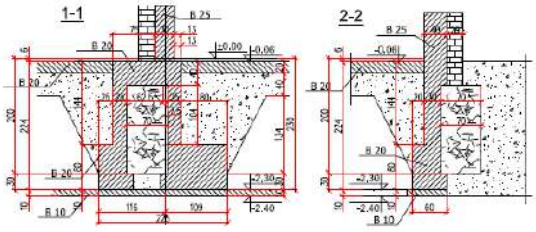


Fig. 13. Cuts through foundations

Widthwise, the middle two-sided foundations are symmetrically located relative to the axis of the reinforced brick-wall and columns. At certain distances, around the columns and at both ends of the longitudinal diaphragms in their upper part, the two-sided foundations are united/tied with a reinforced concrete transverse flat beam to enable distributing loads symmetrically across the common foundation. In the northern part of the building, beyond its edges, there is a three-arms staircase. Its structural attachment to the earthquake-proof building is completed by 3 pairs of metal anchors per floor of Φ25 size (in the floor of 1st floor, in the floor of 2nd floor and in the floor of 3rd floor), as the two pairs run parallel to the short side of the building, and the third pair follows the free, northern facade of the staircase.

Heads of the anchors are anchored externally in 4 vertically placed U-profiles, whereas inside the building in the facade longitudinal beams, by anchor plates. A panoramic elevator is to operate in the central part of the staircase, for an accessible environment. Due to the public functions of the building, it must also have a second emergency staircase. The same is fixed on the exterior of the building by an independent metal structure, as its floor and sides are entirely made of glass. Each of the floors (1, 2 and 3) is accessed by the staircase through doors located opposite the landings. In the project development, a classic way of embedding a modern reinforced concrete structure is realized, which may absorb any new stress-and-strain that may ensue due to its new function – a structure secured/strengthened by observing the Building Code(s) of the day.

4. The Cross-shaped Barracks in the town of Vidin[4]

The **Cross-shaped Barracks** was built at the beginning of the 19th century. It is a two-story building of **an equal-arms cross** in its plan (Fig. 14), thence **the name**. In the past, the building served as barracks for the Turkish troops across that area. Today it is the building that houses the Ethnographic Museum of Vidin hosting a large collection of artifacts from the region. The

construction of the building is combined: stone and brick masonry (Fig. 15). The foundations are of dry stone masonry, whereas the roof and floor structures are wooden.

Long-term use and lack of maintenance, as well as subsidence of the southern corner of the building due to systematic wetting of the soil underneath the foundation have led to the appearance of very serious cracks in the wall of the southern facade, as well as in a large part of the interior walls. Moisture in the brickwork was found at a level of about 1 m above the ground level, and this has led to the destruction of the wall plaster over a very large area. Preservation of the authentic appearance of the building necessitated the search for a solution that would not change the original facades. The reinforcement of the existing masonry was solved by applying sprayed concrete (shotcrete) (Fig. 16 and Fig.17).



Fig.14. Cross-like barracks plan

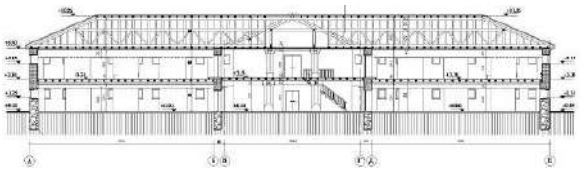


Fig. 15. Cross-section of barracks

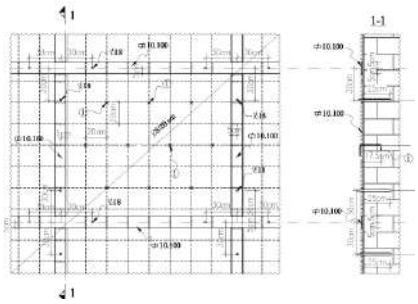


Fig.16. Shotcrete execution detail



Fig.17. Shotcrete execution detail

An additional air channel was filled in alongside the contour of the building (below the ground level) from reinforced concrete for the purpose of ventilation and insulation of the foundations, as well as additional strengthening of the stone masonry foundations (Fig. 18). When construction began and the furniture was removed, new additional serious cracks were found in parts of the brick walls. In order to preserve the façade, an additional new reinforced concrete wall with new reinforced concrete base was designed. The new wall is built on the inside and subsequently formed out a unique interior space (Fig. 19). This project was entirely developed with the aim to preserve the architectural authenticity, as well as to restore its structure in a way that it may withstand vertical impacts. No additional seismic safety measures have been designed because they are not compatible with the idea of preserving the original structure. The new reinforced concrete wall does not oppose the concept of preservation because it aims only to restore the original integrity of the structure.



Fig.18. Channel filled alongside edge



Fig.19. The new wall

The Cross-Shaped Barracks building is a cultural monument and one of the 100 National tourist attractions in Bulgaria[4]. It is situated in Vidin - a city in Bulgaria on the Danube River. The building is famous for its equal-arms cross plan (Fig. 14).

The building was constructed in 1801 by local ruler Osman Pazvantoglu for the deployment of the Ottoman troops (Fig. 15). After the liberation of Bulgaria from the Ottoman Empire, the building was used as a court and barracks of Bulgarian troops.

5. Static analysis and retrofitting of the old timber roof structures for the storage heritage building in Kustendil[5,6]

The building is a model of industrial architecture in the thirties of the twentieth century, which still dominates with its volume in the modern urban structure in the town of Kyustendil. It was designed as a tobacco warehouse by the Austrian architect Rudolf Fischer, who lived in Kyustendil in the period 1909-1934. The building was built in the period 1924-1925. The facades of the building are flat and treated equally on all four sides(Fig.20). The windows are in pairs alternating with flesh. The height of the windows in the different floors is different with a certain gradation. The attic space is illuminated with dormers along the long axis and with windows along the narrow fronts (Fig. 20.1).

a) 

b) 

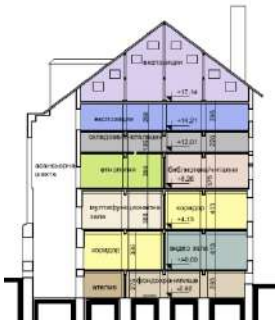
Fig.20.1a,b East elevation-a), Nord elavation-b)

c) 

d) 

Fig.20.1c,d South elevation-c) West elavation-d)

The height of the building consists of a semi-dug basement, three main floors, a mezzanine and attics. The presence of a high brick superstructure favors the use of the attic space. The building is designed as one main volume, with a rectangular base and five main floors, basement and attic floor: Basement, elevation -2.92, with: H_{light}= 2.75 m; First floor, elevation -0.00, with: H_{light} =3.96 m; Second floor, elevation: +4.13, with: H_{liht} =3.88 m; Third floor, elevation +8.26, with: H_{light} =3,50 m; Fourth floor, elevation +12.01, with H_{light} = 1.95 m(installation); Fifth floor, elevation +14.21, with: H_{light} = 2.68 m; Sixth and seventh attic floor: elevation: +17.14 and elevation: +19.73 are from wooden structure(Fig.20.2.).





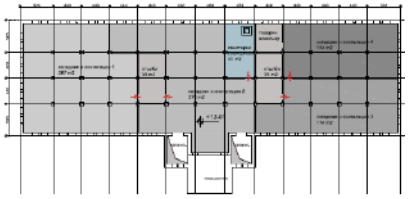


Fig.20.2. Cross section **20.3.** Horiz. section: over -1 floor **Fig.20.4** Horiz. section: over 4- floor

The building in plan has rectangular dimensions with a width of 18.1 m and a length of 59.79 m and a built-up area of approximately 1082m^2. In the transverse direction, the columns of the building are located in three wheel bases with dimensions: 4.60 + 4.00 + 4.00 + 4.60 = 17.200m, which is the light spans of the building. In the longitudinal direction, in the ground floor the columns are arranged as follows: 5.00 + 4.40 + 4.40
+4.40 +4.10(Fig. 20.2; Fig.20.3; Fig.20.4 and Fig.20.5).

a) 

b) 

Fig.20.5a,b General view of the building and the south-east façade

2. Description of the Constructive System of Building

The construction of the building is massive with external load-bearing brick walls 80 cm thick in the basement, 60cm on the first and second floor, 45cm on the third and fourth floor, 40cm on the fifth, sixth and seventh floors. The columns are from reinforce concrete with dimensions 78/78cm in the basement, and from brick with dimensions 64/64 cm on the first and second floor, 50/50 cm on the third and fourth floor (Fig. 2.1a). In the transverse direction, the columns of the building are located in three wheelbases with dimensions: 4.60 + 4.00 + 4.00 + 4.60 = 17,200m, which is the light spans of the building. In the longitudinal direction, in the first section the columns are arranged as follows: 5.00 + 4.40 + 4.40 + 4.40 + 4.10 m. The mezzanine structures at elevation: +(-) 0.00 (slab above the basement) and at elevation: +4.13 m (slab above

the first floor) are a beam system with monolithic reinforced concrete slabs and beams, which lie on brick (respectively) RC columns (Fig.21.1).

a)  b) 

Fig.21.1a,b Distribution of the columns in the basement of the building-a), View on the floor and column in the first level of building-b)

At elevation 8.26 m (slab above the second floor), as well as at elevations +12.01 (slab above the third floor) and + 14.21 m (slab above the fourth floor) the structures are made of wooden beams with a caratavan ceiling. The wooden beams are oriented in the longitudinal direction of the building and are supported in the transverse axes of the building on steel double T-beams, which in turn lie on the columns of the building (Fig. 22.1a,b,c). The wooden beams have different sizes varying in the range: 16/18, 18/20, 17/17, 20/20, 17/18, 18/18. The 7 timber beams are located in the end and 5 in the intermediate axes of the building. The steel beams on the second floor are №30 and on the third and fourth are №22 and №24. The floors are made of wooden sheets, placed on a joist structure of the ridge (Fig.22.1).

a)  **b)**  **c)** 

Fig.22.1a,b,c Supporting of the wooden beams on steel double T- beams; Supporting of the steel beams on the columns of the building.

3. Description of the Constructive System of Timber Roof of Building

The roof structure of the museum in Kyustendil is a gable roof with alpine type and height to the ridge: H = 10.10 m from the level of the ceiling slab. The base of the isosceles configuration of the gable roof is light in size between the enclosing walls: 17,220 m. The supporting structure of the roof is made entirely of wood (Fig. 23.1). The roof structure is made entirely of wood - made of shaped wood. The roof is gabled with a steep slope covered with marseille tiles on a wooden cladding.

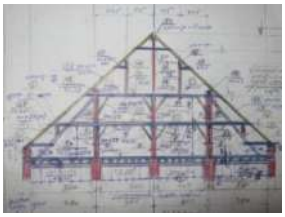
a)  b) 

Fig. 23.1a, b The drawing of the the roof truss structure- and View on the timber roof

The main load carrying elements are: binder, columns-with capitals(from diagonal supporting struts), or slopes to each columns, purlins in the longitudinal direction of the building(Fig.24.1) of the three-storey roof structure and also the presence of kings on the third level of the structure (Fig.24.1).

a)  b)  c) 

Fig.24.1a,b,c Binder columns(kings) anchored on a binder beam-a);capitals in the longitudinal direction, slopes to each column-b) Pliers and kings at each level of the three-storey structure of the roof-c)

4. Computational 3D model of the truss construction of the building

4.1 Load model

Loads are adopted pursuant to Ordinance № 3/2004 years. "Basics of designing the structure of the buildings and the impacts on them" for the region of Kyustendil as follows: snow region III with value 130 daN/m^2 and wind to area II with value 32 daN/m^2. Computing values of these values are achieved by multiplying the normative values of the two impacts with load factor:=1.50. Studied are all occurring in wooden frames species components. Static analysis of the various structural elements of the roof done according to the methods of structural mechanics and verified with FEM program "SAP2000". Static schemes of frames and roof are modeled as plane joint-rod systems. All fixtures in the wooden structure of the roof are filled with bolt-screws. Dimensions of the different components of the roof frame of strength, stability and deformability are carried out according to EC5, 2007. The quality of the wood, when testing the structural elements of the roof frame is adopted as a second category (SI-II). On the Fig.4.1is demonstrated the Computational 3D model of the truss construction of the museums building. The drawings of the main truss construction is on the Fig. 4.2 Cross sections of the constructive element of the timber truss system is the Tab. 1. As additional information is important to know that: length of the truss is: 17200mm, and high: 10010mm. The length of the roof is: 59,79m, width: 18,10m and high: 10,10m.



Fig. 25.1 Computational 3D model of the truss construction of the roof

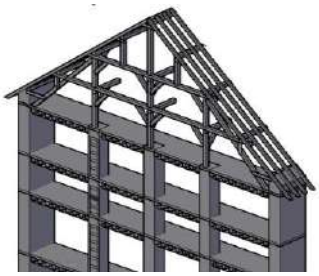


Fig. 25.2 The drawings of the truss construction of the roof

5. Calculation of the Construction according to the Theory of the Second Order

In order to be able to comment on the reserves of the truss supporting structure, we performed a full calculation links with SAP2000 software, according to theory I. and II. order. The combinations of loads and values are given in next text as follows:

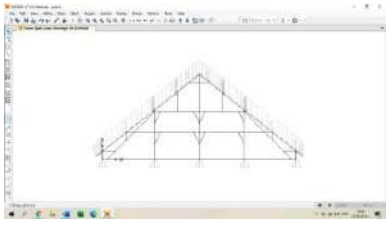
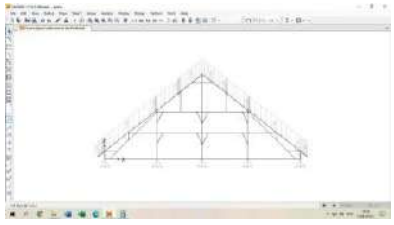
5.1 Load combinations (according: EC5, 2007)

1.35 DL + 1.5 SL; 1.35DL + 1.5SL+ 0.75WL; 1.35DL+ 1.5WL (pressure) + 0.75SL; 1.0DL + 1.5WL (suction); 1.35DL + 0.75WL.

5.2 Load values

a) Covering:0.70 (kN/m^2); b) Snow: 1.30(kN/m^2); c) Wind (pressure): 0,32/-0.32(kN/m^2); d)-Wind (suction): -1.00/-1.00(kN/m^2).

Due to the age of the construction were created extraordinary load values, which in this case can count on a 100-year return Scheme of external load: dead load, snow and wind load and are given in Fig.26.1.

a)  b) 

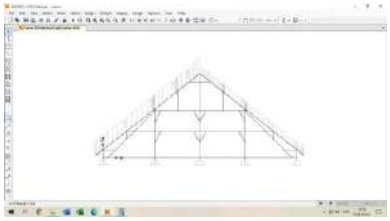
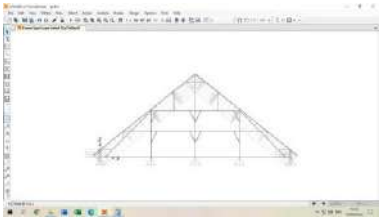
c)  d) 

Fig.26.a,b,c,d Scheme of external load are given

6. Some remark about articles in [5,6]

The calculation verified that if there is no further significant damage to the timber construction by biotic influences, there is no danger of the collapse of the truss of the building in Kustendil. If we think about the constructional feeling of the arch. R.Fischer, we must state that this was considerably advanced. Although they had no modern computing resources, they did comprehensively combine various requirements for the functionality of the work, including some protection against biotic influences. From today's point of view, the building withstood all the effects of a load with a 100-year return and still despite some damage to the truss does not threaten its immediate collapse.

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PROFESSOR EDMUND BALGAČ'S CONTRIBUTION TO THE THEORETICAL SOLUTION AND APPLICATION OF SUSPENDED PRESTRESED STRUCTURES

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***Abstract**: The need to cover large spans forced designers to use saddle-shaped geometric forms as the ideal form for the application of cable suspended systems. The expansion of suspended systems arose in the 50s of the 20th century, with the improvement of simpler numerical methods for solving complex systems of differential equations of the fourth degree. Maciej Nowitski (USA) and David Jawert (Denmark) created their analytical solutions that have brought these systems to a rational application. With his rich theoretical knowledge and practical skills, Edmund Balgač (Yugoslavia) improved existing analytical models and applied them to the construction of several halls. With the uniqueness of his solutions, by the application of suspended structures, prof. Balgač managed to establish a symbiosis between aesthetics and the constructive requirements of covering large areas. Today, we can testify to the fact that a large number of buildings built in the above-mentioned structural system continue to impress with their form and function decades after their construction. For this reason, as part of this research, an analysis was made of the Great Hall of the Textile Fair in Leskovac, the Fair and Sports Hall in Subotica and the Sports and Cultural Center "Pinki" in Zemun. When analyzing the mentioned buildings, the emphasis will be on the architectural heritage of buildings built in the constructive system of pre-stressed suspended cables. The aim of this paper is to review the creativity of prof. Balgač, as well as emphasizing the importance of the existing suspended structures in the form of cultural and architectural heritage of an era.*

***Key words**: Architectural heritage, spatial structures, cable suspended systems, Edmund Balgač*

1. Introduction

Suspended structures represent a characteristic category in civile engineering, characterized by an elegant form and the ability to cover large areas. In the field of structural engineering, suspended structures represent the pinnacle of the combination of theoretical excellence and practical application, often defining modern urban landscapes

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with their graceful appearance and extraordinary engineering challenges. The engineering challenges posed by suspended structures often require innovative solutions. Design consideration, material selection and construction methodology are carefully examined in order to achieve a combination of aesthetics and functionality of the object. When it comes to suspended cable structures, the central part is an understanding of the principles of prestressed concrete combined with the unique challenges posed by suspended cable systems. A comprehensive understanding of the benefits and challenges associated with suspended cable structures is critical to design and construction decisions. Increased structural efficiency, large spans and material savings stand as advantages, while in contrast, challenges such as cable maintenance, dynamic behavior and construction costs arise.

Today's impressive presence of suspended structures in the urban landscape testifies to their long-lasting influence. One of the key protagonists in this field, whose name is inextricably linked to the theoretical understanding and outstanding application of suspended chain structures, is Professor Edmund Balgač. This paper aims to research and analyze the contribution of Professor Edmund Balgač to the theoretical solution and application of suspended cable structures, looking at how his work shaped the modern understanding and practical application of these architectural and engineering masterpieces. For this reason, this paper will focus on the analysis of the theoretical contribution, as well as the analysis of three key objects - the Great Hall of the Textile Fair in Leskovac, the Fair and Sports Hall in Subotica and the Sports and Cultural Center "Pinki" in Zemun. The theoretical contribution of Prof. Balgač is based on the work of Jawerth (Sweden) and Nowicki (USA), whose work prof. Balgač continued and then led to the improvement of the existing methods. Then the emphasis of the analysis will be on the construction heritage of the mentioned buildings, built in the constructive system of pre-stressed roof cables. Through this analysis, the goal of the paper is not only to look at the creativity of Professor Balgač, but also to emphasize the importance of the existing hanging structures as the cultural and constructional heritage of an entire era. This work represents an attempt to deeply understand and valorize the contribution of Professor Balgač in the architectural heritage, shed light on the various applications of suspended structures, and emphasize their undeniable role in shaping cultural identity and architectural landscape.

2. Biography

Edmund Balgač was born on May 24, 1913 in Belgrade, Kingdom of Serbia. During his rich career, he was a professor at the Higher Technical Construction School in Subotica and an engineer at GP "Rad" in Belgrade. He left behind a great contribution, both with his research and teaching work, and with a large number of realized objects that still impress with their function and form.

As a professor, he left a significant opus of research work in the field of suspended cables structures. With his theoretical contribution, he earned the epithet of one of the pioneers and originators of suspended systems, both in the area of the former Yugoslavia, as well as in Europe and the world. One of the most significant contributions to science is reflected in the introduction of numericals into the analytical method of calculating suspended structures, which continued and improved the work of Prof. Nowicki. With his research work in the field of suspended systems, Prof. Balgač laid the foundations for future expansion in the construction of these structures. As a researcher, he made a great contribution to a number of scientific works, as well as the textbook "Engineering structures: reinforced concrete structures" completed at the Higher Technical Construction School in Subotica in 1966. It is important to mention the translation work of Prof. Balgač, who enabled better education of future generations at construction schools and faculties.

Under the translation work, it is important to mention the books: "Spatial roof constructions, their details, their implementation" by Rile Herman and "Prestressed concrete in practice" by Leonhardt Fritz.

Apart from the theoretical work, the practical work of prof. Balgač as one of the most outstanding civil engineers of that era. As an engineer at GP "Rad" from Belgrade, Balgač left many buildings that even today have the role of a landmark in the urban silhouette of the public space on which they were built. Among the more significant projects that have been carried out, we can mention: the Great Hall of the Textile Fair in Leskovac, the Fair and Sports Hall in Subotica and the Sports and Cultural Center "Pinki" in Zemun.

Prof. Edmund Balgač died at the age of 77 on December 9, 1990 in Belgrade, leaving behind an indelible contribution, with theoretical work and constructed objects that still remain a reminder of his greatness and importance.

3. Theoretical contribution of prof. Balgač

3.1.A historical overview of the works that contributed to the elaboration of the analytical solution

When it comes to the theoretical work of Prof. Balgač in the field of suspended cable structures, it is necessary to mention names like David Jawerth and Maciej Nowicki, who laid the foundations for further improvement of the calculation of suspended structures. Prof. Balgač continued his work on the improvement of analytical methods for the calculation of suspended structures set by Jawerth and Nowicki. The first major step forward in the creation of suspended structures was the optimization of the analytical calculation in which the stabilization of chainrings by weight was replaced by prestressing, which introduced a revolutionary concept in this field [5].

Swedish architect David Jawerth has developed an innovative system to overcome unpleasant occurrences of swaying in building structures. The cable truss system, often referred to as the Jawerth system, suppresses unforeseen deformations, especially in the horizontal direction, providing structural rigidity. The system consists of grids or beams that make up two cables and an anchor structure that ensures the load-bearing capacity of the cable structure. Suspended structures in the vertical axis are adapted to standardized modules, which achieves standardization in spans from 25 to 60 and more meters. According to Jawerth's recommendation, the ratio of the added load (v) and the total gravity load (q) should be taken as $v/q<0.6$ for the calculation. The theoretical solution was applied in the design of the Ice skating Arena in Johanneshov near Stockholm (Fig. 1), for which the project was also designed by Jawerth. The Ice skating Arena in Johanneshov, Sweden, is the first facility built with a cable truss system. It was built in 1956 with a capacity of 16,000 spectators. The construction of the arena includes cable grids of the span 73 m to 83 m, with sloping infill rods. The supporting cable has a diameter of Φ58 mm, the stabilizing cable is Φ48 mm, the boom of the supporting cable is 5 m, and the boom of the stabilizing cable is 3.6 m. Cable grids are placed at a distance of 5.28 m, and bundles of Φ26 mm steel bars are used to receive horizontal forces. Ground anchoring relies on prefabricated reinforced concrete blocks. The support structure is made of steel profiles, and part of the hall is buried with a reinforced concrete structure. The roof structure, adapted to various climatic conditions, includes cassettes of cold-formed steel sheets, corrugated sheet, impregnated wood fiber boards and layers of roofing paper [5][9].

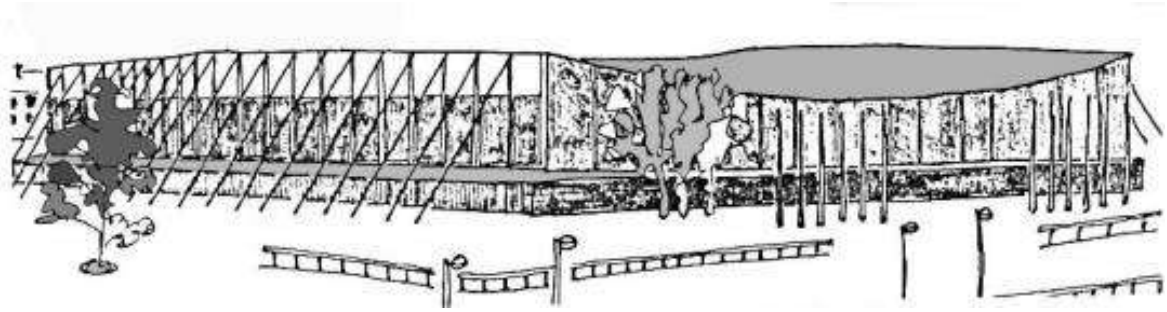


Fig. 1 Ice skating Arena in Johaneshov, Sweden, 1956 (David Jawerth)

Maciej Nowicki (1910-1950.), architect and professor, was a faculty member of the architecture department at the newly formed North Carolina State College School of Design from 1948 to 1950. As a professor, designer and urbanist of international repute, his legacy is mainly reflected in the inspirational influence he exerted in the School of Design, the lost potential to change the course of contemporary architectural and planning thinking and practice, and in the radical design of the only completed project in North Carolina, the famous Dorton Arena, completed after his death. Nowicki produced a series of studies for the development of various facilities at the Raleigh State Fairgrounds, including a bike lane, a running track, and an elevated median (an unrealized project). The city was able to secure 1.5 million dollars for a new livestock auction pavilion, and Nowicki was entrusted with the project. This became the only completed building in North Carolina designed by Nowicki. Encouraged by Dorton and in collaboration with engineer Fred Severud, Nowicki drew up a plan for the Raleigh arena (Dorton Arena) (Fig. 2) whose shape is defined by two inclined parabolic arches of reinforced concrete that meet near the ground, and the arches are connected below ground level. at both ends and prestressed above by numerous cables running from arch to arch. The cable structure would additionally serve as a support for the roof covering. Dorton Arena has gained wide and lasting admiration in the state and beyond, becoming one of the few North Carolina facilities to be internationally recognized. The extraordinary hyperbolic-paraboloid structure was echoed in a series of later "saddle dome" designs produced by architects around the world in the 1950s and 1960s. For North Carolina in the mid-20th century, it was a bold symbol of modernity and progress, and became a celebrated icon of modern architectural design from the day it was built. Professor Nowicki also made a significant theoretical contribution to the development of cable constructions, unfortunately his untimely death led to the analytical method, the originator of which, remains unfinished. After Nowicki's death, Prof. Balgač continued work on improving the analytical method and managed to improve and complete it, which will contribute to the additional development of suspended constructions in the coming decades [9][12][13].

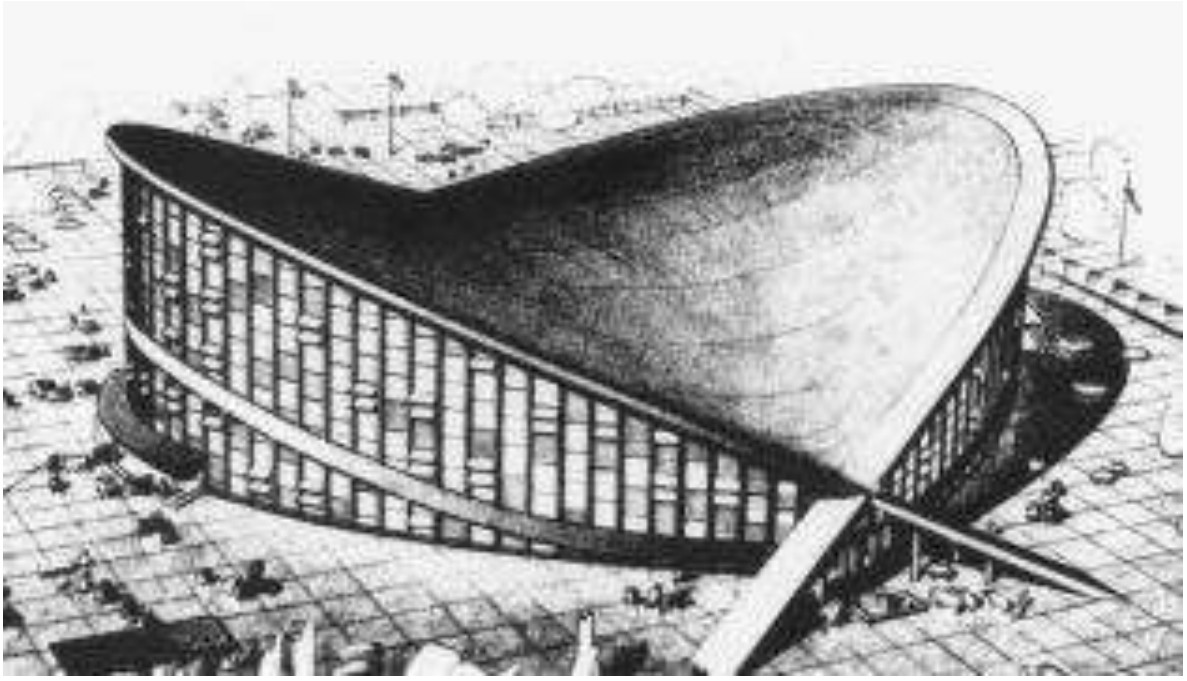


Fig. 2 Dorton Arena, Raleigh, North Carolina, USA, 1952 (Maciej Nowicki)

3.2. Basic principles of analytical solution

After the sudden death of Professor Nowicki, his theoretical work on solving the stability problem of suspended cable structures remains unfinished. For this reason, Prof. Balgač builds on Nowicki's work and succeeds in improving the method. The main feature of the theoretical work of Prof. Balgač is reflected in the introduction of numericals into the analytical method started by Nowicki. Professor Balgač based the calculations for achieving roof stability on his original method, which, for pre-stressed networks, implied an additional load on the supporting cables. The establishment of balance is obtained by additional load of the supporting cables in direct contact with the prestressing cables, thus transferring the reactive load which in the unloaded phase causes tension in the stabilizing cables. In the supporting cable, the highest forces will occur in the phase of the highest gravity load, and the least in the phase of prestress, on the contrary, in the stabilizing cable, the highest forces will occur in the phase of prestress, and the least in the phase of full load. For the input data of the calculation, the determination of the arrow of the supporting and connecting cable in all load phases and the determination of the loads acting on the cables, taking into account the correctly calculated values of k and $k1$, were applied. According to the professor's method, the additional load (v) should have an intensity of $0.15\ kN/m^2$ to $0.20\ kN/m^2$ [5][6].

4. Selected advanced constructions of prof. Balgač

Conventional structural systems were not sufficient to meet the requirements of creating aesthetically attractive, functional and economical buildings of medium and large spans. The modern approach requires optimization of construction with minimal use of materials and faster construction, which led to the development of suspended roof systems. These systems allow elegant covering of large spans without the need for internal supports, using high tensile strength materials. Architecture later embraced hanging systems, resulting in diverse variations. Steel tensioners, ropes and cables have become standard materials, manufactured to high steel standards. Research has improved the efficiency of building structures, encouraging further research.

The use of suspended roof systems to cover large areas brings innovative, bold and economical solutions. These systems are integrally tensioned structures, the result of the

geometric characteristics of load-bearing roof elements, i.e. cables. The cables do not transmit pressure forces, but only tension forces, and they are extremely long and have small cross-sections, which means that their bending stiffness is zero. Their carrying capacity is maximally used, making them economical structural systems. Fast preparation through CAD and CAM technology, cheap transportation, simple connection of elements with less qualified labor represent the advantages of suspended systems compared to other constructions for covering large spans. However, suspended roofs are subject to aerodynamic challenges due to loads such as self-weight, snow, wind, earthquakes and other vibrations. The stability of the structure becomes a key task in order to prevent negative effects, such as swaying of the roof, cracking of the ceiling and damage to the installations. Modern double layer cable systems are used for stabilization, applying weight loading, prestressing and combination with bending elements. A modern approach involves prestressing by introducing tensile forces into the roof cables during construction. These cables, when pre-tensioned, exert pressure on the structural elements, and stabilization is achieved by using elements of high compressive strength and cables of opposite curvature in different spatial arrangements [5][6].

The many advantages that characterize hanging chain systems led to their massive application in the middle of the 20th century, both in the world and in the territory of the former Yugoslavia. Among the engineers who created halls in this system, the name of prof. Edmund Balgač, who left behind several halls built in the hanging chain system. As part of this work, a case study will be conducted on the facilities - the Great Hall of the Textile Fair in Leskovac, the Fair and Sports Hall in Subotica and the Sports and Cultural Center "Pinki" in Zemun. The unique form of the mentioned halls led to the fact that even after 6 - 7 decades of existence, the mentioned halls captivate with their aesthetic value and represent rappers in the cities where they are located. Years of service, adaptable functionality and indisputable aesthetic value have contributed to the fact that we look at the mentioned objects through the prism of cultural monuments, as well as the architectural heritage of an era.

4.1.The Great Hall of the Textile Fair in Leskovac

The Great Hall of the Leskovac Textile Fair, also known as the Round Pavilion or "Šajkača" (traditional Serbian hat), was designed by Prof. Edmund Balgač and architect Cvetić Milorad. The pavilion is the second facility in the world and the first in Europe built according to the revolutionary design and construction principles pioneered by the famous "Dorton Arena" in the United States of America. The building was built in 1959 in the eastern part of the Fair complex and has a characteristic circular shape (Fig. 3). The constructive system of the roof is provided in the form of a pre-stressed network of steel cables, in the shape of a hyperbolic paraboloid and cross section 8Φ5 mm (carrying cables) and 3Φ5 mm (auxiliary cables). The steel ropes are stretched on two supports with a section of 70/300 cm, inclined outwards in the form of pinched arches, thus forming an approximately circular base with a diameter of 60 m. The perimeter supports are supported by slender columns with a section of 40/90 cm and variable height. The facade consists of parapets with a height of 180 cm from brick and glass to the supports. The gallery, in the form of an inner ring, separated from the main structure, has a width of 8 meters and staircases on the north and south sides. Although it was originally planned to be covered with copper, the pavilion is actually covered with galvanized sheet metal. The ceiling is made of lightweight volcanic tuff concrete with an average thickness of 6 cm. The total usable area of the building is 3440 m^2, where the ground floor covers 2375 m^2, and the gallery 1065 m^2 [3][4][9].

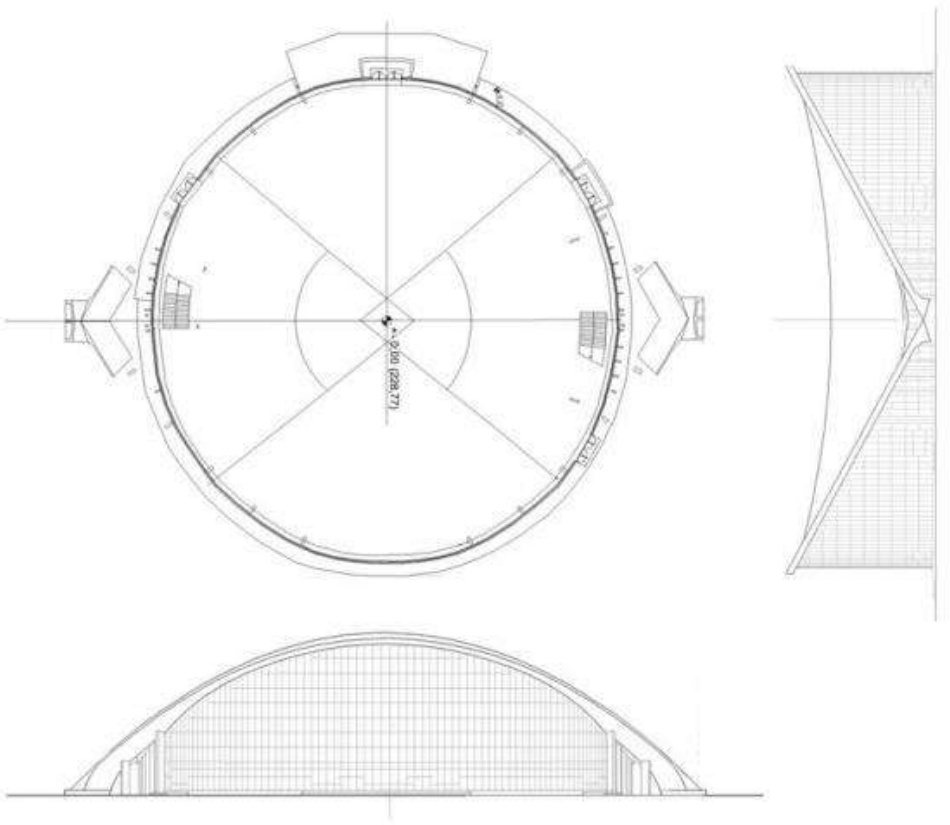


Fig. 3 Basis and facades of the Large hall of the Textile fair in Leskovac, Serbia, 1959

The building represents a cultural and historical place in Leskovac, which derives its importance from a deep-rooted connection with the city's identity as a center of free market exchange. Throughout history, it has been of great importance to the local community that is rising towards a sustainable presence in the global market by specializing in the free exchange of economic goods. The construction of the Round Pavilion as a fair destination was a logical consequence of this development process, given the vitality and strength that the city displayed, despite the anti-market approaches that had come into force by then. Today, places like the Round Pavilion, which base their lives on the universal principles of private property, freedom of contract and responsibility, are global symbols of human progress. Its historical value is emphasized by the fact that it is an expression of the persistence of the spirit of enterprising people who preserve their heritage in a way that relates to the future. The construction of the Round Pavilion at the end of the 1950s represents a kind of testament to cultural freedom, which, from a historical perspective, is inevitable and without an alternative.

By the act of the "Institute for the Protection of Cultural Monuments Nis" from 2004, the Fair complex, and thus the Round Pavilion itself, was included in the category of Immovable cultural property. However, in 2010, the Municipality of Leskovac initiates the privatization of the building and it is removed from the list of Immovable cultural assets of the city of Leskovac, thereby losing legal protection and creating the opportunity for new owners to demolish it. The new owners submit a plan for the demolition of the buildings and the construction of a new residential complex, to which there is opposition from the citizens. After many petitions from citizens and then the initiative of the Municipality of Leskovac, under public pressure, the demolition of the Round Pavilion is abandoned. The revitalization of the building was partially started in 2017, while in 2021 the building will

wear a new outfit and restore its shine. The pavilion was reopened in 2021, now in a new role as a shopping center (Fig. 4). The inner ring of the gallery space was unfortunately permanently removed from the building due to the lack of compatibility with the new function of the building. In the process of renovation, the roof structure made of concrete elements and the covering were renewed, while the ropes remained preserved and thus the unique geometric form of the roof structure [4][10].



Fig. 4 After renovation works - The Large hall of the Textile fair in Leskovac, Serbia, 2011

4.2.Sports and Fair Hall in Subotica

According to the project of prof. Balgač Edmund and architect Ivan Antić, in 1969, the Sports and Fair Hall in Subotica was realized (Fig. 5). The location of "Dudova šuma" is planned for the construction of the new hall. The building is characterized by a square base, sides 57,6 m, dominated by an attractive roof structure with a geometric shape of a hyperbolic paraboloid. The new facility was intended for multifunctional use with simple adaptation, both for fair exhibitions and numerous sports and cultural and artistic events. The attractive construction of the hyperbolic paraboloid was realized in the form of a suspended structure of a pre-tensioned cable network, like the hall in Leskovac. Steel cables of section 6Φ5 mm placed at a certain distance form the supporting cable structure, while the auxiliary cables are of section 3Φ5 mm. A network of prestressed cables is tensioned on a reinforced concrete frame. The frame construction consists of two clamped triangular concrete elements. The triangular structure has different slopes which results in the opposite ends having different roof heights. The edge construction of the concrete frame is supported by columns placed at a distance of 7.2 m. Edge posts are of variable height and cross-section. The tallest column has a cross section of 50/120 cm [1][9].

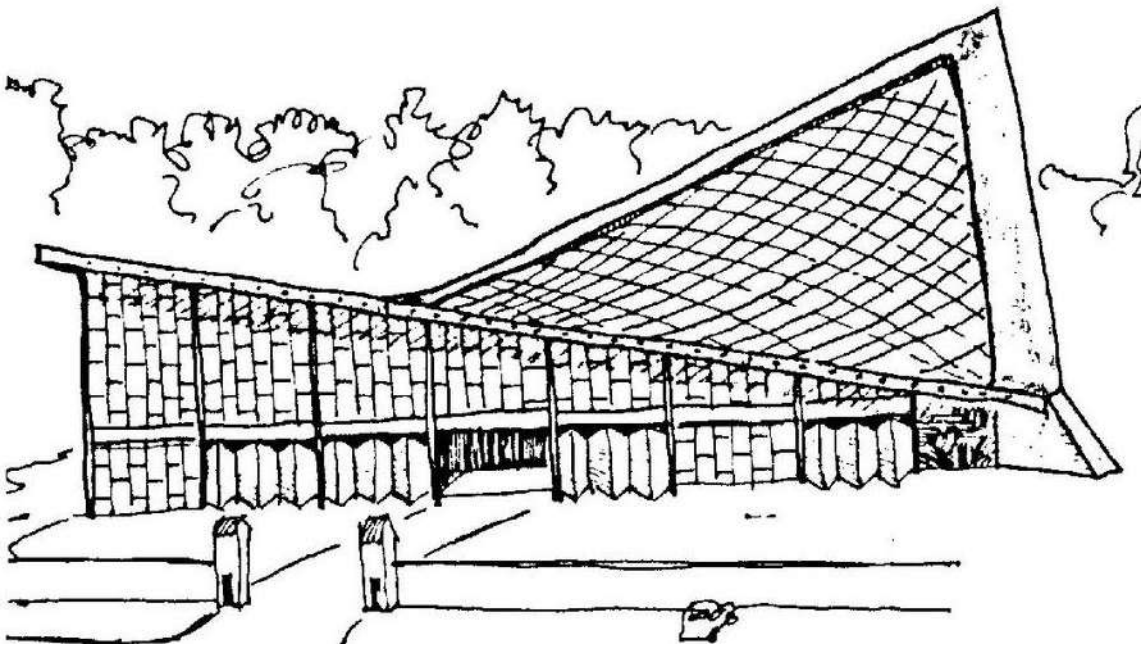


Fig. 5 Sports and Fair Hall in Subotica, Serbia, 1969

The major problem of the hall from the very beginning was the leaking roof and the large volume of the space, which created difficulties for heating the space. For this reason, after the construction of the hall, it goes through two reconstruction processes. The first reconstruction of the building was carried out in 1988. Prefab stands were built into the hall space, after which the facility acquired a purely sporting character. The hall becomes divided into two parts after the reconstruction, i.e. two separate courts and a larger spectator capacity were obtained. During 2011, the stands were again reconstructed (Fig. 6). Although there was a reduction in the capacity of the spectator seats, the reconstruction of the stands was carried out due to the discomfort of the existing space for spectators. The problem of the existing bleachers was reflected in the non-standard dimensioning of the seating area, which is why such works were necessary. After 2011, the space of the hall remains unchanged, with the hope that in the future there will be a solution that will improve the possibilities of using the space [11].



Fig. 6 Reconstruction of the stands - Sports and fair hall in Subotica, Serbia, 2011

The Fair and Sports Hall in Subotica is an exceptional example of architecture and cultural heritage that testifies to the rich history and development of the city. This imposing sports hall bears the stamp of an impressive architectural style, recognizable by its elegant roof structure and adaptive function. In addition to serving as a venue for sporting events, the Hall was the central gathering place for citizens during various cultural events and manifestations, making it an indispensable point in the cultural life of Subotica. Its architectural value is reflected in the careful balancing between aesthetics and functionality, which contributed to its status as a cultural monument. In January 2022, the Fair and Sports Hall in Subotica was considered by the Institute for the Protection of Cultural Monuments, but after assessing the historical, artistic and cultural value of the building, it was not included in the list of Immovable Cultural Properties.

4.3.Sports and Cultural Center "Pinki" in Zemun

The "Pinki" Hall was built in Zemun in 1973 as the "Pinki" House of Sports, Youth and Pioneers. It was named after the national hero of Yugoslavia Boško Palkovljević Pinki. Responsibility for the design of the new hall fell to prof. Balgač Edmund and architect Ivan Antić. For the design of the Sports and Cultural Center, the designers were set strict requirements by the Institute for Urban Planning of the City of Bograd, such as strict compliance with regulatory lines and heights. The requirements were skilfully met and a building was designed that fits perfectly into the urban structure. Rectangular in shape, the building is 68 m wide and 54 m long. Structural elements and design were subordinated to the smooth functioning of the building. The roof construction of concave shape gives character to the building (Fig. 7). The main facade of the building clearly represents the function and structure of the building, acting as a precise mechanical system that serves sports. The material chosen for the facade perfectly places the building in the old core of the municipality of Zemun, at the same time showing the beauty of the modern buildings of that time. This construction project, simple at first glance, radiates elegance, especially thanks to the ease of reading. The cubic shape softens the shape of the concave roof, which appears on the facade as a thin paraboloid above a large wall of glass. The slope of the roof follows the slopes of the stands and stairs, allowing for easy tracking of movement inside the building. Side approaches to the building contribute to its monumentality [7][8].



Fig. 7 Main facade - Sports and cultural center "Pinki" in Zemun, Serbia, 1973

Roof structure, according to the project of prof. Balgač is solved in the form of a unidirectional curved suspended cable roof. The original proposal was to make a roof

structure out of wood with cables running through it, but this was rejected. In the final design, wood was replaced with concrete. The flexible cables are composed of prefabricated reinforced concrete elements "pearls" that are filled at the joints, and after that they take the form of a monolithic structure by prestressing. Ropes with a section of 6Φ7 mm are attached to the frame structure at a distance of 7.2 m. The continuous frame construction is additionally reinforced with crossbars. The pillars of the frame structure are placed at a distance of 14.4 m. In order to allow the acceptance of vertical and horizontal components of the roof, additional elements were introduced, which formed a lattice support. A trapezoidal steel sheet with a thickness of 1.5 mm and a fold height of 3 cm was placed over the pre-stressed roof cables, which was then filled with perlite concrete. The specific solution of this object was also represented by the construction of its foundations. Due to the high level of underground water caused by the proximity of the Danube river, in addition to the transfer of cargo, it was necessary to protect the underground premises from water penetration. The contractor used new machinery and special waterproofing to make this possible. The rib base plate was chosen for the construction because of the heavy loads it had to carry. The structure of the pool is recessed in relation to the rest of the building and is connected to the foundations via a neoprene pad. The pool is deeper on the side under the diving board, so this difference in height is used to accommodate the equipment needed for the pool, but which does not require full floor height. On this small segment of the building (Fig. 8), one can see the skill of the designer who simultaneously thinks about function and structure [2][8][9].

This type of construction is special because its horizontal forces form a closed system. Horizontal forces from the roof are transmitted through horizontal and vertical trusses, which transmit vertical forces to the foundations. At the bottom of the side column, the forces change from tension to load as they are used as a support for the slabs. In the structural analysis of the building shown by the master plan, it is possible to see the relationship between the reinforcement and the prestressed cables in these columns. Prestressed bars are used to transfer the stress, while the reinforcement is calculated for the load [5].

Hall "Pinki" in Zemun is a sports and cultural center that has a long history and is an important part of the local community. It was built to serve the various needs of the community, including sporting events, cultural events and other activities. The Sports and Cultural Center facility houses an arena for small sports with a grandstand for 3,000 spectators, a 25 x 12.5 m swimming pool with a diving board, as well as other supporting services. In addition to its basic function, the hall also plays a significant role in the silhouette of Zemun, where it represents one of the symbols of the old part of Belgrade with its recognizable form of roof structure. At the beginning of the 21st century, the first and so far the only reconstruction was carried out. The reconstruction project included the restoration of the roof, which began to leak due to the wear and tear of the roof covering. The largest decline of the hall is experienced in 2021, when there is an initiative for its demolition. Fortunately, there is an agreement between the administration and the authorities and the hall remains in its place. The city of Belgrade is launching a renovation initiative with the aim of sustainability, and as a first step in 2023, the construction of an underground garage under the plateau of the "Pinki" hall was initiated. The Municipality of Zemun, in the Secretariat for Investments, also launched an initiative to create a reconstruction plan for the building itself in order to improve its functionality and adapt it to modern needs [12].

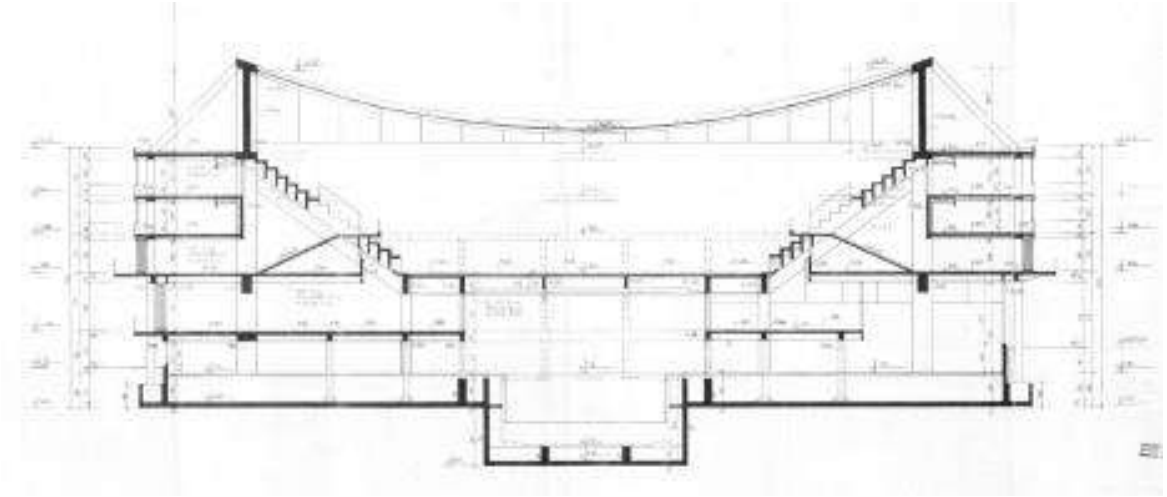


Fig. 8 Section - Sports and cultural center "Pinki" in Zemun, Serbia, 1973

4. Conclusion

Within this paper, the importance of the architectural heritage of Professor Edmund Balgač was investigated, focusing on the analysis of suspended structures and their application in buildings such as the Great Hall of the Textile Fair in Leskovac, the Fair and Sports Hall in Subotica and the Sports and Cultural Center "Pinki" in Zemun. Realized projects are conditioned by the theoretical solutions of prof. Balgač, which represent the continuation of world trends in architectural design and construction.

The theoretical contribution of Prof. Balgač arises from the improvement of analytical methods for the calculation of suspended cable structures, continuing the work of pioneers such as David Jawerth and Maciej Nowicki. With his theoretical contribution, prof. Balgač laid the foundations for the practical application of suspended structures.

The analysis of the selected objects testifies to the uniqueness of form, functionality and aesthetics of hanging constructions. These buildings, even decades after their construction, retain their cultural and architectural significance, representing benchmarks in urban landscapes.

The aforementioned works also pointed out the challenges that accompany suspended structures, including aerodynamic factors and the need for stabilization to preserve structural integrity. The changes that some of the buildings experienced through the reconstruction process were also shown, highlighting the need for maintenance and adaptation of structures over time.

Finally, the paper emphasized the importance of suspended structures as a cultural and construction heritage, investigating their influence on the architectural landscape and contribution to the creativity of prof. Balgač in this domain.

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RECONSTRUCTION OF CASTLES AND MANSIONS IN SLOVAKIA NOT FINANCED BY THE STATE

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Abstract: *There are more than 17 000 monuments in Slovakia* including many *national cultural monuments of different kinds. 717 of immovable national cultural monuments are in the list created by responsible ministries, which defines monuments with priority for protection and restoration. It is obvious that the help of private individuals and volunteers is essential in the restoration of many monuments. Examples of reconstructions of historical buildings that were not financed by the state are described in the paper: Water Castle in Hronsek, Ruffínyi's villa in Dobšiná, Manor house in Tajná, Manor house in Záborské, extramural settlment of former Sklabiňa Castle and Hrušov Castle.*

Key words*: reconstruction, castles and mansions, education of structural engineers*

1. Introduction

The list of national cultural monuments with priority for protection and restoration in Slovakia contains [1]: A) immovable national cultural monuments, B) movable national cultural monuments. The list of immovable national cultural monuments consists of: a) 13 archaeological monuments, b) 64 castles, châteaux and fortifications, c) 88 mansions and manor-houses, d) 144 churches, monasteries, calvaries, bell towers, synagogues (sacral architecture), e) 33 wooden churches (sacral architecture), f) 36 medieval wall paintings in churches (sacral architecture), g) 44 town halls, schools, theaters, redoubts, museums, spa houses, hotels, etc. (public buildings), h) 6 city palaces, i) 31 monuments of history, j) 47 technical monuments, k) 33 architecture of the 20th century, l) 19 parks and gardens, m) 23 other (statues, towers, stud farm, barracks, etc.), n) 136 monument reserves. The list of movable national cultural monuments consists of enormous number of items.

In March 2023, the Ministry of Culture of the Slovak Republic identified 24 priority national cultural monuments owned by the state, for the restoration of which it wants to use 220 950 000 euros from European funds, available within the new program period 2021 - 2027.

In the proposed list there is also a proposal for a specific amount intended for restoration of the first building of Matica slovenská in Martin, Galéria Ľ. Fullu in

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Ružomberok, the Museum of the Slovak National Councils in Myjava, the house of Master Pavel in Levoča, the mansion in Svidník - Dezider Milly Gallery, the birthplace of Vladimír Clementis in Tisovec, the mansion in Dolná Strehová, the Museum of Czech Culture in Slovakia, the Carthusian monastery in the Červený Kláštor and Penov house in Banská Bystrica. Also the Strážky manor, the Betliar manor, the Museum of Ukrainian Culture in Svidník, the Dolná Krupá manor and also the Blue Stone manor and castle. The list also includes Spiš Castle, Zvolenský and Bojnický castles and the Ethnographic Museum in Martin.

Among the four monuments that are under the administration of other ministries, the SNP Museum - the monument in Banská Bystrica and the exhibition in Kalište (founded by the Ministry of Defense of the Slovak Republic), the manor house in Svätý Anton (administered by the Ministry of Agriculture and Rural Development of the Slovak Republic) and The Jozef Kollár Gallery in Banská Štiavnica, which is managed by the Ministry of the Environment of the Slovak Republic, were included in the list.

The individual amounts and the total allocation of 220 950 000 euros are based on estimates based on prices in 2022.

The ministry reminds that of the more than 17 000 monuments in Slovakia, 25 percent are in a damaged state, of which five percent are in a desolate state and points out that as a result of long-term underfunding of the restoration of the monument fund, investment debt at the level of 5.4 billion euros has accumulated in this area during previous periods.

It is obvious that the help of private individuals and volunteers is essential in the restoration of monuments. Examples of reconstructions of historical buildings that were not financed by the state are described in the next part of the paper.

2. Water Castle in Hronsek near city Sliač [2, 3, 4, 5]

There was once a moat around the historical building, therefore it is known as a water castle. The moat served as a defense element against the enemies. In the past, various wastes ended up in the moat, especially excrement from the toilets, leftovers from the kitchen, etc.

Who knows what fate would have awaited the iconic building in a small central Slovakian village Hronsek, if it had not been acquired after long years of waiting by Mr. Stanislav Rajnoha, the person who knows exactly how to deal with dilapidated historical buildings [2].

The castle, which avoided nationalization in 1948, was bought in 1961 by the state from the then owners. Although state planned to improve castle condition, it never happened. In the 1980s, the castle was managed by the Banská Bystrica Regional Monument Office. At that time, Stanislav Rajnoha was working in the State Restoration Studios as an art historian.

In 1983, a research on wall paintings was performed in the castle. These wall paintings are works by amateur painters and trained artists based on Italian designs. They are all from the 16th century. The object immediately caught Rajnohas eye. Six years passed and the project preparation was already underway and it was supposed to move in the castle as a state-owned company. However, after the fall of the former regime in 1989, there were changes, so Rajnoha said to himself that what we missed as a state-owned company, I will try to catch up," recalls the ambitious businessman.

Stanislav Rajnoha applied for the purchase of the building in 1990. It took 12 years before he got the castle. Stanislav Rajnoha became the owner of a 1.7-hectare plot of land with a dilapidated castle

The Water Castle in Hronsek from the 14th century, which was rebuilt to its current form at the end of the 16th century, was in a truly deplorable state in 1990. Leaky roofs,

broken windows, deep cracks in the walls, a disappeared tower, and especially the gradually disappearing unique items that can be seen only in a few places. The castle is unique especially with its rich fresco decoration.

Stanislav Rajnoha rebuilt the castle out of his own pocket, he only paid for the reconstruction of the roof from a contribution from the Let's Renovate Your House program. Finally, he decided to make it available to the public.

For the first time 40 years ago, Rajnohas steps were directed to the devastated building, inextricably linked with the families of Géczy, Šóš, Battik, Schmideg, Róth and Teleky. At that time, he had no idea that later the ruin would employ him for more than 20 years and he would invest 5 million euros in the castle.

At that time, probably few could have imagined what form the castle would gradually take. Today, it attracts attention, there are more and more curious people who want to see the once noble residence. It also caught the attention of filmmakers who filmed the fairy tale Johanka's secret there.

Rajnoha pointed out that renovating such an object teaches patience. It would be naive to think that somebody will live in the castle or use it two to three years after its purchase.

Rajnoha claims that the restoration took place comfortably, because nobody was chasing him, they were not pushing for specific deadlines for subsidized projects, on which more than one municipality has already burned out. Rajnoha was able to examine the building in peace. The first four years were strictly a rescue phase, focused on cleaning, the roof and statics. Only when everything was cleaned and stabilized did he start working on the murals. Rajnoha knew that the work would take years. The technology of procedures cannot be accelerated. In addition, his priority was to give the building its original form.



a) **b)**

Fig. 1. a) Historian Stanislav Rajnoha, owner of the Water Castle in Hronsek; **b)** castle in the year 2016. Source: Ján Krošlák/Petit Press, [3].



a) **b)**

Fig. 2. Water Castle in Hronsek in the year 2016. Source: Ján Krošlák/Petit Press, [3].

Figures 3-7 show situation in the year 2023 [2], figures 8-10 show the progress of the reconstruction and figures 11-13 show the frescoes from the 16th century.



a)



b)

Fig. 3. a) Water Castle in Hronsek in the year 2023. **b)** Stanislav Rajnoha, owner of the castle. Source: Marcela Ballová, [2].



Fig. 4. Water Castle in Hronsek in the year 2023. Source: Marcela Ballová, [2].



Fig. 5. Water Castle in Hronsek in the year 2023. Source: Marcela Ballová, [2].



Fig. 6. Water Castle in Hronsek in the year 2023. Source: Marcela Ballová, [2].



a) **b)**

Fig. 7. Water Castle in Hronsek in the year 2023. Source: Marcela Ballová, [2].



a) **b)**

c) foundation of dissapeared tower **d) new tower**

Fig. 8. The progress of the reconstruction. Source: Vodný hrad [4].



Fig. 9. The progress of the reconstruction. Source: Vodný hrad [4].



Fig. 10. The progress of the reconstruction. Source: Vodný hrad [4].



a)



b)

Fig. 11. The frescoes from the 16th century. Source: Marcela Ballová [5].



a)



b)



c)

Fig. 12. The frescoes from the 16th century. Source: Marcela Ballová [5].



a)



b)

Fig. 13. The frescoes from the 16th century. Source: Marcela Ballová [5].

3. Ruffínyi's villa in Dobšiná [6]

The villa was built by Eugen Ruffínyi (*1.3.1846 – †13.1.1924). He was a mining engineer, caver, co-discoverer of the Dobšinská Ice Cave.

When Ladislav Vagner, as the owner, wanted to restore Ruffínyi's dilapidated villa in Dobšina in the past, he ran into a problem that is common in similar cases - lack of funds.

At the same time, he developed two projects for the restoration of cultural monuments. First, there was supposed to be a home for seniors, later he wanted to build a training workplace for youth. Students would go there on field trips. It would be the center of speleology in Slovakia, as the villa was built by one of the discoverers of the Dobšinská Ice Cave. He already had a building permit issued for these plans. Vagner made an enormous effort to save the villa, but it was not possible without subsidies from the state. The state in the past twice rejected his request for financial support to restore the villa.

The city Dobšiná wanted to buy the villa, but they did not agree with the owner, as he already had plans to renovate it. However, his plans fell through due to lack of money. After negotiations with Mayor of Dobšiná Mr. Ján Slovák, Vagner agreed that the city would receive the villa as a gift. But the Dobšiná city did not have money in the budget not only for the restoration of the dilapidated monument, but also for basic maintenance, so the deputies rejected Vagner's offer. It already looked like the historic building was simply condemned to a gradual demise. It would not be the only monument that would end up like

this in Slovakia. However, the mayor and deputies finally came to an agreement and the villa now belongs to the city. Since it is a cultural monument, by law they have to keep it in good condition. Vagner handed over the villa to the city together with his two projects.



Fig. 14. Ruffínyi's villa in Dobšiná. Source: Čierne diery.[6].

4. Manor house in Tajná near Vráble [7]

The state is again looking for a buyer for the manor house in Tajná, which is a national cultural monument. There was an orphanage in it. Last year, the minimum price was set at 1 010 000 euros, based on an assessment from November 2020. The new price this time is 955 000 euros. It is based on a new assessment that was prepared in April 2023.



Fig. 15. Manor house in Tajná. Source: Miriam Hojčušová [7].

5. Manor house in Záborské near Prešov [8]

Originally a Renaissance mansion, also called the Záturecký**'s** mansion, it was built in 1614 in the central part of the village. In 1741, it was rebuilt in the Baroque style. Further structural modifications to the mansion were made in the middle of the 19th century. Since 1963, it has been registered in the Central List of Monument Fund, but the building has been unused for a long time and is falling into disrepair. There are 20 rooms in it. The state sold the monument to a private person, then to another, and it is for sale again. Much more expensive.



Fig. 16. Manor house in Záborské near Prešov. Source: reality.sk [8].



Fig. 17. Manor house in Záborské near Prešov. Source: Pamiatky na Slovensku [8].

6. Extramural settlement of former Sklabiňa castle near Martin [9]

Extramural settlement of former Sklabiňa castle was burned down by the Hussites and, at the end of the Second World War, also by German troops. Sklabiňa Castle turned into a ruin. Extramural settlement of former Sklabiňa castle have been restored by volunteers for years without state support. In the past, the building served as an artillery fortification, and in other parts there was a carriage house and a castle brewery. In the first week of November, a fire broke out in the castle grounds, and the rest of the biggest castle in Turiec was once again engulfed in flames after decades. The fire affected the farm building in front of the entrance gate. The attic, which was used for housing volunteers working on the restoration of the castle, caught fire. A castle museum was established right under them. During the fire, three attic rooms and part of the roof burned down. And where the flames did not reach, there the water left its consequences. During the extinguishing, the water flooded both rooms of the museum exhibition, and seeped into the basement. The exhibits are soaked, some of them were moved out and are now being dried.

The amount of damage was estimated at approximately 60 000 euros. But something that cannot be quantified was also burned, namely the long-term work and efforts of people who work daily without state support to restore the rest of the castle. After the fire at extramural settlement of former Sklabiňa castle, volunteers are repairing the damage. People supporting their work announced a public money collection.



Fig. 18. Extramural settlment of former Sklabiňa Castle near Martin. Source: archiv Michal Valko [9].

7. Civic associations of volunteers reconstruct castles [10, 11]

On the territory of Slovakia, we register around 180 known castle sites. The term castle refers to building complexes built in the high to late Middle Ages, i.e. roughly in the 12th to 15th centuries, which are divided into at least two parts, the so-called castle core and forecourt. This set does not include small one-piece castles, fortified mansions, watchtowers, fortified churches, courtyards, etc., which would multiply the mentioned number. Around 90 percent of the castle ruins were subject to decay almost unnoticed.
Today, we have only about 25 usable and maintained covered castles in Slovakia, which is roughly 14% of their total number. Approximately half of the castles were preserved only in the state of castle ruins, and another third only in the form of small relics, after which

only unevenness remained in the terrain. We cannot even locate the remaining 10% of the castles exactly. We know them only from written sources. In the future, however, they may be located on the basis of archival or field research [10].

Civic associations have a large participation in the renovation of castles. Just one example. The more than 750-year-old Hrušov Castle, located between villages Hostie and Skýcov, stretches over an area of approximately half a hectare on a quartzite rock. For less than twenty years, the civic association (OZ – Občianske združenie in Slovak) Laustach has been restoring castle using original technologies. Restoration of the castle using original technologies is very difficult, but it prolongs life of the castle.



Fig. 19. Hrušov Castle in 2006. Source: Archív OZ Laustach. [11].



Fig. 20. Hrušov Castle in 2022. Source: Archív OZ Laustach. [11]

Conclusion

Information about reconstruction of cultural monuments in Slovakia by state, cities, civic associations, and individuals.

Examples of reconstructions of historical buildings that were not financed by the state are described in the paper: Water Castle in Hronsek, Ruffínyi's villa in Dobšiná, Manor house in Tajná, Manor house in Záborské, extramural settlment o the former Sklabiňa Castle and Hrušov Castle.

The Slovak and foreign students are informed by members of the Department of Languages at Faculty of Civil Engineering, Slovak University of Technology in Bratislava about current problems related to the architecture and civil engineering. The topic related to the reconstruction of castles was very interesting to the students, and several of them plan to work on the restoration of some important castle during the holidays.

Acknowledgement

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TRULLA BUILDING AND OVEN FROM THE MEDIEVAL TOWN OF LYUTITSA, NEAR IVAYLOVGRAD, HASKOVO DISTRICT

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Abstract: *The medieval town of Lyutitsa is in the Eastern Rhodopes, northwest of Ivaylovgrad. It is an impressive medieval centre and diocese from the 9th c. It is important to note that it is the best preserved monument in the Bulgarian lands. The fortification walls, over 600 m long, enclose an area of 26 dca and some sections of the walls are preserved up to 14 m in height.*

Two stand-alone facilities were unearthed in 2022. Structure 1/2022 is an oval-shaped stone cluster, a pavement, and a thick brick-lined oven base built on the pavement at a height of 0.30 m. The stone spoil of Structure 1 measures NS-5.60 m and EW-5.30 m. Numerous large timber nails and small 'upholstery' nails were found in and around the structure. On the north side of the structure a 0.40 m high stone threshold was unearthed, in front of which there is a south facing pavement of local material and thin slabs varying in size from 0.10 m x 0.20 m to 0.40 m x 0.50 m. This is a building of the "Trulla" type registered in the UNESCO World Heritage Sites. The building was constructed and used in the 14th c.

Structure 2/2022 is a stone cluster - a fire chamber and scattered broken roof slabs. The fire chamber opens from the north-northwest. In front of the fire chamber there is a step 0.50 m x 2.10 m. The chamber itself measures 1.50 m x 1.80 m. Structure 2/2022 is a stand-alone cooking oven also dating to the 14th c.

***Key words**: Bulgarian lands, Lyutitsa, Trulla building, UNESCO, 14th century.*

The medieval town of Lyutitsa is to be found on one of the last hills of the Eastern Rhodopes. It is situated on the most southeastern point of the mountain range and has a strategic view of southern Thrace and medieval Adrianople, present-day Edirne in Turkey. The panorama and the strategic location make it possible to keep an eye also in the northern direction, i.e. on the main road to the present-day Bulgarian towns of Krumovgrad and Kardzhali. Lyutitsa was localized northwest, 6 km away from Ivaylovgrad and 45 km away from Adrianople. It was an impressive medieval centre and diocese as early as the 9th c. The medieval town is the best preserved monument on Bulgarian land, with fortification walls over 600 m long enclosing an area of 26 dca and in some places rising up to 14 m in height.

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Structures 1 and 2 were found near the entrance to the fortified area in 2022 (Fig. 1). They are heating facilities for cooking. Structure 1/2022 is closed with a timber frame, roofed with slabs, no bonding material and has an entrance from the northeast. Structure 1 is an oval-shaped structure with a pavement and oven, made of single-faced dressed stones bonded with mud. The heating facility survived at a height of 0.30 m and is lined with thick bricks. Parts of Structure 1 were localised in four squares B2, B3, D2, D3, revealing an oval plan which is no longer quite regular in shape at present. This is a perfectly natural process given the toll of time and the drastically destroyed structure. Three layers of slabs were removed during the excavations (Fig. 2). Many of them have fallen apart as a result of a fire in which the building collapsed. The facility had a massive stone oven measuring 2.44 m by 1.82 m, whose fire chamber was lined with bricks and slabs. The bricks are over 8 cm thick. The remains of the clay chimney that was above the oven were on the pavement of the building (Fig. 3). The chimney was erected on the south side of the facility and pieces of it fell to the north. The stone rubble from Structure 1 measures NS-5.60 m and EW-5.30 m. After a two-year excavations and clearing of the fallen slabs, the dimensions of the structure were identified as follows: NS-5.48 m and EW-5.20 m (Fig. 4). Around the pavement of the facility, where no footings or stake pits were found due to the soil structure, a solid amount of nails from the timbers and charcoal were found, giving us clues to the structure of the walls. Small 'upholstery' nails were also found holding textile or leather sheets that lined the walls (Fig. 5). One of the structures excavated during 2018-2020, next to the fortification wall, had the same dating from the late 13th and early 14th c. and contained similar supporting elements (Петрунова, Петрунов, Кирякова 2019, 533-535). The threshold stone was uncovered in situ also on the north; it is 0.40 m high. There is a pavement of local material extending southwards from it, fashioned into thin slabs of varying sizes from 0.10 m x 0.20 m to 0.40 m x 0.50 m. The threshold stone itself, remains partly outside the scope of the facility. The localized animal bones and the point that no iron slags and loose lumps of softened sponge iron were found, give reason to presume that the facility was used for food processing and to identify it as a 'Trullo' type building with a cooking oven. The animal bones that were unearthed during the course of the excavations were of the domesticated and wild animal species typical of the medieval period and commonly used for food. These included: sheep-goat, pig, cattle, deer, horse, wildfowl and tortoise (Fig. 6). The DIGITAL ELEVATION MODEL (DEM) is presented here, made in 2023, after year 2 of the excavations of Structure 1. The threshold stone is at 559.83 m above sea level (Fig. 7). Fortunately, through it, the contours of the facilities are most clearly visible.

Structure 2/2022, is a stone oven that was roofed with slabs and had a combustion chamber open from the north-northwest. A step, measuring 0.50 m x 2.10 m was formed in front of the combustion chamber (Fig. 8). The chamber itself measures 1.50 m x 1.80 m. A number of studies have been published on the kitchen and the dining table, which we shall not go into here. The main source where this is mentioned is the Typikon of Gregory Pakourianos, a Georgian by origin, adviser to Alexius I Comnenus (ГИБИ, 48-50). There are a number of publications on the Trullo type of buildings. Naturally, those originating in Italy stand out among them, but there is also plenty of research by professionals in the field of architecture

from England. For instance, the work of Edward Allen, first published in the 1960s. In his book "Stone Shelters", he discusses in detail the layout, the architectural parameters and the types of trulli from individual Italian regions. The trulli themselves appeared in Italy as early as prehistory, as cattle shelters, and their structure, although it needs no bonding material, is virtually everlasting (Rinaldi, Sterlacci, 251). At the beginning emerging as tombs known as Tolosi, the Trulli used after the 2nd c. AD already had an economic and residential function (Rinaldi, Sterlacci, 252). Their structure, entirely of stone or of stone and timber, does not change the type of architecture. Similarly, there are different trulli in plan, rectangular and round in shape. The meaning of the term Trullo originates from the domed conical roof, which actually gives the name to this type of buildings. Today, they are the pride of the Italian city of Alberobello in the province of Bari, in the region of Puglia, in southern Italy. They were registered as World Heritage Sites in 1996, and ever since they have been under the auspices of UNESCO. The process of protecting them as cultural heritage began in the early 20th c. A number of similar buildings, both single and terraced, can now be visited in the tourist village. Thanks to a change in the 18th c. that brought about the use of building bonds and mortar, visitors to Alberobello at present can spend the night, shop and eat in these whimsical architectural masterpieces. (Fig. 9).

The trullo from Lyutitsa, quite as may be expected, appeared at a time when the influence of the Western barons in the Balkans was at its best. Although the Byzantine emperors restored power in Constantinople, western principalities developed independently in the Peloponnese. After 1275 and after signing the Union between Rome and Constantinople, quite a few Italian crusaders were sent to the Balkans to spread and impose the new canonical norms adopted by the Patriarch of Constantinople. On the other hand, the point that the Bulgarian Kingdom was weakened and was no longer ruled by the Asenevs dynasty at the end of the 13th c., made it possible to impose a foreign and fundamentally unfamiliar cultural and, as is the case, architectural model in this medieval town. The stratigraphic position of these buildings, structures 1 and 2, implies precisely that. They are founded on a burnt layer of late 13th c. materials, sealing not only material traces of the money circulation in the period, but also a rich assortment of arrowheads, as well as other monuments not typical of Eastern European culture. The coins found in the unearthed area of the town, indicate exactly these processes and exactly the period when the buildings were in use, from 1285-90 to the 1350s and 1360s (Fig. 10), (Петрунов 2023, 105-116).

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Fig. 1

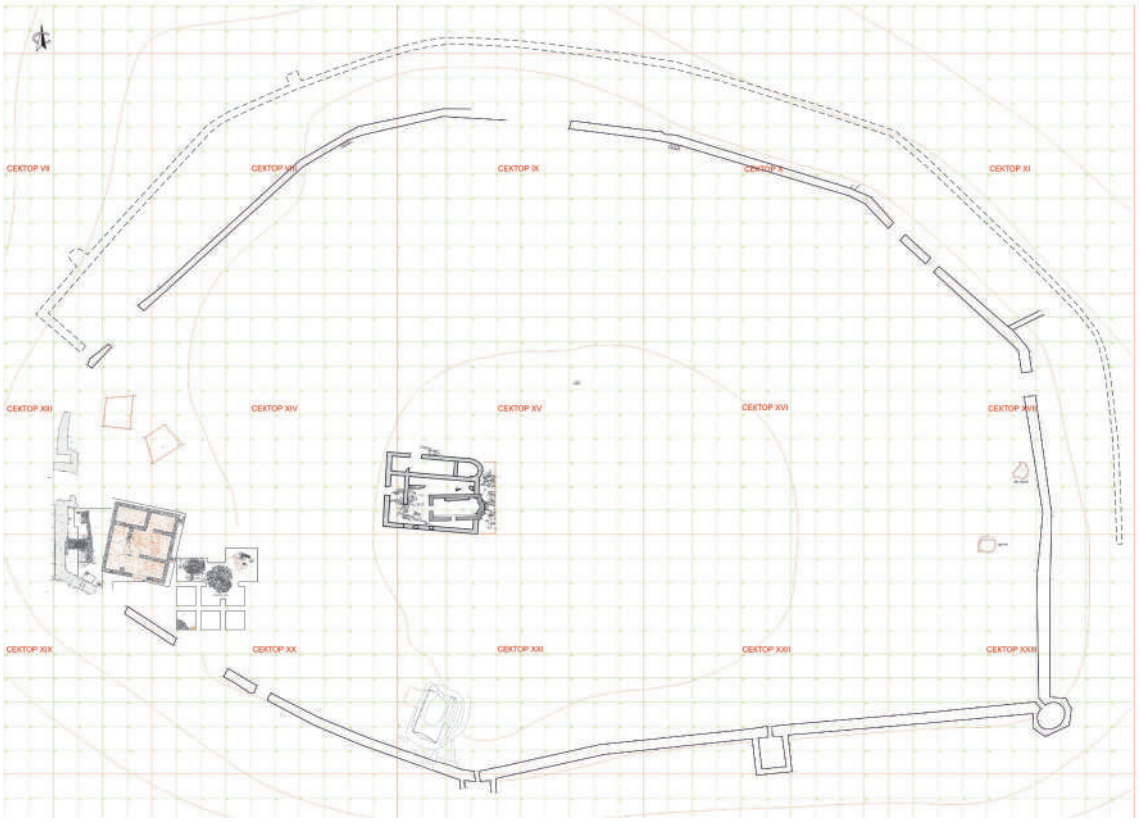


Fig. 2



Fig. 3



Fig. 4

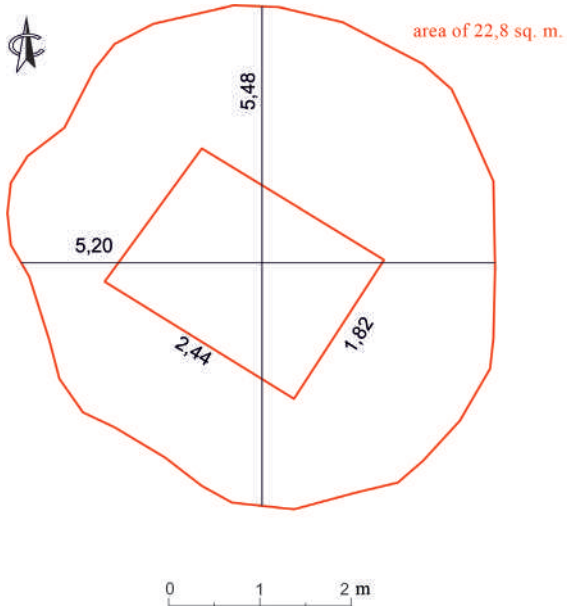


Fig. 5



Fig. 6

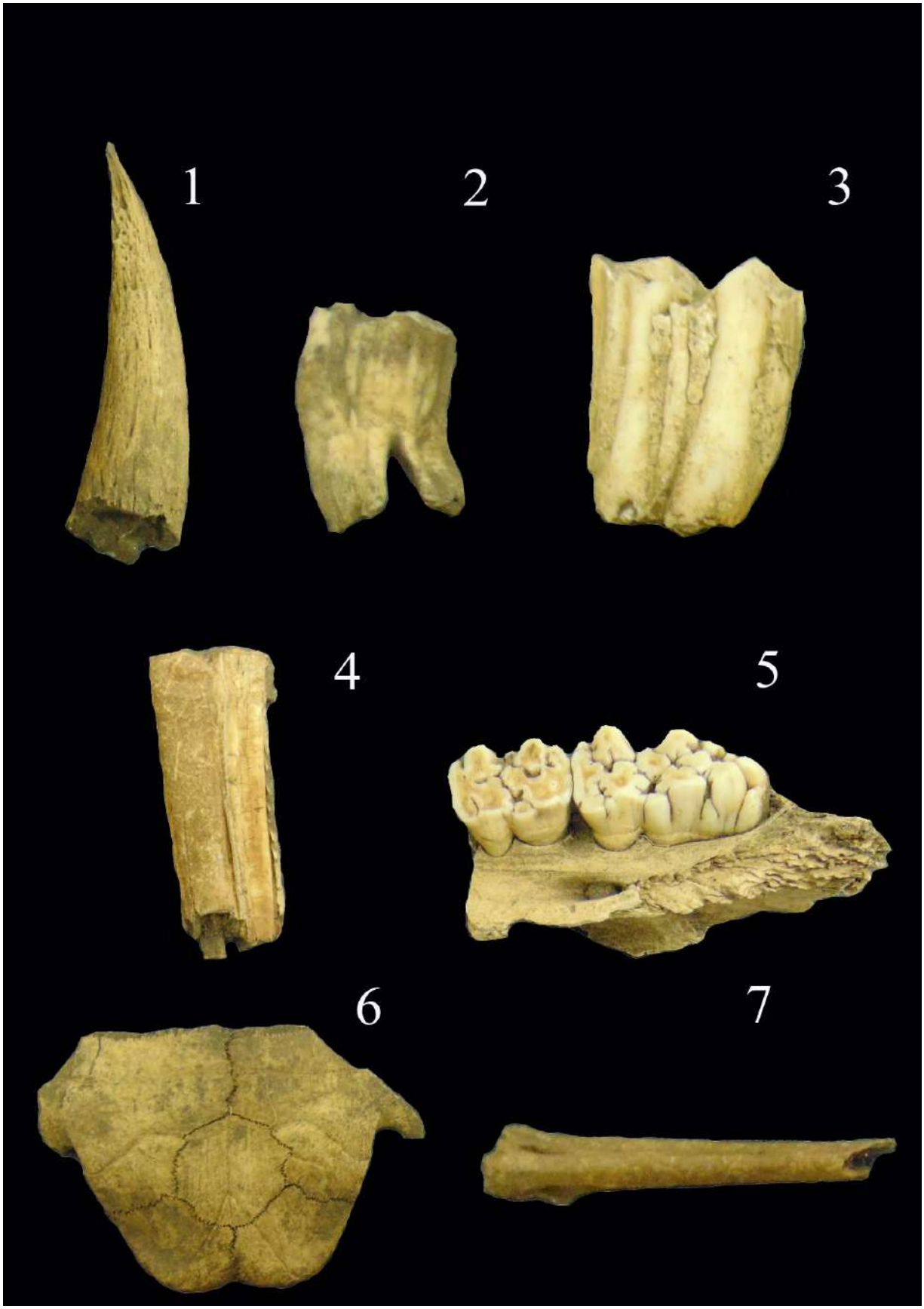


Fig. 7

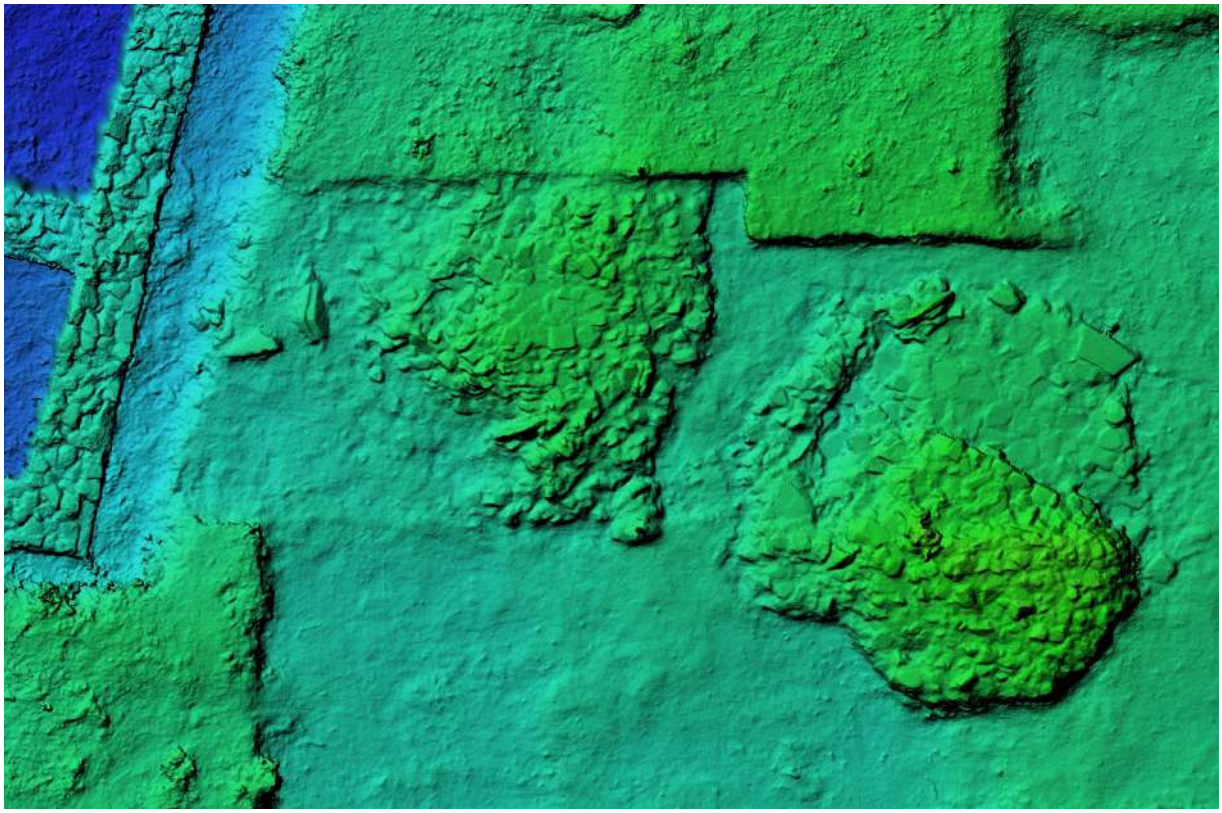


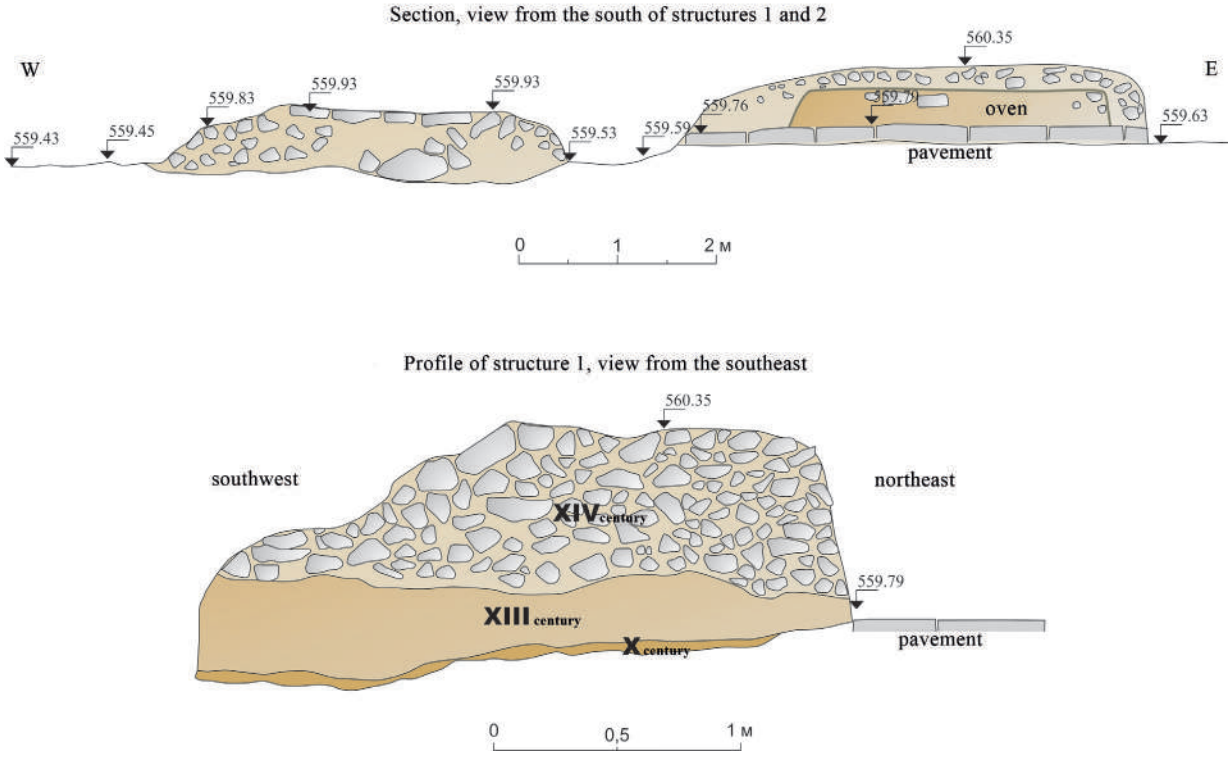
Fig. 8



Fig. 9



Fig. 10


Section, view from the south of structures 1 and 2
W
E
559.43
559.45
559.83
559.93
559.93
559.53
559.59
559.76
559.79
560.35
559.63
oven
pavement
0
1
2 м
Profile of structure 1, view from the southeast
560.35
southwest
northeast
XIV century
559.79
XIII century
X century
pavement
0
0,5
1 м

EARTHQUAKE IN THE SLOVAK REPUBLIC IN THE EDUCTION OF CIVIL ENGINEERING STUDENTS

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Abstract: *Past and future earthquakes in Slovakia. Map of earthquake epicentres in Slovakia. Territorial extent and result of the earthquake on October 9, 2023. Overview of locations with observation of earthquake effects based on macroseismic questionnaires received by 11/10/2023 8:08 a.m. (1824 questionnaires from 447 locations). Main findings from macroseismic questionnaires. Earthquake relation to nuclear power plant locations. Photographs of damaged mansions, churches, temples and family houses after earthquake in 9.10.2023. Monitoring of earthquakes in Slovakia. What should be done in Slovakia. Map of influenced areas and map of earthquake epicentres in Slovakia.*

Key words*: earthquakes in Slovakia, education of structural engineers*

1. Introduction

Contemporaries in Slovakia have never experienced anything like this. After the earthquake of October 9, 2023 with an epicenter in the cadastre of the village of Ďapalovce in Vranov nad Topľou district near Humenné (Fig. 2 and 22). The earthquake occurred on 09.10.2023 at 20:23:10, had a local magnitude of 4.9, epicenter coordinates +49.076,+21.759 and a hypocenter depth of 17.8 km. There were also 13 aftershocks. Two aftershocks could also be localized. The first of them was on 09.10.2023 at 20:39:49, it had a local magnitude of 1.3, epicenter coordinates +49.025,+21.759 and a hypocenter depth of 3.2 km. The second was 11.10.2023 at 04:15:36, had a local magnitude of 1.5, epicenter coordinates +49.053,+21.791 and a hypocenter depth of 1.9 km. Eleven aftershocks were unlocalizable because they were so small that they were not recorded by a sufficient number of seismic stations.

Seismologists delivered the first data about the earthquake on express basis already an hour after the earthquake. Experts from the Faculty of Mathematics, Physics and Informatics of the Comenius University (FMFI UK) and the Institute of Earth Sciences of the Slovak Academy of Sciences (ÚVZ SAV) and other employees of the ÚVZ SAV have also been intensively working in the affected area since October 10.

However, as Peter Moczo, a seismologist at Comenius University in Bratislava, Faculty of Mathematics, Physics and Informatics, said at press conference, that seven

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Slovak seismologists, even full-time, do not have the chance to adequately document all the damage and manifestations of the earthquake.

They are helped by completed questionnaires from people who felt the earthquake in some way. "However, those most affected by the disaster have other concerns at the moment," Moczo noted.

Information about history of earthquakes in Slovakia is important part of education. Slovak and foreign students are informed about it also in the frame of subject English language by members of Department of Languages, FCE, STU in Bratislava.

2. Past earthquakes in Slovakia

Since 1034, a total of 804 earthquakes with macroseismic effects have been documented in Slovakia. The low number of recorded earthquakes is the result of very sparse and insufficient documentation until the period of the local government reform introduced by Maria Theresa (Maria Theresia Walburga Amalia Christina; 13 May 1717 – 29 November 1780).

The earthquake of June 5, 1443, with an epicenter in central Slovakia, destroyed **Banská Štiavnica** and seriously damaged **Kremnica**. Its intensity was 8 degrees EMS-98. The effects of this earthquake are still known only for the most important cities of the period, for example Vienna, Krakow and Brno. A recent archaeological survey also documented the destruction of a large part of the Ľupča castle.

The earthquake of November 16, 1613 near **Žilina**, which was supposed to cause damage as far away as in Bratislava, is little explored. The epicentral intensity could reach up to 8 degrees EMS-98.

Since 1643, 31 earthquakes have been documented in **western Slovakia**, whose macroseismic intensity in the epicentral area reached 5 to 8 degrees of the 12-degree European macroseismic scale (EMS-98).

The earthquake of June 28, 1763 with an epicentral intensity of 9 degrees EMS-98 destroyed seven churches and 279 houses in **Komárno** alone. The town hall was also heavily damaged. It is the largest documented earthquake with an epicenter on the territory of Slovakia. 63 residents died and 102 were injured. According to more recent research, there were at least twice as many dead. According to the latest serious estimate by Hungarian seismologists, the earthquake in Komárno could reach a magnitude of up to 6.3. Such an earthquake releases approximately 126 times more energy than a magnitude 4.9 earthquake. Twenty years later, on April 22, 1783, to be more precise, Komárno was hit by another devastating earthquake with an epicentral intensity of 7 to 8 degrees EMS-98. Approximately 500 houses were damaged, including the Komárno fortress.

In the 20th century, more than 300 earthquakes with macroseismic effects were detected in Slovakia.

On January 9, 1906, the largest earthquake with an epicenter on the territory of Slovakia in the 20th century occurred near the village of **Dobrá Voda in western Slovakia**. Its epicentral intensity was 8 degrees EMS-98. It is the first earthquake with an epicenter on the territory of Slovakia, for which an instrument record has also been preserved. The calculated magnitude of the earthquake is 5.7.

There were also four earthquakes with macroseismic effects **near Bratislava** (two in the **Modra-Pernek** source zone and two in the **Dobrá Voda** source zone). The strongest was on December 3, 1967 and had an epicentral intensity of 6 to 7 degrees EMS-98 and a local magnitude of 4.3.

However, historical earthquakes in the territory of Slovakia are still not sufficiently researched. Slovak seismologists have been unable to convince historians to cooperate with them for a long time, and it is impossible to study historical earthquakes without them.

3. Future earthquakes in Slovakia

According to experts, there is no single objective reason to believe that similar earthquakes will not occur in the future. "We have been emphasizing for several decades that even if an earthquake comparable to those that occurred in the past were to occur tomorrow, it would not contradict seismological knowledge and experience," said Peter Moczo.

Earthquakes in the future may occur mainly in one of the focal zones, in which earthquakes occurred in the past. Pernek, Dobrá Voda, Žilina, central Slovakia, Komárno, Vihorlatské vrchy (Vihorlat mountains) and eastern Slovakia are particularly at risk.

However, an earthquake in a place where it has not yet occurred is not ruled out. Slovakia is located inside the lithospheric plate.

4. Territorial extent and result of the earthquake on October 9, 2023

In this case, the cooperation of heads of district offices, mayors of municipalities and local residents is necessary for sufficient documentation.

Heads of district offices, mayors of municipalities and local residents cooperated very well with the seismologists. They deserve a big thank you in this situation.

ÚVZ SAS has so far received 2485 completed macroseismic questionnaires from 515 locations. This is the largest number of macroseismic reports in Slovakia.

Damage to buildings was reported in 196 questionnaires. Many houses and buildings are in a very serious condition. In the most affected municipalities, they had more worries after the earthquake than in other locations, and therefore it is likely that the number of questionnaires reporting damage to buildings is not representative. However, summary reports will already be sent from some municipalities based on an agreement between seismologists and mayors.

Main findings from macroseismic questionnaires:

Perception/sensation of movements: rocking, swaying, shaking, tremors, vibrating, twitching, moving or bending the wall in a horizontal direction, "swimming" of the house and things; passing something big around the house; crash into the house.

Perceiving/feeling sounds: whirlwind, rumbling, humming, banging, explosion. Feelings: lightheadedness, dizziness, feeling of falling off, balance problem; stress, fear, panic from the unknown. Feeling of duration: from a few seconds to 30 (seismologist's note: over 10 seconds it can be assumed that it was an oscillatory movement of the buildings).

Earthquake relation to nuclear power plant locations.

This earthquake could not in any way limit or threaten the operation of the Bohunice and Mochovce nuclear power plants. The power plants are secured for greater values of seismic movement than were recorded in the case of earthquake on October9. Due to the safety of the operation of nuclear power plants, an algorithm based on the sensing of seismic movement at the site of the power plant works in nuclear power plants, which enables early intervention in the operation of the power plant.

Earthquakes are continuously monitored and automatic earthquake localization is also continuous. However, there is no continuous seismological service. Interpretive supervision of automatic localizations is normally carried out in weekday mode. An extraordinary situation always requires the ad hoc intervention of seismologists outside of working hours. In the case of the 9/10/2023 earthquake, the seismologists of ÚVZ SAS managed to expertly interpret the available seismometric data already an hour after the earthquake.

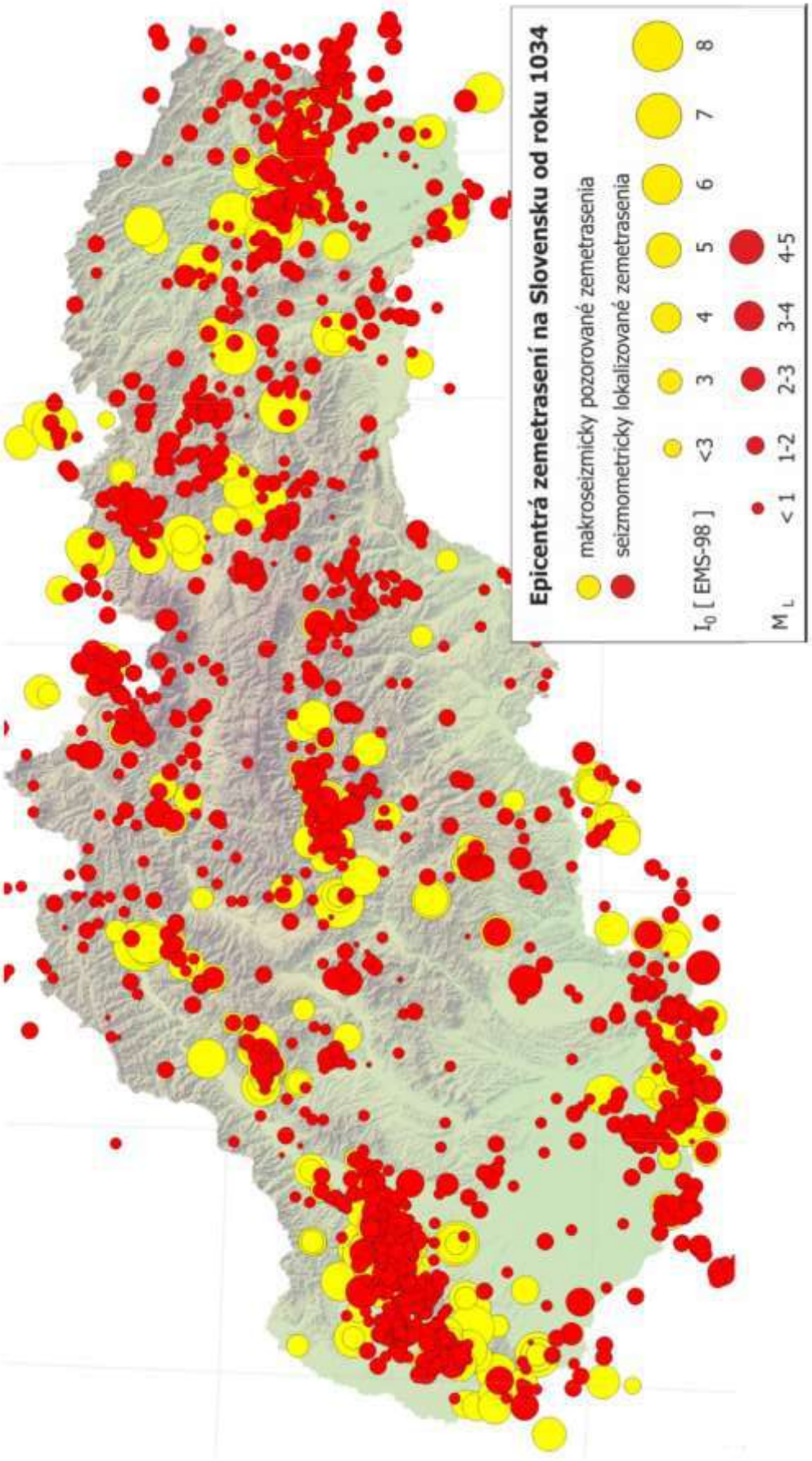


Fig. 1. Map of earthquake epicenters in Slovakia.
Source: Institute of Earth Sciences, SAS (SAV in Slovak).

5. Monitoring of earthquakes in Slovakia.

The National Seismic Station Network (NSSS) is built in such a way as to enable the localization and quantification of earthquakes with an epicenter on the territory of Slovakia, which have macroseismic effects on the territory of Slovakia. Fifteen stations: Kolonicé sedlo, Červenica, Stebnicka Huta, Kečovo, Skalnaté pleso, Izabela, Liptovská Anna, Vyhne, Zbyňov, Moča, Šrobárová, Iža, Hurbanovo, Modra, Železná studnička. Thirteen stations belong to ÚVZ SAS, two stations have devices provided by the Czech Republic.

The Local Seismic Network of Eastern Slovakia (LSSVS) belongs to the CU in Bratislava and is managed by FMFI CU. It was built in response to the earthquake on May 20, 2003, which had a magnitude of 4.2 and an epicentral intensity of 6 degrees EMS-98. Six stations: Podhoroď, Blatné Remety, Maškovce, Lesné, Veľká Trňa, Piskorovce.

The local seismic network in the vicinity of the Bohunice nuclear power plant belongs to Slovak Power Plants and consists of eleven stations. The local seismic network in the vicinity of the Mochovce nuclear power plant belongs to Slovak Power Plants and consists of eleven stations.

Local seismic networks in the vicinity of nuclear power plants make it possible to locate even microearthquakes and monitor the seismic activity of a small area around the power plants, while the Dobrá Voda focal zone is also partially covered.

Slovak seismologists do not have the equipment to monitor other focus areas. Even if there was an increase in nanoseismic and microseismic activity – a possible indication of zone activation – it is not possible to capture this increased activity. Slovak seismologists do not have a sufficient number of portable seismometers to adequately cover any shaken area. All over the world, after an earthquake, a network of seismometers is installed in the epicentral area. Every piece of data is important as an input to the seismic risk analysis.

6. What should be done in Slovakia

Start sufficiently financially ensuring earthquake monitoring. With sufficient funds, update the NSSS and LSSVS to the level of the European standard. Purchase seismometers for focal zone research. Start the process of revising the seismic hazard analysis of Slovakia in accordance with the European Union Eurocode 8 standard in cooperation with the CO and the relevant ministries. The last update of the seismic hazard map is more than 10 years old.

After calculating the hazard map, proceed with the regionalization of the territory of the Slovak Republic for the needs of planning, design and construction and possible anti-seismic measures.

It is necessary to examine historical earthquakes in cooperation with historians so that the data on historical earthquakes can be used in the calculation of the seismic hazard of the territory of Slovakia, especially the locations of nuclear power plants.

It is necessary to have people for all this. If trained PhD experts continue to leave seismology and geophysics because of salaries, it will not be possible to implement the morally necessary care of the state for the seismic safety of the people in the state.

The NSSS modernization project was completed in 2003. The LSSVS was built in 2007. Since then, investment funds have been missing for a long time.

Both NSSS and LSSVS seismic stations need a one-time injection of investment funds in order to function properly. It is necessary to introduce sufficient funding for NSSS and LSSVS. Otherwise, their degradation and degree of non-functionality will continue. Not to mention lagging behind the European standard.

FMFI UK and ÚVZ SAS cooperate very closely and on a daily basis. Together, they ensure the operation of NSSS and LSSVS, data collection and interpretation, analysis of

the seismic hazard of the territory of Slovakia and the locations of nuclear power plants, teaching of seismology and geophysics, basic and applied seismological research.

Over the last 14 years, 6 completed doctoral students and several masters of geophysics/earth physics left for financial reasons to various companies and companies where they were offered higher salaries. The other three work abroad.

If seismologists could predict the time, size, and location of the next earthquake's hypocenter at least 24 hours in advance, it would be possible to evacuate residents, halt critical operations, and shut down utilities—completely or at least partially. Shorter-ahead forecasting would allow people to get out of buildings and gather in places that would not be threatened by falling buildings and objects.

Today's seismology cannot predict the time, size and location of a future earthquake. This is mainly due to the inaccessibility of the interior of the Earth in the depths where earthquakes are being prepared due to the high financial demands of adequate research.

Whether the forecast is possible or not, after the earthquake, people should return to residential houses and buildings, power plants and factories should continue to operate. This means that houses, buildings, factories and power plants should not be disabled by an earthquake and there should not be any ecological disaster. Therefore, the main task of seismologists is to sufficiently accurately estimate the maximum values of important characteristics of seismic movement at a given location during future earthquakes. Designers should design buildings accordingly, and builders should build them honestly according to the projects.

In order for seismologists to be able to estimate the seismic movement during future earthquakes as accurately as possible, they need to know everything possible and adequate about the given territory and focal zones. And this is not possible without monitoring earthquakes, researching focal zones and regionalizing the territory according to the degree of seismic risk.

7. Mansions, churches, temples and houses after earthquake in 9.10.2023



Fig. 2 The earthquake hit the east of Slovakia near Humenné. See also Figs. 22 and 23

The earthquake damaged 27 churches and temples, 2 mansions (all are listed buildings) and several family houses. It is supposed that 2 churches will be demolished.

One of the most affected is the village of Ďapalovce, where the epicenter of the evening earthquake with a magnitude of 4.9 was located. In Ďapalovce, 44 residents were evacuated from 15 houses. They also declared an emergency situation in the village. Due to damage to houses during the earthquake, a state of emergency was declared in two other villages. These are Holčíkovce in the district of Vranov nad Topľou and Kolbovce in the district of Stropkov.



Fig. 3 Ďapalovce (Source: TASR)



Fig. 4 Ďapalovce (Source: TASR)



Fig. 5 Ďapalovce (Source: TASR)



Fig. 5 Ďapalovce (Source: TASR)



Fig. 6 Pakostov (Source: TASR)



Fig. 7 Pakostov(Source: TASR)



Fig. 8 Pakostov (Source: TASR)



Fig. 9 Baškovce (Source: TASR)



Fig. 10 Baškovce (Source: TASR)



Fig. 11 Baškovce (Source: TASR)



Fig. 12 Baškovce (Source: TASR)



Fig. 13 Baškovce (Source: TASR)



Fig. 14 Baškovce (Source: TASR)



Fig. 15 Jankovce (Source: TASR)



Fig. 16 Jankovce (Source: TASR)



Fig. 17 Nižná Sitnica (Source: TASR)



Fig. 18 Nižná Sitnica (Source: TASR)



Fig. 19 Nižná Sitnica (Source: TASR)



Fig. 20 Holčíkovce (Source: TASR)



Fig. 21 Holčíkovce (Source: TASR)

The earthquake damaged almost all the houses in Ďapalovce to a greater or lesser extent, with the exception of three or four. Statics went through the entire village, they were repeatedly in problem houses.

According to the processed static assessments, eight houses and one detached garage are slated for demolition. Seven houses should be partially demolished, either the floor, or the rear part or the roof. Reconstruction work is underway in five houses due to cracked walls. Of the houses condemned for demolition, at least three were permanently inhabited.

In Ďapalovce, approximately half of the houses are insured, the rest are not. Many people are disappointed, confused and angry. For many years, they lived in the assumption that they had the house insured and paid the policies, and now they received a decision that the payment will not take place, beacuse earthquake damegaes are not covered. Only a tiny percentage of people received money from the insurance companies.

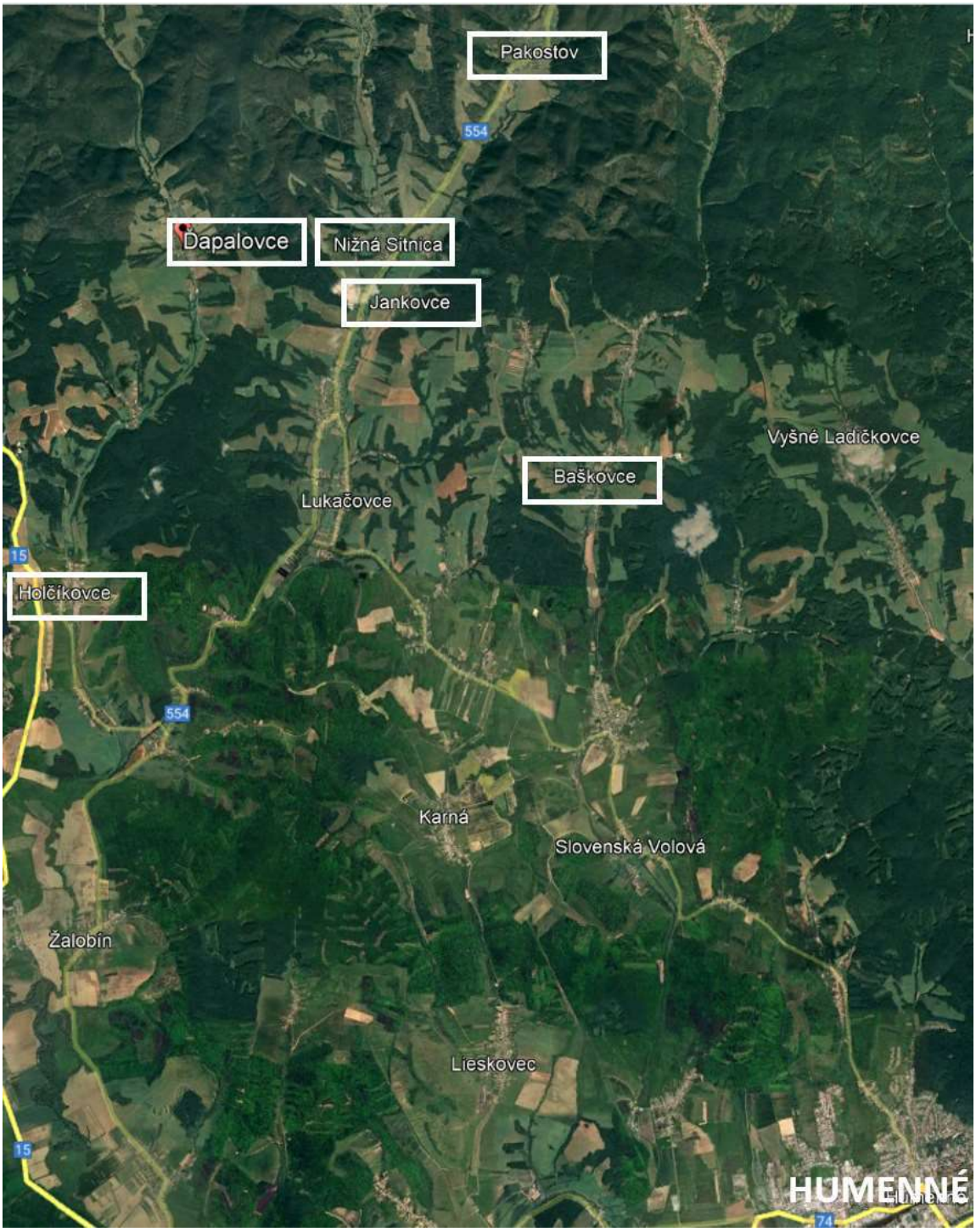


Fig. 22 Location of villages Ďapalovce, Nižná Sitnica, Jankovce Pakostov, Baškovce and Holčíkovce near Humenné city. (Source: Google Earth).

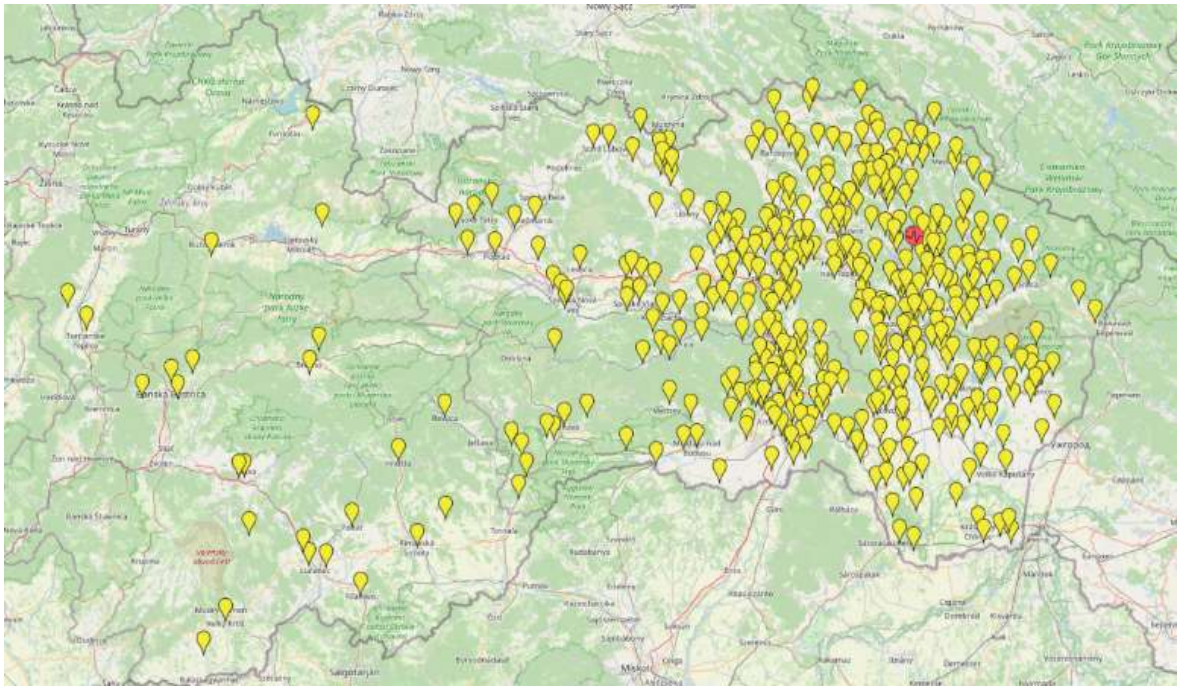


Fig. 23 Overview of locations with observation of earthquake effects based on macroseismic questionnaires received by 11/10/2023 8:08 a.m. (1824 questionnaires from 447 locations). (Source: the Institute of Earth Sciences of the Slovak Academy of Sciences, Ústav vied o Zemi Slovenskej Akadémie vied, in Slovak)

Conclusion

Information about earthquake of October 9, 2023 with an epicenter in the cadastre of the village of Ďapalovce, district Vranov nad Topľou, in east of Slovakia is presented in details, together with tasks and problems of Slovak seismologists.

Information about actual events together with explanation of Slovak experts are very interesting for regular and PhD students as well. Moreover, our foreign students in this way learn more about the problems, successes and failures of Slovakia and about its geography.

Together with appropriate terminology of civil engineering and architecture [2] such information is very important part in the education of structural engineers. This fact was clearly confirmed by experience of members of Department of Languages at Faculty of Civil Engineering, Slovak University of Technology in Bratislava, Slovakia.

Acknowledgement

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ABOUT THE FATHER OF THE CZECHOSLOVAK SCHOOL OF ROPE ROOFS ENGINEER JOZEF POŠTULKA

Ján Kyseľ[1]

Founder and President of Society of Structural Designers of Slovakia,

Abstract: *Jozef Poštulka's life story is full of extraordinary moments in which moments of exceptional appreciation alternated with painful moments. His engineering intuition, sense for capturing the behavior of the structure, courage, uncompromisingness and directness in pursuing his designs made him a personality that stood out among civil engineers, especially structural engineers. He was very popular with many of his peers, but also with considerably younger colleagues, because everywhere he tried to give away not only professional experience but also good mood.*

Key words*: parabolic arches, rope roofs, inventions, wood-concrete beam*

1. Introduction

1.1. Early life

After his studies at Professor Kurt Klöppel in Darmstadt (together with Jiří (Juraj) Kozák, later Ing. Dr. CSc. in Bratislava and Josef Říha, later Prof. Ing. DrSc. in Brno), Jozef Poštulka completed his studies in Bratislava at the Faculty of Civil Engineering of SVŠT in 1950. His classmates were: top designers Ing. Behúl Vojtech, CS., Ing. Bustin Ján, CSc. and renowned scientists of the Slovak Academy of Sciences Ing. Gustáv Martinček, DrSc., and Ing. Rudolf Skrúcaný, DrSc., from ÚSTRACH SAV in Bratislava. From that year he worked as a designer of load-bearing structures in Stavoprojekt in Bratislava – first in the Design Centre for the Design of Industrial Buildings and worked his way up to the position of a top expert and became a household name in Stavoprojekt (Fig.1).



Fig. 1. Jozef Poštulka (6.1.1923-5.3.2009)

1.2. Rope Roofs Design

Jozef Poštulka is generally considered the father of the Czechoslovak school of rope roofs. He was a true pioneer and implemented more than 110 of them – at home and abroad. The most important is the roof of the Sports Hall in Pasienky in Bratislava – often called the "Poštulkova hall"(Fig.2 and Fig.3).

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Fig. 2. Hall in Bratislava



Fig. 3. "Poštulka`s hall".

The mass-spatial scheme of the hall is defined clearly and definitely. This fact ranks this undoubtedly attractive architecture among the better realizations that were carried out in Slovakia in the 20th century. The great advantage is that the concept of the hall is clearly legible. It is obvious from the first glance what kind of object, with what volume and function it is. Nothing is masked or covered.

Since in this type of construction the shape is primarily determined by the static action of the structure and not vice versa, we can also rank it among the top works of Slovak civil engineering of the 20th century. Two compositional axes are perceptible here. Main and minor. The course of the main axis is defined by the center of the playing field, which it divides into two equal parts. The Paraboleum hall is designed more elegantly - two parabolic arches intersect above the center plane in the shape of a reclining letter X.

The dominant load-bearing structure of the hall consists of two expressively unambiguous and aesthetically appealing parabolic arches with a span on which the progressive rope structure of the roof shell in the shape of a hyperbolic paraboloid is suspended. This whole system, which is still unique in Slovakia, is stored on a subtle and elegant reinforced concrete skeleton, which is filled with glazing and stretches along the entire perimeter of the building.

1.3. Other structures

He also actively participated in the design and construction of the suspension roofing of the Olympic Stadium in Munich – he was physically present during the project work. The design of the stadium and the entire complex is by a German couple, architect Günter Behnisch, designer and structural engineer Frei Otto. The project of forty-five-year-old Behnisch became a symbol of the transformation processes of West Germany in the 60s years of the 20th century, which, in addition to jury chairman Egon Eiermann, was supported by several influential politicians(Fig.4).



Fig. 4. Olympic Stadium in Munich

The project was timeless for its time and was soon associated not only with Munich, but with the whole of West Germany. The concept of large tent roof structures, consisting of plexiglass and steel ropes, covered not only the stadium auditorium, but also passed over adjacent buildings and access roads.

It was the first time such a structure had been carried out on such a large scale, albeit on a smaller scale, in 1967 by Rolf Gutbrod at Expo '67 in Montreal. However, authorship cannot be denied precisely to Frei Otto, whose tent structures were the pride of Germany at that time of engineering feats in architecture.

The whole complex is connected by a steel-perspex roof, above which rises a 290 m high TV tower. The roof structure itself consists of 410 km of steel ropes. The net is filled with 7 centimeters thick plexiglass plates that protect spectators from rain and wind, while not changing the spectrum of penetrating sun rays. This allowed even color television to achieve an image with a balance of light and shadow(Fig.5).



Fig. 5. Roof structure of Olympic Stadium in Munich

1.4. Patents

Poštulka was the author of seven inventions, which were widely applied and were also mentioned in practice by the author's name. The most famous is the invention of the circular rope roof – implemented on dozens of buildings at home and abroad. Furthermore, it is a method of strengthening historical wooden ceilings, applied to many buildings at home and abroad and its type series of lightweight steel trusses – called "Poštulkove beams". He pioneered the design of composite wood-concrete ceiling structures.

1.5. Another Practical interests

Ing. Poštulka did not have only rope structures in his activity. He was a versatile designer and designed structures made of steel and concrete. Of the steel structures, the most important are the production hall in Snina, a large garage for 600 cars in Bratislava and Žilina. His circular garages in Bratislava and Ostrava are also known(Fig.6).

Concrete structures are mainly residential buildings – in total more than 1,000 apartments. Important are the tower panel houses near the tunnel in Bratislava, in Karlova Ves, the experimental tower house in Krasňany, on Miletičova Street, in Partizánske.



Fig. 6. Large garage for 600 cars

Jozef Poštulka's work is remarkable for a large number of projects and realizations (representing more than a thousand) from various fields. He was always able to choose the optimal solution from various structural-material options and solve buildings comprehensively – whether the part is steel, wood or concrete, lower or upper structure. He had the courage to solve structures innovatively with a natural preoccupation with economy, namely in new buildings, breakdowns or reconstructions of building objects.The construction of a music pavilion in the spa park in Piešťany with an 11 m cantilevered roof from the end of the 70s of the 20th century is originally designed

The Church of Christ the King and St. Andrew in Koš In the 90s of the last century, due to the coal seams under the church and the danger arising from the undermined terrain and disturbed statics, its demolition began to be considered. In the end, the preservationists managed to promote at least the preservation of the Gothic presbytery, which, however, could not remain in its original place.

Therefore, in December 2000, a unique transfer of this almost 400-tonne shrine took place in Slovakia. After separating from the church, it was transported by road about two kilometers further to the newly built church. The remaining part of the church - a building from 1940, together with the original tower, was blown up in March 2001. in 2000, when a Gothic sanctuary from 1410 was separated from it and transported in its entirety several kilometers to the center of the village to the newly built church(Fig. 7).



Fig. 7. The Church of Christ the King and St. Andrew in Koš

2. Summary of the famous rope structures designed by Jozef Poštulka

With its characteristic structures, mainly rope structures, it penetrated far abroad, and very far. Its structures can be seen in Pyongyang (Korea), the roof of the power plant in Nanjing (China), the roofing of the hall in Kabul (Afghanistan), the roofing of the theatre in Győr (Hungary), the exhibition hall in Welsi (Austria), also the circular exhibition hall in Augsburg (Germany) in 1981 and later in Békéš Čaba (Hungary)(Fig. 8).



Fig. 8. Rope structures in abroad

3. Conclusion

Poštulka worked at Stavoprojekt from 1950 to 1966. In the relaxed times, he was the head of the Expert project cooperative and the SVTS Expert Corps. But his initiative did not pay off. At the time of normalization, he was charged with illegal business and was held in custody from 1971 to 1974. But even this time he was a useful designer for the Interior.

After his release, he worked at the Project Institute of Culture in Bratislava. Many constructions are realized from this period (theatre in Nitra, cooperation in reconstructions and preservation of monuments, such as Trenčín Castle, Bojnice Castle, Červený kameň). Even in his old age, he was active and active, and his enthusiasm was often transferred to younger colleagues.

He lectured on the fruits of his extraordinary activities not only at home but also at congresses in Paris, London, Leningrad, Almaty, Mexico, Madrid, Essen, Stuttgart, Dresden and published more than 50 professional articles in journals and anthologies. He was a member of international organizations IASS and AICP.

The Jozef Poštulka Award, awarded by the Association of Structural Engineers of Slovakia since 2010, is intended for personalities in the field of load-bearing structures of buildings who significantly participated in original solutions of load-bearing structures and contributed to the development of information about the statics of buildings and their bearers - structural engineers.

Engineer Jozef Poštulka died on 5.3.2009 at the age of 86.



Fig. 9. Jozef Poštulka and his wife



Fig. 10. Jozef Poštulka, as 85 year old man (2008)

Acknowledgements

I would like to thank to Prof. Doncho Partov for his idea to commemorate jubileum of 100 years of Ing. Poštulka birthday and for his writing this contribution in English Language based on [1] and many other documents and publications.

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ОТНОСНО ПРЕМЕСТВАНЕТО НА ГОТИЧЕСКА ЦЪРКВА ОТ XVII СТОЛЕТИЕ В ГР. КОШ-СЛОВАКИЯ, ИЗПЪЛНЕНО ПО ПРОЕКТ НА ЙОЗЕФ ПОШТУЛКА (1923-2009)

Йозеф Поштулка[1]

Abstract*: In this paper the author described some problems, connected with: static analysis constructive solutions and technology operations, using by translation of gotic church from 1630 in Kosh – Slovakia*

Key words: *static analysis, constructive solutions, technology operations, translation, gotic church, concrete fundament*

1. Общи данни

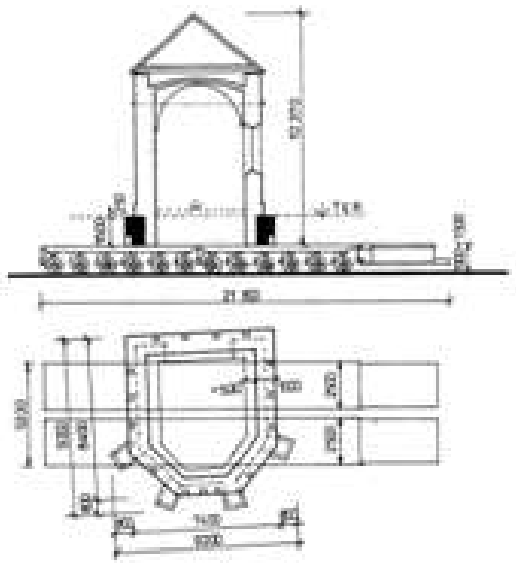
Град Кош в Словакия лежи върху 5милиона тона кафяви въглища, които миннодобивната компания "Новаки" възнамеряваше да започне да разработва през 2001г. Според закона за минното дело обаче там, където се добиват въглища под земята, не може да съществува строителен обект, тъй като рано или късно той може да пропадне заедно с хората в минния басейн. Затова се наложи да се изгради ново селище на 1,5km. източно от старата община. В новото селище беше построена и нова църква, като старата църква в Кош трябваше да бъде съборена. Тази "стара" църква всъщност бе построена през 1941 год. В нея обаче бе вградена наистина една стара готическа църква и то така,че олтарът и остана всъщност като един страничен параклис, изпълнен със сводест таван и дървена покривна конструкция(Фиг.1). Този ранноготически олтар от 1630 год. представляваше един безценен паметник на културата, който трябваше да бъде преместен в новото селище, за да бъде запазен за следващите поколения. Финансирането трябваше да бъде извършено от миннодобивното предприятие. Възможността, обектът да бъде разделен на части, от които след това отново да бъде изграден беше отхвърлена. Олтарната част бе отделена от църквата чрез нейното предварително разчленяване на два части, като по-новата част от 1941 год. бе взривена през април 2001 год. Строителните работи бяха извършени от местна фирма.

[1] Йозеф Поштулка, *EXPERTA, Инженерна и консултантска дейност в строителството, Братислава, Словакия,*

Службата за защита на паметници на културата в Банска Бистрица поръча през 1998г на автора на тази статия, статическото изследване на конструкцията на църквата в процеса на всичките й етапи на преместването. Авторът имаше вече опит в решаването на подобни проблеми от 1983 год, получен при транспортирането на един малък параклис на 400 m от неговото местонахождение, и от 1993 год. при транспортирането на две стоманени трибуни и триетажен скелет на сграда с огромен автокран по конната писта на Братислава на разстояние над 300 m.

2. Статическо изследване и конструктивно укрепване на конструкцията преди повдигането.

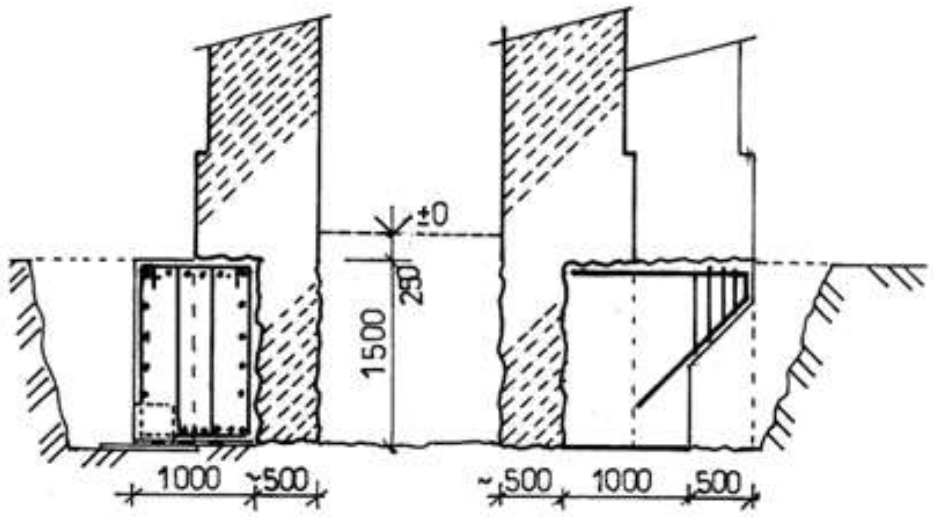
При църквата от Кош ставаше въпрос за обект от каменна зидария, т.е. изпълнена от недялани камъни, свързани с хоросан. Конструкцията с дебелина на зидарията 1 m. е покрита с готически кръстат свод, изпълнени с каменни сводести дъги, над които е разположена дървена покривна ферма, покрита с керемиди.
Ивичните фундаменти на църквата са от същия материал като външните стени и лежат 1,5 m под терена(Фиг.2).



Фиг.2. План и разрез на олтарната конструкция в момента на транспортирането

Подът на конструкцията лежи на +0,25 m. над терена. За да не се разруши обекта при повдигането и транспортирането му, беше необходимо да бъдат изпълнени здрави ивични фундаменти, които да подпират сградата и поемат всички външни въздействия. Този ивичен фундамент трябваше да бъде изчислен за най-неблагоприятната комбинация от натоварванията, възникващи при повдигането на обекта с помощта на 16 хидравлични крика, разположени на различни места в основната схема на сградата.
В случая бетонирането не можеше да се изпълни на части, както това обикновено се прави при укрепване на фундаменти, а трябваше да се изпълни непрекъснато. Проблемът бе така решен, че новата ивица-фундамент, широка 1 m. се бетонира в стария фундамент на обекта, след разрушаването на половината от стария ивичен фундамент под външната стена. Обектът остана да стои

безпроблемно върху другата половина на съществуващия фундамент и издържа през цялото време на непрекъснатото бетониране на новата фундаментна ивица, която излизаше на около 0,5 m. навън от цокъла на сградата. На новото място, този фундамент беше предвиден да служи за пътека около църквата(Фиг.3а). Преди бетонирането на фундаментните ивици, върху терена бе положено фолио, което трябваше да улесни отлепването на фундамента при неговото повдигане. Колоните на църквата бяха подпрени отделно с помощта на стоманобетонни конзоли, които излизаха пред фундаментната конструкция(Фиг.3б).



Фиг. 3а. Усилване на каменния фундамент на църквата със стоманобетонен
Фиг. 3б. Конзолен стоманобетонен фундамент, върху който ще стъпи колона на църквата

За осигуряване устойчивостта на каменната зидария на църквата срещу разпадане при трениспорта, същата бе осигурена и с един обръч от кръгла стомана, със сечение ∅20мм, монтиран на нивото на стъпването на свода върху зидарията. Двете конструктивни усилвания изпълниха много добре своята роля, не само при повдигането, но и при транспортирането и разтоварването на обекта, който не показа никакви деформации в периода на преместването му. Не се появиха нови пукнатини в стените, а дори и остатъците от стария фундамент не паднаха, въпреки че това се очакваше.

3. Технология на повдигане на сградата и нейното поставянето върху транспортната платформа

Първоначално бе предвидено сградата да се повдигне с кран, както се преместиха трибуните в Братислава. Поради това на определени места в ивичния фундамент трябваше да се закрепят четири монтажни халки.
Но при наличието на голямо собствено тегло на църквата, което възлизаше на 400 тона, изпълнението на тези монтажни халки и тяхното оразмеряване бе много

трудна задача.. Една холандска фирма изрази готовност да пренесе църквата с помощта на огромен кран, който трябваше да бъде докаран от Холандия, което по наша преценка беше твърде сложно и скъпо.
Две фирми от Острава-Чехия, които имаха опит в транспортирането на големи товари, получиха поръчката.Тези фирми предложиха да повдигнат църквата с хидравлични крикове до 1,35м. височина, така че отдолу под фундаментите да могат да се вмъкнат две свързани транспортни платформи, които да закарат обекта по улиците до отдалечената на 1700 m цел. Хидравличните крикове бяха оставени в общо 15 малки ниши в петата на фундаментния пояс(Фиг.4).



Фиг.4. Повдигане на църквата с 16 хидравлични крика

Под тези ниши още преди бетонирането на ивичния фундамент върху земята бяха положени стоманени плочи 800x1000x20 mm, за да не потънат криковете в земята. Товароподемността на криковете възлизаше на 50 t. Тъй като църквата трябваше да бъде повдигана неравномерно в различни точки от хоризонталната проекция, конструкцията на ивичния фундамент трябваше да бъде изследвана като, статически неопределима система и оразмерена да поеме раничните състояния на якост и деформации. Начинът на разположение на обекта върху транспортните платформи е показана на (фиг.5).



Фиг.5. Транспортиране на обекта

4. Начин на транспортиране на сградата на църквата.

Трасето определено за транспортиране на църквата до новото място беше невъзможно за преодоляване с нормални транспортни средства. Затова в Италия бяха поръчани специални транспортни платформи на фирмата “Фаджиоли”. Те се транспортираха до местостроежа разглобени на 6 оси, с помощта на товарен камион. На определеното от нас място, те се сглобиха, за да се получи необходимата товароподемност. Отделните части бяха 4, всяка от които с 6 оси, т.е. с 12 колела или общо 48 колела(фиг.5).



Фиг.6. Разтоварване на църквата на новото и местоположение

Свързаното транспортно устройство притежаваше в единия си край дизелов двигател, който задвижваше хидравличния двигател на отделните колела, така че всяко колело да може да се движи в различни посоки до 90°. Шофьорът на композицията не седеше в превозното средство, а беше извън него и обслужваше цялата операция по транспорта с компютър. Товароподемността на транспортното устройство възлизаше според данните на фирмата на 750 тона, докато теглото на църквата беше около 400тона.
Транспортирането бе извършено безпроблемно. Положението на църквата бе контролирана непрекъснато. Тя премина пътя от 1700 m без допълнителни усложнения. Тъй като сградата върху платформата беше с ширина 9,20 m., трябваше да бъдат отсечени някои клони от страничните дървета, а заради височината от 14 m. да се демонтират всички електрически проводници по нейния път(Фиг.6).

5. Изграждане на новото място

Тъй като сградата имаше собствен ивичен фундамент, възникна въпросът как да се изпълни нулевия цикъл на новото място. Долните ръбове на новия и съществуващия фундамента по ред технологични причини не лежаха в една равнина. На новото място трябваше да бъде подсилена повърхноста, за да не потъне транспортното устройство, заедно с целия си товар, т.е. 110+400=510т.
Затова на новото място не бяха изградени нови ивични фундаменти, а повърхността на терена бе укрепена с фундаментна стоманобетонна плоча, дебела 20см, армирана с мрежи(Фиг.6). Слабо армираната стоманобетонна плоча се приспособи към неравната повърхност на фундамента на църквата, като поради незначителната си коравина спрямо огромната коравина на църквата и нейния фундамент, в плочата се появиха очаквани пукнатини, които предизвикаха в непросветените в тази област наблюдатели необосновани реакции. Долният ръб на сградата на новото място лежи 1,30 m. над терена. За да не замръзне през зимата, около църквата бе направен наклонен насип и по този начин фундаментната плоча остана под нивото на терена.

6. Заключение

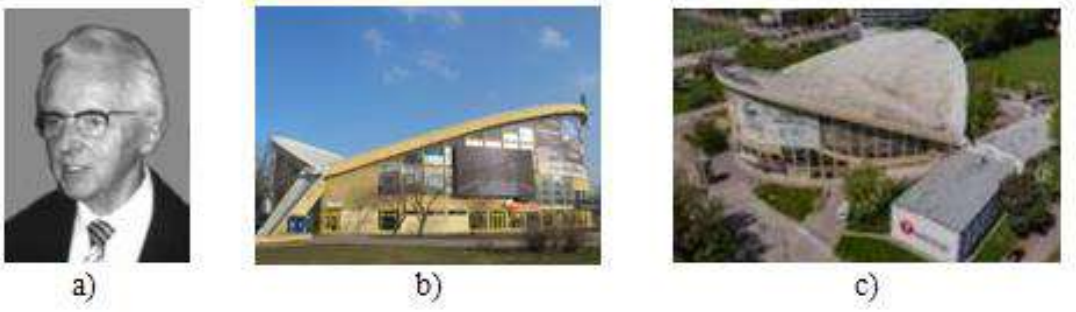
Целият транспорт заедно с натоварването и разтоварването продължи стъпка по стъпка около 3 часа. След това транспортното устройство бе демонтирано, а отделните му части бяха натоварени с помощта на кран на товарен камион и бяха върнати още същият ден на 06.12.2000г. в Германия.
Църквата представлява един ценен ранноготически паметник на културата, който чрез описаният технологичен процес(***от статическите изчисления до технологично-монтажните операции***) беше съхранен за следващите поколения. Главната заслуга за това имаше службата за защита паметниците на културата в Банска Бистрица под ръководстото на г-н арх.Сура и неговите сътрудници.
Изказвам моята благодарност на научния комитет на конференция на БАНИ2023 за приемането на този доклад и се надявам той да послужи за нови идеи при опазването на подобни исторически паметници, с които България е изключително богата.

7. Послеслов

(*Тази част на доклада е написана от членът на организационния комитет на научната конфренция БАНИ2023: проф. д-р инж. Дончо Партов, който имаше прекрасната възможност да поддържа близки контакти с този неповторим, словашки колега, раздаващ безрезервно и щедро своите ценни знания, в областта на въжените покривни конструкции).*

Докладът е посветен на 100 годишнината от рождението на големия словашки строителен инженер Йозев Поштулка, които бе голям специалист и в областта на висящите покривни конструкции, които бяха негов патент и специалитет.
Над сто уникални разнообразни по вид и фукция обществени и граждански обекти с леки въжени покривни конструкции украсяват до ден днешен, строителната палитра на красива Словакия, няколко от които са показани по долу.
Няколко от тях са изпълнени в Германия, Южна Корея и Аганистан.

Инж. Йозев Поштулка през 1950 г. е един от първите дипломанти на Строителния факултет на STU в Братислава. Негови съученици бяха: изветните словашки инженери: Behúl Vojtech, Bustin Ján и известни учени от Словашката академия на науките инж. Густав Мартинчек, д-р и инж. Рудолф Скручани, д-р от ÚSTRACH SAV в Братислава. инж. Poštulka проектира няколко въжени покрива (фиг. 29). Най-известните му произведения включват напр. спортна зала в Pasienky в Братислава (Фиг. 7) и няколко други известни спортни комплакса с плувни басейни, с въжени покривни конструкции(фиг.8.) Проектира няколко сгради и за екзотични страни като Китай, Афганистан и Южна Корея.



Фиг. 7. a) Ing. Йозеф Пощулка (1923 – 2009); b) Спортна зала на Pasienky в Братислава, проект от 1958 г.: Ing. Poštulka, архитектура Ing. арх. Йозеф Хованек, открита през 1962 г.; c) изглед от горе, залата има формата на липов лист и е национален паметник на културата



Фиг. 8. Няколко въжени покривни конструкции над сгради с плувен басейн

Той работи в процеса на проектирането и удари своя печат върху конструкциите и на Олимпийския стадион в Мюнхен през 1958 год. Той е автор на седем изобретения. Когато негов колега го обвинява в незаконен бизнес по време на нормализирането на политическата ситуация през 1968 г., той излежава четири години затвор. Там той получава след разрешение от съдебните власти, необходимите за целите на строителното проектиране съответните чертожни помагала, чрез които да упражнява своята професия, на проектант по време на престоя си в затвора.

Инж. Пощулка посещава България за първи път през 1986 г., когато посещава град Велико Търново, по повод конференция по стоманени конструкции, организирана от НТС София. Там бяхме свидетели на уникалната му среща с доц. д-р. проф. д-р инж. Борислав Банков, един от големите български мостови строители. На тази среща научихме, че инж. Пощулка има диплома по строително инженерство от Политехниката в Дармщад, където е преподавал, българския инженер: д-р Б. Банков в областта на стоманените мостове, докато е бил докторант по стоманени конструкции при професор Курт Кльопел, в периода (1939-1944). Този контакт дава повод, на инж. Поштулка да приеме колегите на д-р Б. Банков, като много близки за него хора.

От тези години той е редовен гост в нашите инженерни среди като участник в конференции в УАСГ и ВСУ „Любен Каравелов". В знак на уважение към нашите учебни заведения той ни запознава най-безкористно, с идеите на прочутите си патенти за окачени покриви, изпълнени с въжени конструкции. В бивша Чехословакия се говореше, че той е оправдал средствата, които държавата е отделила за образованието на своето поколение, тъй като сградите му са правени с разход на стомана не повече от 15 kg/m^2, докато конструкции, изпълнени с прътови или пълностенни греди са сравнително по тежки, при които теглото обикновено е не, по-малко от 50 kg/m^2.

MONOŠTOR BRIDGE – NEW ROAD BRIDGE OVER DANUBE RIVER IN KOMÁRNO

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Abstract: *The existing 131, 128 and 80 years old bridges on the 150 km long section of the border river Danube between the Slovak Republic and Hungary do not meet today's requirements. After sixteen years of effort, the new road Monoštor Bridge was open on September 17, 2020. The single tower cable-stayed bridge has two fan-shaped cable planes. The bridge structure has uniquely transversely inclined L-shaped pylon.*

Key words*: cable-stayed bridge,*

1. Introduction

Elizabeth Bridge over Danube River in Komárno was opened on September 1, 1892. The technical parameters of the more than 130-year-old border bridge do not meet the requirements for bridges in the third millennium. The load capacity of the bridge built at the end of the 19th century in 16 months (!) is very low (20 t) and the width of the bridge is only 5.6 m (Fig.1). The construction of a new bridge therefore became a necessity.

The name of the bridge accepted the suggestion of the Hungarian Commission for Geographical Names: "The road connection to the bridge passes, on the right bank of the Danube, along Fort Monostor, which is the most important part of the historical fortress system in Komárom/Komárno, nominated to the UNESCO List of World Heritage. The name thus would refer to the geographical location (Koppánymonostor District, where the fort is situated) and the fortress system which has been significant in the common history of the two banks of the Danube since the Roman fortifications in the area." The name of the bridge in the Slovak language is: Monoštor Most, in the Hungarian language: Monostori Híd.

The preparation for the construction of the new bridge started already in 2004-2005 with a feasibility study for the solution of the new Danube Bridge. The selected concept and architectural solution given in the study of proposed variants was realized after sixteen years of effort. The survey work and the process of the impact bridge structure on the environment were carried out in 2006-2007. The documentation of the intention of building plan and the documentation for the zoning decision were drawn up and approved.

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The project for a building permission and the realization project were developed in 2013-2015. After the contractor was selected, construction began in 2017.

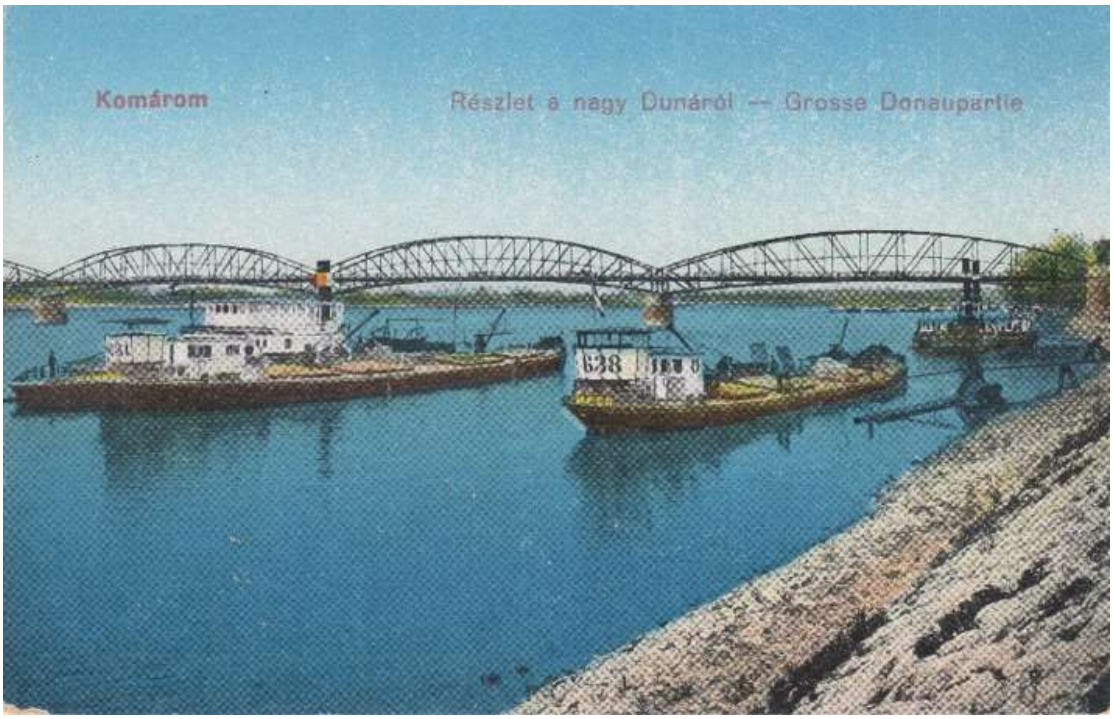


Fig. 1. Elizabeth Bridge over Danube River in Komárno was built in 1891-92. It was reconstructed after two of its spans were destroyed in World War II. Postcard was sold before 1919. (Courtesy of Ivan Baláž).

2. Integration of the bridge structure into the environment

The bridge is located 170 m far from the existing railway bridge (Fig. 2, upstream towards Bratislava) and 2.8 km far from the Elizabeth Bridge over the Danube River located in the center of Komárno. The 601.2 m long Monoštor Bridge between the cities of Komárom and Komárno is part of a new road of category C 11.5/80 with a length of 2.599 km, which connects main roads No. 1 in Hungary and No. 63 in the Slovak Republic. The bridge object supports two-lane main road. The width of communication between barriers is 11.50 m. There is a 1.80 m wide pedestrian path on the upstream side of the bridge and a 2.50 m wide bicycle path on the downstream side.

This new road in Hungary is an extension of the main road No. 13, which leads to the M1 highway. Among the urgent future tasks is the completion of the Komárno bypass by extending this road to the main road No. 64 Nové Zámky – Komárno. At the station of this road at km 1.545 22 above the Danube River at river km 1770.6 is the state border between the Slovak Republic and Hungary.

As part of the project, three other smaller bridges were realized in Hungary and one footbridge for pedestrians and one for cyclists in the Slovak Republic, which connect the bridge with the cycling route Eurovelo 6 on the left-bank of the flood dam of the Danube River.

3. Architectural and urban solution

The local geographical conditions influenced the design of the architectural solution of the transversely inclined single-pylon with two fan-shaped cable planes supporting bridge structure (Fig. 2). The following facts were determining: the approx. 15 m higher bank on the right (Hungarian) side as well as the terrain behind the approx. 8 m high flood barrier on the Slovak side; the requirements: a) of the navigation authorities (190 m wide and 10 m high waterway located significantly closer to the right bank of the river and b) of

the river flow manager (protection zone of the dam, bridging of the seepage channel and clearance profile on the dike for the passage of mechanisms during floods).

The pylon, as a basic element of the designed solution, meets the current static and aesthetic requirements and is the dominant feature of the entire system. Its shaping has become a prestigious issue of the entire architectural solution.

Several bridges became famous in the world having the single tower of the cable-stayed bridges (e.g.: the Alamillo Bridge with an asymmetrical harp system in Seville, Andalucia, Spain; the bridge over the Elbe River in Ústí nad Labem under Mariánská Skála, Czech Republic). The pylon structure is placed eccentrically with respect to the axis of the bridge on the upstream side of the middle pillar and leans over the road. The unique system of the cable-stayed bridge with a single *L*-shaped pylon inclined in transverse direction with two cable planes ranks this bridge among the world's unique ones (Fig. 2).



Fig. 2. Monoštor Bridge over Danube River between the Slovak Republic (left) and Hungary (right). Road steel bridge with orthotropic deck, main span 252 m and transversely inclined composite steel and concrete pylon 118,1 m high above base plate, 84,5 m high above the road. Open on September 17, 2020. The green Railway Bridge in the background has spans 81,84 m + 102,90 m + 2 x 103,32 m + 102,90 m. It was built in 1910, reconstructed in 1954. (Photo: NIF Zrt.).

4. Choice of structure system

Due to the proximity of the railway bridge, navigational aspects were decisive in the design of the bridge openings. For vessels passing through the relatively small, 100 m wide bridge openings, an additional pillar located so close to the middle of the fairway would present difficulties. Therefore, the largest span of the new bridge has a length of 252 m, which enables trouble-free navigation. In addition to the aforementioned navigational requirements, the determination of the location of the pillars was also influenced by anti-flood aspects on the Slovak side.

The 252 meter wide opening can be bridged most economically with a cable-stayed bridge structure. The spans from the Hungarian side are as follows: 66 m + 252 m + 120 m + 96 m + 66 m (Fig. 3). The width of the bed of the Danube River at this place is less than

500 m. The inclined cables are anchored at their upper end in an asymmetric pylon, the structure of which is made of steel in the upper part. Up to a height of +189.48 m above sea level the pylon has a composite steel and concrete cross-section. The L-shaped pylon is located on the upstream side of the bridge. It is connected to a pillar that is founded in the bed of the Danube River. Despite the asymmetric design of the pylon, the bridge is suspended in two symmetrical cable planes.

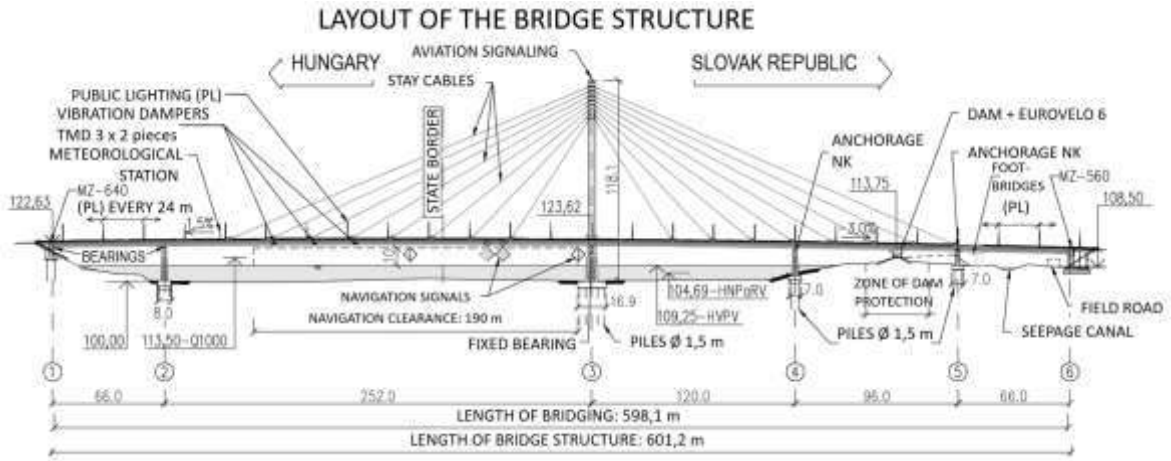


Fig. 3. Layout of the Monoštor Bridge over border Danube river between Hungary (left) and the Slovak Republic (right). (Source: Gabriel Drdanko).

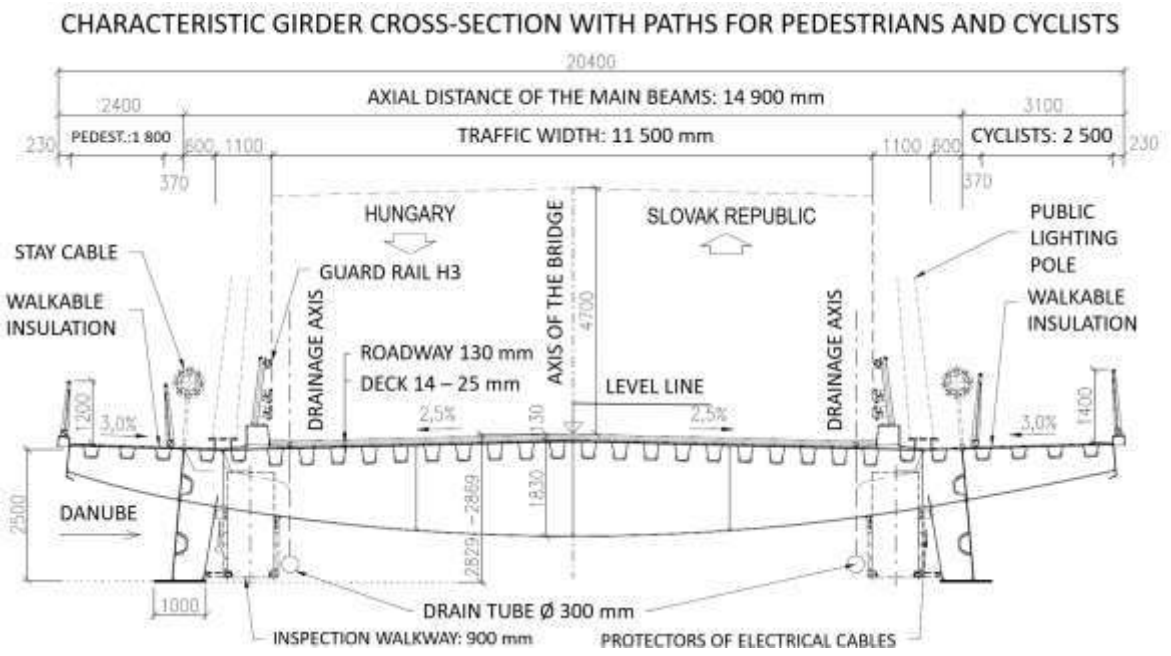


Fig. 4. Characteristic cross-section of the Monoštor bridge. (Source: Gabriel Drdanko).

5. Construction solution

The foundation of the bridge, apart from the abutment on the Slovak side, is realised by reinforced concrete piles with a diameter of 1.5 m or 1.0 m with lengths of 12 m, 15 m and 17.5 m, which support the base plates. The pillars have the similar appearance, but they are significantly different in size and function. They consist of the lower massive reinforced concrete part, which ends above the maximum flood level and is fixed into the base plate. Both faces of the pillars in the river bed, whose shape is similar to a Gothic vault, are lined with stone cladding. The faces are built with a slope of 6.5:1 and the slope

of the side walls is 20:1. The upper slender wall of piers supporting the bridge girder has vertical side walls. Both pillar foreheads are rounded. In the central part - between the bearing blocks - these walls are relieved by a semi-circular opening.

The difference of pillar No. 3 (Fig. 3) lies in the fact that the steel structure of the pylon is anchored to the substructure. With the help of a fixed bearing, this pillar transmits to the foundations, in addition to transverse horizontal forces, considerable longitudinal forces from seismic effects.

The difference of the pillars No. 4 and No. 5 (Fig. 3) in contrast to the pillar No. 2 are in that, that next to the bearings on each side, there is an anchor shaft for tie rods transmitting negative reactions from the bridge. The ramps and stairs of the walkways from the anti-flood dam are connected to the pillar No. 5 (Fig. 3).

The bridge girder consists of two beams of open section and has the steel orthotropic plate (Fig. 4). The total structural height of the beam is 2,869 m. The axial distance of the beams is 14.90 m. The length of the console for pedestrian sidewalk is 2.4 m and the one for cyclists is 3.1 m.



Fig. 5. Bottom of the bridge girder. Pillar of the pylon. Three Tuned Mass Dampers (TMD) on both sides of the bridge girder. Location of the dampers see in Fig. 3. (Photo: NIF Zrt.).

Location of 2 x 3 Tuned Mass Dampers (TMD) of the bridge girder may be seen in Fig. 3 and Fig. 5.

The orthotropic bridge deck under the roadway is reinforced with closed 30 cm high and wide trapezoidal longitudinal stiffeners. Under the walkways for pedestrians and cyclists, the stiffeners are only 20 cm high. The thickness of the deck plate is generally 14 mm, in the areas over supports16-25 mm.

The height of the two plate girders is 2.50 m and they webs have longitudinal closed trapezoidal stiffeners. The thickness of the webs in the field is 16 mm, above the supports 20-30 mm and in the anchorage area of the stay cables 50 mm.

The width of the bottom flanges is 1.0 m and they are extended to 1.3 m at the supports. Their thickness varies from 40 to 80 mm. The axial distance of the cross beams, the bottom flange of which web has the shape of arc, is 3.0 m (Fig. 5).

The height of the pylon above the road is 94.5 m and above the base plate 118.1 m. The bridge has two symmetrical cable planes with a fan-shaped arrangement of stay cables

(Fig. 7). The spacing of the stay cable anchoring to the bridge girder is 24 m (Fig. 6) and to the head of pylon is 2.78 m (Fig. 8). Prestressing was done in the pylon. This significantly reduced the time for moving the stressing jacks between individual anchors. The stay cables are made of the bundle of 7-wire strands (Fig. 10). Adjusting the anchorage in the pylon, if necessary, also allows fine rectification of the entire cable at once with a large stressing jack. The number of 7-wire strands in the stay cable varies from 43 to 85 pcs. The inner part of the composite steel and concrete cross-section of the pylon, which is in compression from bending moment due to the large eccentricity, is filled with reinforced concrete.

Vibration dampers are built into each stay cable near the bottom cable anchorage (Fig. 6). They weigh 5 tons.



Fig. 6. The spacing of the stay cable anchoring to the bridge girder is 24 m. Cables are equipped with dampers. (Photo: NIF Zrt.).



Fig. 7. Transversely inclined pylon and two symmetrical cable planes. (Photo: NIF Zrt.).



Fig. 8. The spacing of the stay cable anchoring to the head of pylon is 2,78 m. (Photo: NIF Zrt.).

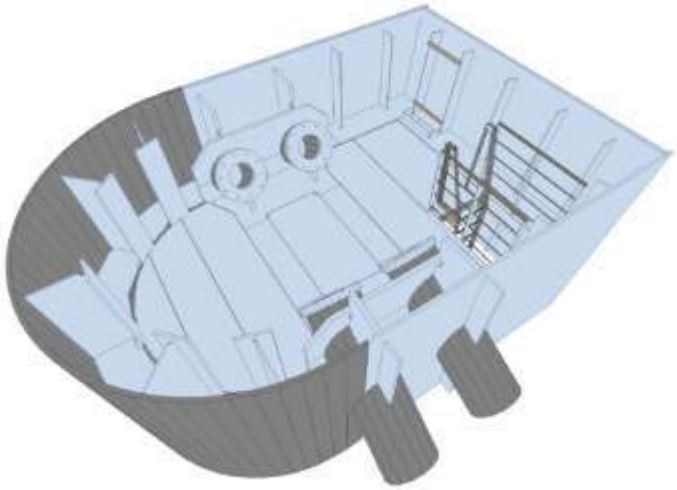


Fig. 9. Model od the pylon head with attachments of cables. (Source: Petr Brosch).

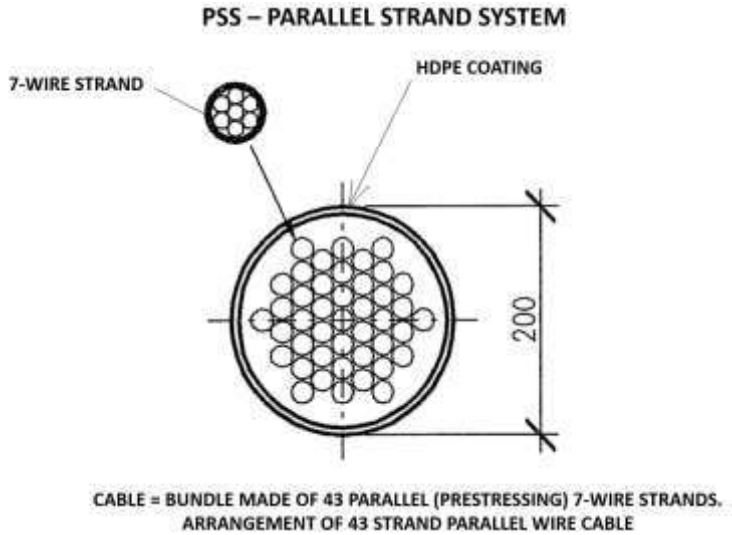


Fig. 10. The number of 7-wire strands in the stay cable varies from 43 to 85 pcs. (Ivan Baláž: terminology according to Eurocode EN 1993-1-11 of 2nd generation).

6. Bridge construction

Load tests of piles. Non-systematic static pile load tests were performed for each pile. The results of the tests proved the sufficient load-bearing capacity of the proposed pile foundations.

Foundation and construction of supports outside the riverbed. The abutment on the right bank and pillar No. 5 behind the dam outside the riverbed were built in the classic way (piles, base plate, rising wall). The abutment on the Slovak side has spread footing.

Foundation and construction of supports in the riverbed. The establishment and construction of three pillars No. 2 to No. 4 took place on artificial islands. The base plates are located below the level of the bottom of the trough. During the construction of the artificial islands, the contractor built a double sheet pile enclosure. The outer sheet pile wall of the cofferdam is parallel to the inner rectangular cofferdam and was supplemented on the inflow and outflow sides with a V-shaped cofferdam. This maintained acceptable hydrotechnical conditions. The space between the sheet piles, 0.6 m apart, was filled above the bed level with concrete ensuring the waterproofing of the construction pit. The space for the base plate of the pylon with dimensions of 17.0 m x 41.0 m was defined by the internal sheet pile cofferdam. Construction work was supported from boats moored along the islands. In addition to the machines located on the boats and in the construction pit, the work itself was supported by a floating crane with a capacity of 200 tons. The supply of materials and machines to these vessels was carried out from the shore along a road perpendicular to the axis of the river. It consisted of two connected boats and a small bridge between the shore and the boats.

The inner sheet pile walls of the artificial island were supported by steel frames, where the pylon assembly work was carried out. These were realized primarily by a tower crane with a load capacity of 50 t anchored to the foundation block of pillar No. 3 and attached to the pylon at two levels. The assembly of the last highest-mounted parts was carried out with a boom at a height of approximately 135 m above the surface of the base plate.

Islands No. 2 and No. 4 had similar technical solutions but had smaller dimensions.

The sheet pile walls were realized as being vibrated out of floating bay. However, during the implementation, it became clear that the composition of the subsoil in the place of the artificial islands is significantly variable. In some sections, it was not possible to vibrate the sheet piles to the designed depth, or to drive them with a pile driver weighing first 8 t and then even 12 t. For this reason, it was necessary to pre-drill the foundation soil using a Ø 800 mm screw drill. The guardrails were delivered from the factory as paired, their total width is 1.2 m. The locks of the paired sheet piles of the inner walls are welded and those of the outer walls are pressed. Sheet piles 19.3 to 22.3 m long were built.

The use of the island itself, and the associated technologies, can be characterized by three stages.

In the first stage, the island filled with a fill made it possible to carry out piling works and jet grouting. Jet grouting ensured practical watertightness of the pit, especially in the second stage, when the base plate of the pillar was built. The level of the footing bottom of the underlying concrete of the foundation is at an elevation of 95.65 m above sea level, i.e. 4.35 m below the level of the river bed. In the third stage, the support including the pylon was built.

Erection of the bridge superstructure. The bridge girder of the Monoštor Bridge is exceptional also in that it was produced in all V4 countries. Individual parts were produced in the bridge factory BANIMEX, Sp. Z. o. o. in Poland and in Slovakia in SAM, Inc. (Komárno shipyard). Subsequently, the entire cross-section was completed and welded on the Csepel island in Budapest, and from there it was transported by boat on the Danube River to Komárno.

The construction of the load-bearing structure can be divided into three parts. In the first stage, the construction took place by extension from the scaffolding located in the stream immediately behind the pylon between support No. 3 and No. 4. The parts of the load-bearing structure transported by ship were unloaded by a ship's crane onto the scaffolding and subsequently moved towards the Slovak side to support No. 6. The second part of the construction of the load-bearing structure between support No. 1 and pillar No. 2 was also carried out by extension it from the scaffolding. It also moved towards from the Danube River towards bank. The extension from the Danube towards the banks was justified because the parts of the load-bearing structure were transported by waterway. The third part of the construction of the 252 m long field of the load-bearing structure between pillar No. 2 and pillar No. 3 was constructed using cantilever method and a ship's crane. In this way, a total of 15 assembly parts were installed in the main span of the bridge. The length of one part was up to 18 m. During the construction using cantilever method, the largest length of the cantilever was up to 224.4 m. By inserting the last part of the load-bearing structure with a ship crane in December 2019, the superstructure of the bridge built from both banks of the Danube River was connected (Fig. 11).



Fig. 11. Inserting the last part of the bridge girder. (Photo: János Karkus).

7. Pylon construction

For the construction of the pylon, the time limit was only one year according to the construction schedule.

The pylon of the bridge consists of a steel closed section with a variable height. The width of the section is 4.0 m. The compressed part of the steel pylon was filled with C50/60 concrete up to a height of 90 m, thus utilizing the advantages of both materials. The most stressed parts of the pylon with a thickness of 30-80 mm were designed from S460 NL steel. The steel part of the pylon is fixed into the concrete shaft in the lower part, in which the inclined steel strut of the pylon is also concreted. For coupling with concrete, mandrels with a diameter of 25 mm and a length of 225 mm are welded to the steel part. There are 8 cables located in the tension part of the pylon cross-section (Fig. 12). The cable consists of bundle of 19 7-wire strands (Fig. 10). The cables enable minor compensation of horizontal deformations of the pylon caused by shrinkage or creeping.

During construction using cantilever method, several conditions had to be taken into account and all of them precisely synchronized. The following conditions were involved:

1. installation of a part of the pylon, as well as a part of the load-bearing structure, 2. welding of the load-bearing structure and the pylon, 3. tensioning of one pair of cables from the left bank side to 70%, 4. tensioning of one pair of cables from the cantilever side of the main field to 70% and subsequent tensioning up to 100%. After the cables were tensioned in the first level, reinforcement and concreting was still in progress inside the pylon one floor below. The assembly and related work of the individual parts took place from a three-story movable scaffolding attached to the pylon's external stiffeners.

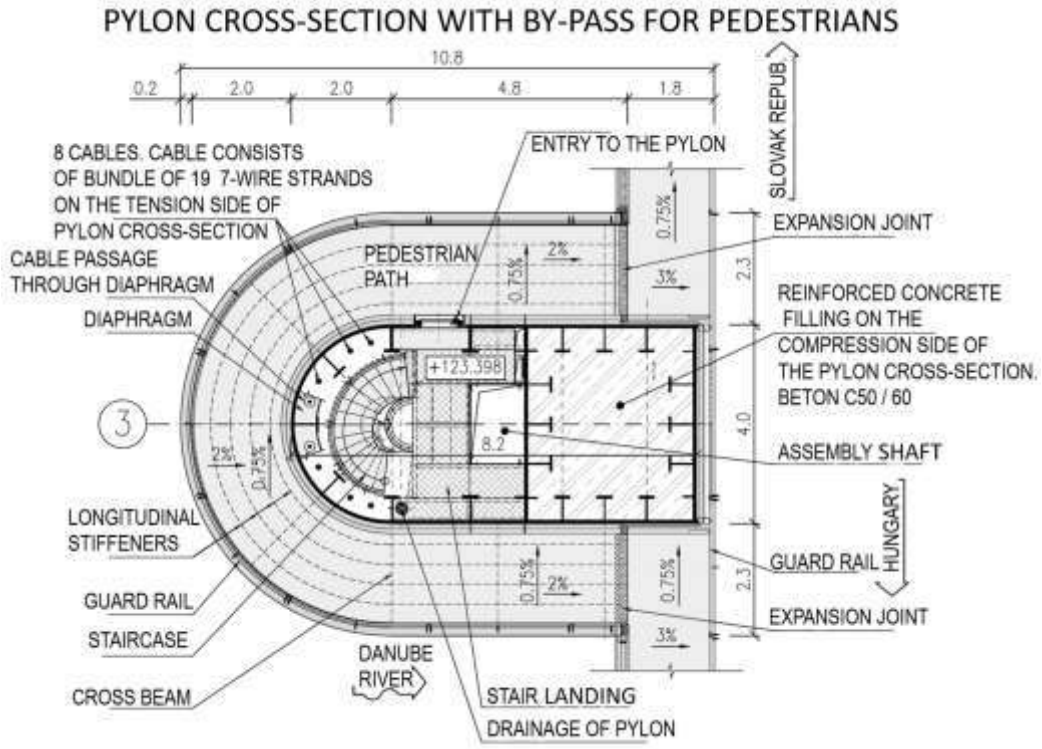


Fig. 12. Pylon cross-section. (Source: Gabriel Drdanko).

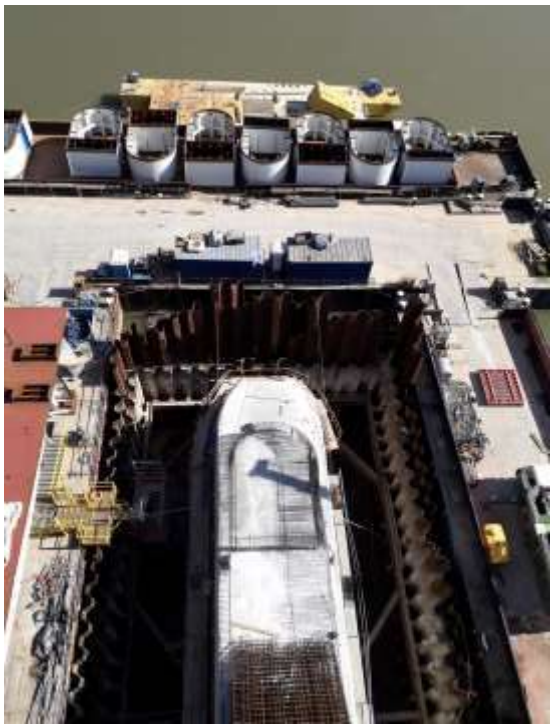


Fig. 13. Pylon construction. (Photo: NIF Zrt.).

The construction of the pylon was probably logistically the most difficult part of the construction of the entire bridge, as it involved work at height in difficult conditions, in which several professions participated. The pylon was produced in the MCE Slaný bridge factory, Ltd., in the Czech Republic, from where the individual parts were transported to the place of installation. Individual assembly parts were flooded directly to the pylon using pontoons.

During the processing of the pylon documentation, today's design methods in the BIM mode were used. The coordination of all construction professions, which were carried out on the small floor area of the pylon, was only possible thanks to the 3D model (Fig. 9). It was the main source of data for the creation of drawings of the production technical documentation of the steel structure. In addition to examining the structural and shape connections of OK, it also made it possible to assemble complicated parts from individual components. The model also solved the assembly space for the placement and tensioning of vertical cables, bridge deck cables, concreting, communication roads, utility networks, drainage, preparations for crane assembly and disassembly, and others.



Fig. 14. Loading test. Position No.8 with 10 + 22 loading trucks giving maximum support bending moment. One of many different test loading positions depending on influence line of internal force in investigation. (Photo: NIF Zrt.).

8. Bridge accessories

Railings for pedestrians with a height of 1.2 m and for cyclists with a height of 1.4 m were installed on the bridge in a total length of almost 2500 m with a weight of close to 100 tons.

The lamellar expansion joints, installed along the entire width of the bridge girder, enable large expansion movements of the steel load-bearing structure divided by a fixed bearing at the place of support No. 3 into two dilatation lengths of 318 m and 282 m.

On the bridge, one-sided steel guardrails with retention level H3 are installed (Fig.4).

The bridge is supported on each of the six supports on two spherical bridge bearings. It is located in one of the most seismically endangered areas in Slovakia (reference peak ground acceleration a_{gR} = 1.10 m/s^2). Seismic stoppers were designed on the lower flange of the main beams.

9. Conclusion

The Monoštor Bridge is the 2nd largest cable-stayed bridge in Slovakia with an orthotropic plate deck surpassed only by the SNP Bridge opened in Bratislava (1972) over the Danube River.

From the point of view of the construction of bridges on the Danube River, we experienced an exceptional period in the Slovak Republic in the years 2016-2021. In this six-year period, three road bridges were built on a 100 km long section of the Danube River: in 2016, the Starý Most (Old Bridge) in Bratislava was commissioned, in 2020 the Monoštor Bridge, and in 2021 the Lužný Most (Meadow Bridge) on the highway bypass (D4) of Bratislava. With this article, the authors wanted to present the realization of a bridge with a unique transversely inclined *L*-shaped pylon, which changes the landscape around Komárno.

The bridge was awarded in Slovakia in the competition Structure of the Year 2020-2021 the award "For an exceptional and progressive project solution" and in Hungary in 2021 the award "Exceptional quality in construction".

10. Decision-making participants of bridge realization

Investor:	NIF Zrt. (Hungary) and the Slovak Road Administration.
Author:	Ladislav Nagy (bridge) and Ivan Marko (territorial integration).
Designer:	DOPRAVOPROJEKT, Inc. and Pont-TERV Zrt. (Hungary),
Main contractor:	H-M DUNA-HÍD Konzorcium (Hídepítő Zrt. and Mészáros és Mészáros Kft.)
Construction manager:	Réka Hlatky and Balázs Varga.
Period of realization:	08/2017-05/2020.

Acknowledgement

The project was launched in charge of the Ministry of National Development of Hungary and the Ministry of Transport, Posts and Telecommunications of the Slovak Republic, invested by the National Infrastructure Development Company in Hungary and Slovenská správa ciest (Slovak Road Administration), financially supported in 85 % by the EU.

The English translation of the paper was made by Ivan Baláž, who also slightly modified the content of the original paper written in Slovak language [7].

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ABOUT TWO ORTHOTROPIC STEEL DECK BRIDGES ALONG HEMUS HIGHWAY IN BULGARIA AND ANALYSIS OF THEIR OVERALL CONDITION.

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University of architecture, civil engineering and geodesy

Abstract: *In this paper two OSD (Orthotropic Steel Deck) bridges along Hemus highway in Republic of Bulgaria are presented. Both of them are built at 1985 and are in operation almost 40 years. First bridge superstructure is with closed section longitudinal ribs of the OSD and steel plated main girders with 72,45m span. The second bridge superstructure is with open section longitudinal ribs of the OSD and steel box main girder with 162m span. The intensity of traffic and the weight of vehicles have considerably increased up to present times. Present status of the bridges, based on visual inspection of accessible parts, is evaluated and discussed. The statement in respect to fatigue behavior in accordance with the used structural details in the OSD and the prescriptions of the present design standards is analyzed and discussed.*

Key words*: Steel bridges, OSD, structural details, present condition*

1. General Characteristics of OSD bridges and Realization in BG

Orthotropic steel decks (OSD) are relatively complex constructions containing steel plate, stiffened by longitudinal ribs and cross girders. The bending stiffness is different in the two perpendicular directions with respect to the stiffness of longitudinal ribs and the cross girders. The OSD for bridges are officially defended by a patent in Germany in 1948 [1], nevertheless such type of decks are realized before [7]. The advantage of OSD in comparison with reinforced concrete (RC) deck is the relatively low self-weight, which makes them suitable for long span bridges and movable bridges. The disadvantage is the relatively higher price of the OSD, but considering the lower self-weight for bridges with long spans over 100-150m the OSD solution can be economic. In these cases serious optimization of the main girders and substructure can be realized, compensating the higher price of OSD, and the overall effect can be positive. In the long span range for girder bridges the most popular solutions are OSD bridges and prestressed concrete cantilever bridges. One of the main problems in the prestressed concrete cantilever bridges is to estimate correctly the progression of deformations in time due to long term processes in concrete – creep, shrinkage [12]. One of main problems with OSD bridges is fatigue

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damage accumulation. In the OSD there is high number of complex details of crossings between elements leading to low fatigue strength zones [7]. In accordance with the practical experience in Netherlands fatigue cracks occur in OSD after about 20-30 years of service [3] and adequate monitoring and preservation measures must be taken from the relevant authorities in order to provide repair works and to restore the reliability of the bridge. Some parameters of OSD road bridges in Republic of Bulgaria are presented in Table 1. It can be seen that 7 of the considered 8 bridges are in service over 20 years to 2019 and taking into account the unsatisfactory fabrication and erection quality in some cases the probability of fatigue crack occurrence in the near future is high.

Table 1. Some parameters of OSD road bridges in Bulgaria [2], [4], [6], [7], [13,14].

№	Bridges	Year of comletition and puted in exploitation	Maximun span of the bridge [m]	Years of service to 2019	Type of structural steel	Type of longitudinal stiffeners	Authority responsible for maintenance and preservation
1	Bridge over the old channel, Varna (small Asparuhov bridge)	1975	31.620	44	M16C и 10Г2СФ	open type section	Varna Municipality
2	Asparuhov bridge, Varna	1976	160.000	43	St 52-3	open type section	Varna Municipality
3	Road overpass near Elin Pelin railway station	1981	30.000	38	St 37-2	closed section	Road Infrastructure Agency
4	Road overpass along "Brussels" blvd., Sofia	1983	43.180	36	Ст09Г2Б-М	open type section	Sofia Municipality
5	Viaduct along Hemus highway at old km 26^{+884} (Viaduct 1)	1986	72.400	33	Ст09Г2Б-М	closed section	Road Infrastructure Agency
6	Viaduct along Hemus highway at old km 27^{+500} (Viaduct 2)	1986	162.000	33	St 52-3	open type section	Road Infrastructure Agency
7	Bridge near "Vartopo" park, Sofia	1999	60.000	20	Ст09Г2Б-М	closed section	Sofia Metropoliten
8	Road overpass over Andrey Liapchev blvd., Sofia	2013	37.000	6	S355J2	open type section	Sofia Municipality

Considering the mentioned above it is important for the responsible authorities to specify and evaluate the critical details and to provide adequate monitoring and preservation measures where relevant.

2. Description of two OSD bridges in Bulgaria

During the construction of the Hemus highway two steel viaducts with OSD were built at km.35+144 (old km.26+884) – Viaduct 1 and km.35+760 (old km.27+500) – Viaduct 2.

Viaduct 1 (Fig.1, Fig.2) contains of two continuous girder OSD bridge superstructures with length 60+4x72,45+60=410m (maximal span 72,45m) in horizontal curve with radius

850m and longitudinal slope 4,61% [4]. The steel consumption for Viaduct 1 is 400kg/m^2. In accordance with original design the bridge superstructure is composite steel and concrete plate girder – Fig.3, but during the execution of one of the superstructures by means of incremental launching technology in 1985 failure occur [5], [6] and in order to decrease the repair time OSD is developed – Fig.4 [4]. After the failure due to destruction of the temporary supports steel girders fail to the piers and displacement up to 1500mm from the design position in cross direction is registered [5], [6].



Fig. 1. Viaduct 1 at km 35+144



Fig. 2. Viaduct 1

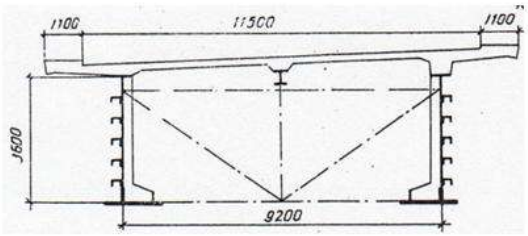


Fig. 3. Original design [4]

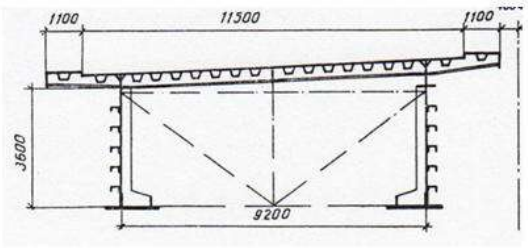


Fig. 4. OSD cross section [4]



Fig. 5. Steel bearing at abutment



Fig. 6. Support zone

During repair works damaged plates of the steel girders with deflections up to 50-70mm are heated and set to design position by means of hydraulic jack influence, zones with deflection over 70mm are cut and replaced. The superstructure is supported on elastomeric bearings at internal pier supports and steel longitudinally movable bearings at the abutments. At abutment zones under dilatation joints serious corrosion damage of bearings and steel girders is registered – Fig.5 and Fig.6. The degree of carrying out their function is questionable for these longitudinally movable bearings due to significant corrosion degradation – Fig.5. Local cyclic variable deformations in steel plates and secondary stresses are frequent reason for fatigue damage accumulation and crack initiation in some critical locations. Mechanisms of fatigue crack initiation in steel plate and cross girder web in OSD are presented on Fig.7 and Fig.8. In order to reduce the influence of local load – Fig.7 in present Eurocode standards the minimal plate thickness of OSD for road bridges is increased – Table 2 [7].

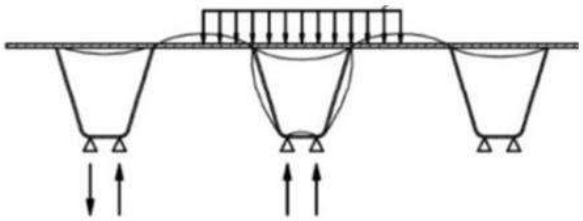


Fig. 7. Influence of local load on plate [9]



Fig. 8. Cross girder web deformation [9]



Fig. 9. Crack in weld at main girder Support



Fig. 10. Crack in longitudinal rib connection



Fig. 11. Longitudinal rib to cross girder web connection



Fig. 12. OSD at connection joint

Potential fatigue crack at weld connection between closed trapezoidal longitudinal rib to 12mm thick steel plate at the zone of the first intermediate support of the main girder is presented on Fig.9. The connection between longitudinal rib and cross girder web without openings in the web is presented on Fig.11. The reduced plasticity in cold formed trapezoidal rib in cold bended zone, combined with high tension residual stresses from welding leads to low fatigue strength detail – fatigue category 36 [10]. This detail is not recommended for road bridges [9].

Table 2. Minimal thickness of the steel plate in OSD for road bridges.

Minimal thickness tpl [mm] of the steel plate in OSD for thickness of the asphalt layer t_{asf} [mm]			
Design standard	Year	t_{asf}>=70mm	t_{asf}<70mm
BDS ENV 1993-2:2001	2001	12	14
BDS EN 1993-2:2007	2007	14	16

Probably due to complex geometry there are considerable imperfections between prefabricated elements, compensated by cut after measuring at the place parts of the longitudinal trapezoidal ribs – Fig.12. Parts of the longitudinal ribs are connected by means of steel strip welded to the connected parts by fillet weld. Potential crack in such detail is presented on Fig.10. The overall condition of the Viaduct 1 is relatively satisfactory for 34 years of service. In near future the bearings must be repaired and due to described fatigue problems the monitoring must be intensified and measures for registered problems must be taken. After adequate rehabilitation and strengthening the design load bearing capacity of the bridge can be achieved.

Viaduct 2 (Fig.13, Fig.14) contains two continuous box girder OSD bridge suerstructures with length 100+162+100=362m (maximal span 162m) in horizontal curve with radius 1467,25m and longitudinal slope 4,61% [4].



Fig. 13. Viaduct 2 at km.35+760



Fig. 14. Viaduct 2 at km.35+760

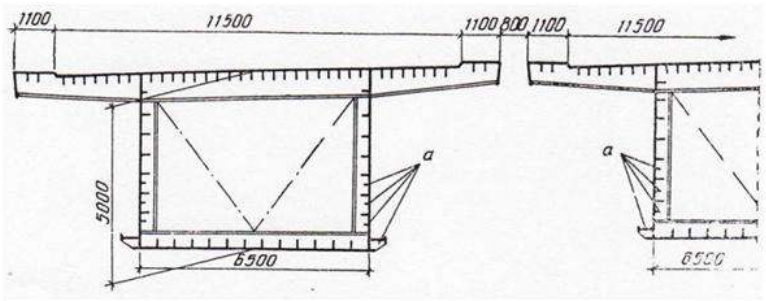


Fig. 15. Cross section of the box [4]



Fig. 16. OSD in the box

The bridge superstructure is executed by means of incremental launching technology [6]. The slenderness of the superstructure is 32,4=162/5 (relation between maximal span and cross section height). The steel consumption for Viaduct 2 is 420kg/m^2. Cross section of the bridge is presented on Fig. 15. The OSD is constructed with open type longitudinal ribs – Fig.16, Fig.19, passing through one sided openings in the web of the cross girders. The transfer of vertical load between longitudinal ribs and cross girders is eccentric. In [9] it is recommended to use symmetric openings in cross girder web and both sides of the longitudinal rib to be welded to the web. At the abutment zones of superstructure for "Sofia to Varna" direction two concrete blocks each with dimensions 6500x2000x3500mm (~100ton weight) are provided to prevent negative reaction – Fig.18. When launching the other superstructure the external bearing (with respect to center of the horizontal curve) didn't step on the relevant support and for compensation of this problem one sided concrete block with dimensions 6500x4000x3500mm is realized to set this bearing in design position – Fig.17. It must be mentioned that this decision due to eccentricity of the concrete block leads to considerable torsion in the box section and induces additional stress-strain state in the relevant superstructure zone.



Fig. 17. Concrete block



Fig. 18. Concrete blocks



Fig. 19. Corrosion inside the steel box



Fig. 20. Corrosion inside the box

Inside the steel box there is corrosion mainly around some welds and in joints between prefabricated elements where there are holes and due to humidity and condensation corrosion areas occur – Fig.20. At Fig.19 significant corrosion at connection zone between steel plate and cross girder web is presented. It must be mentioned that corrosion destruction can serve as initiation of fatigue cracks in zones with cyclic variable stresses and increase the fatigue crack propagation velocity. In [8] strengthening concepts for OSD are discussed and a FEM with Shell finite elements of a simply supported test specimen with dimensions corresponding to zones of the OSD of Viaduct 2 is developed. It is evaluated that maximal values of normal stresses in longitudinal ribs due to characteristic value of tandem system TS of LM1 [11] are relatively high for local bending (bending of longitudinal rib between cross girder supports) - 268MPa (from local bending of the longitudinal rib in the real case with continuous girder static scheme this value is 218,7MPa). These stresses must be added to global stresses of the main box girder in relevant zones and after multiplying by partial safety factors. The overall normal stresses

due to traffic loads in accordance with [11] in these zones can exceed the yield strength of the steel. There is corrosion damage in zones of superstructures near dilatation joints at abutments. The movable bearing of superstructure for "Varna to Sofia" direction is presented on Fig.21. The bearings at abutment of superstructure for "Sofia to Varna" direction are replaced – Fig.22, but there is corrosion in superstructure over the bearing.



Fig. 21. Bearing at abutment "Varna to Sofia"



Fig. 22. Bearing at abutment "Sofia to Varna"

The overall condition of the Viaduct 2 is relatively satisfactory for 38 years of service. Bearings of one of the superstructures are replaced (superstructure in "Sofia-Varna" direction). The corrosion zones must be rehabilitated in order to stop the progression and where relevant compensate the corrosion damage. Detailed analysis for load bearing capacity and fatigue resistance of the superstructures in accordance with present design standards must be provided and strengthening and preservation measures to increase the service life must be realized.

3. Conclusions

The considered in the paper two OSD bridges along Hemus highway in Republic of Bulgaria are in operation from 38 years at present time and excluding described above problems are in relatively satisfactory overall condition. In many zones near bearings and dilatation joints there is serious corrosion damage. Only several small fatigue cracks in OSD are registered in the provided partial visual inspection. It must be mentioned that for the same service life of 38 years (only about 30% of the 100years prescribed for bridges in EN1990) some superstructures with simply supported GT - prestressed concrete girders along Hemus and Trakia highways achieved serious damage and near to failure condition and were replaced. In accordance with the prescriptions of the present design standards in the considered viaducts there are some details with low fatigue resistance and local zones with potentially insufficient load bearing capacity. It must be mentioned that additional internal stresses, with difficult for estimation values, due to specific operations during execution are present is particular zones of these superstructures. Also in Viaduct 1 and Viaduct 2 there are zones with steel plate thickness 12mm which do not correspond with prescriptions of [9], Table 2. From this point of view the probability of fatigue crack

occurrence in the steel plate is high. In conclusion due to their 38 year service life the monitoring of the two steel OSD viaducts must be intensified in order to register and repair in adequate time period possible initiated defects (fatigue cracks). For the future periods strengthening program must be developed in order to evaluate the condition and to put the described viaducts in correspondence with the requirements of the present design standards. Combination between the described above adequate evaluation of condition, repair and preservation can significantly extend the service life of the considered two OSD viaducts.

Acknowledgements

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CONCERNING A BRIDGE BUILT IN THE BEGINNING OF THE 2ND CENTURY, OVER THE RIVER STRYMON (STRUMA), TO ULPIA PAUTALIA (KYUSTENDIL), ON THE ROMAN ROAD STARTING ON THE EASTERN SIDE OF THE PASS OF SUCCI (PASS OF TRAJAN) TO THE ADRIATIC AND ROME

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***Abstract**: The report is a scientific study of an unknown Roman bridge from the beginning of the 2nd century, from the infrastructure of the main imperial road directly connecting "Via Militaris" (today also called "Via Diagonalis") with the military port near the city of Lissus (Italian: Alessio, modern: Leža, Albania) on the Dalmatian coast of the Adriatic Sea and from there through the port of the city of Brindizium (modern-day Brindisi) to Rome. Several of the great Roman bridges along this route are also indicated.*
The bridge was located next to Ulpia Pautalia (now Kyustendil) and crossed the river Strymon (present-day Struma) in close proximity to a Roman road station. Around 1900, the historian Yordan Ivanov studied and measured the remains of the abutments of this bridge, and published the results in 1906 in one of his books.
According to this photographing, it is the five-arch construction typical of Roman bridges with two bank and four river abutments, with roadways that descended quite steeply to the two river banks from the top above the center of the middle arch. Its characteristic is unique with a width of 9m, and side arches of 12m. The only other bridge in Europe with such an anomaly is "Ponte Alconetar" over the Tegus River on the route of Via de la Plata, today in the Extremadura region of Spain, considered the work of the imperial architect of Trajan and Hadrian - the brilliant Syrian Apollodorus of Damascus. Since he was building the bridge over the Danube at the time when Emperor Trajan issued his decree declaring Pautalia a city, he most likely commissioned his architect to build some of his structures, including the bridge.

***Key words:** Concrete arch bridge, arch composition, bridge preservation, main Roman roads, Bulgaria, history of the antique architecture, Balkan peninsula, Apollodorus of Damascus, Emperor Trajan*

One of the most important Roman roads in Eastern Europe was "Via Militaris" (today also called "Via Diagonalis"), connecting Rome to Byzantium (Constantinople). It was built as a continuation of "Via Aemilia" from the city of Placentia (now Piacenza) in the direction of the colony of Aquileia, with construction starting in 148 BC. by the consul Spurius Postumius Albin Marcus (148 – 146 BC), on the new road section "Via Postumia". This entire route actually served the connections of the important port of Genoa with the northern Adriatic coast.

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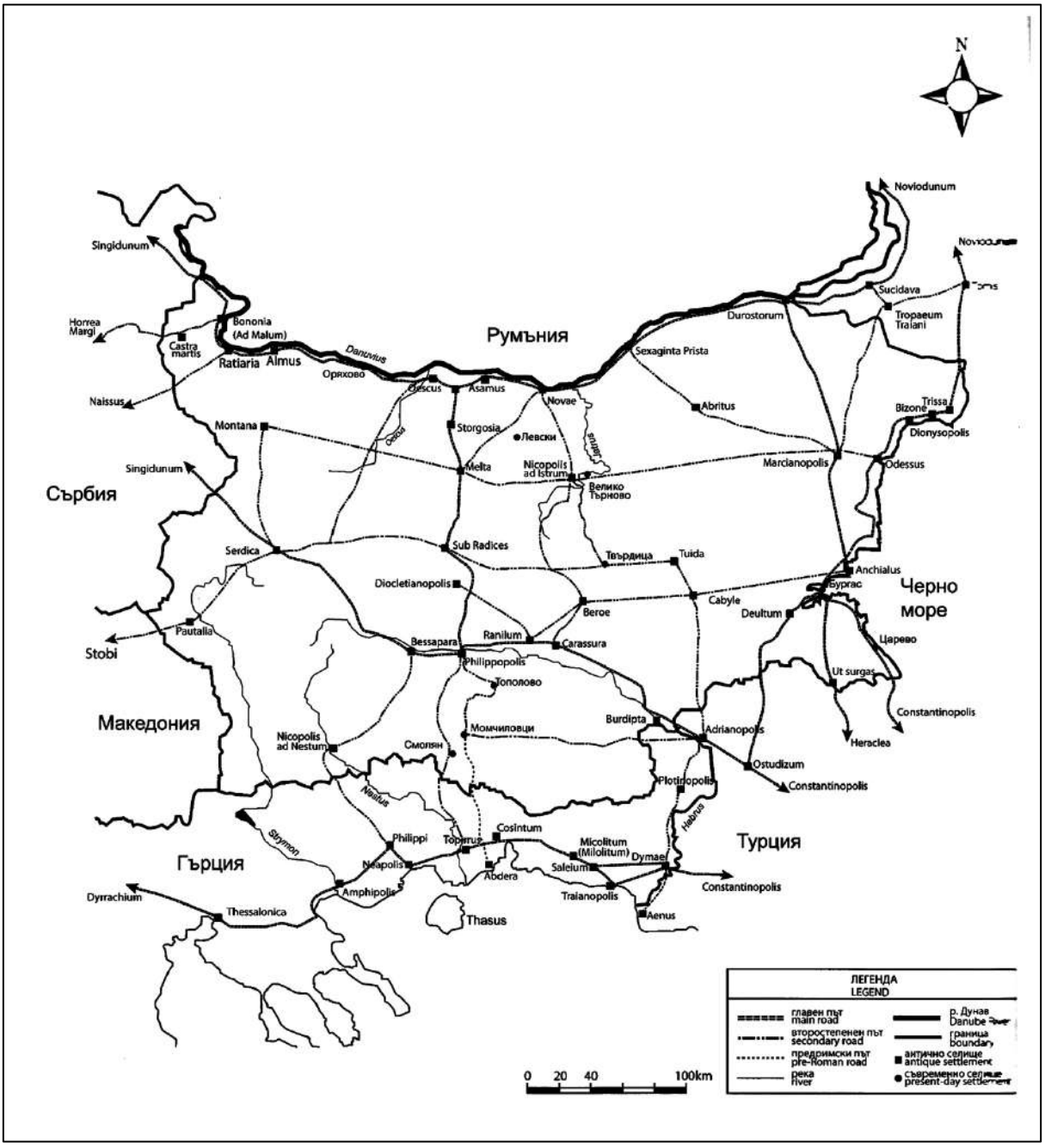
Fragment of a map of the Roman Empire with the main roads in South-Eastern Europe from the first quarter of the 2nd century

After Aquileia, in the time of Gaius Julius Caesar (49-44 BC), the road continued to be modernized further, entering the Balkans through the new extension "Via Cykia", through which it reached Sirmium (now Sremska Mitrovica, Serbia) in the province of Pannonia (after Emperor Trajan's administrative reform of AD 103 - "Upper Pannonia"). Therefore, the route of the "Via Postumia" along with its extension began to be collectively called the "Via Sirmium".

Much after the time of Caesar, in the second half of the 1st century AD, the construction of a new extension following Sirmium began, on a very ancient pre-antique route known until then as the "Road to Italy". Its first section runs in a direction along the Danube (and is therefore called "Via Istrum") and passes through Singidunum (present-day Belgrade), Gratiana (present-day village of Dobra, Golubacko) and Viminacium (present-day village of Stari Kostolac, Pozarevacko) , where another route separates from the river which turns towards Naissus (now Nis) and from there continues through Remesiana (now Bela Palanka), Serdica (now Sofia), Succi Pass (now Trajanov Pass), Philipopolis (now Plovdiv), Hadrianopolis (modern Edirne), Arcadiopolis (Thracian Bergula, modern Lüleburgas), Tzirallum (modern Chorlu), to reach the Thracian city of Byzantium (later Constantinople, modern Istanbul) joining, near Heraclea (now Marmara Ereili village, Tekirdag province), the route of "Via Egnatia". From the ports of the Bosphorus and Heraclea, connections were established by sea to the Roman provinces in today's Asia Minor and the Arabian Peninsula.

The construction of this longest section of the "Via Militaris", from Sirmium to the Sea of Marmara and the Bosphorus, began during the reign of Emperor Nero (54 – 68 AD), and was mainly built between 29 AD (the route on the territory of present-day Serbia) and 61 AD (most of the remaining section), but it was completed under the reign of emperor Trajan (98–117). Subsequently, mainly in the 4th and 5th - 6th centuries AD, some road reconstructions were carried out and new road stations, fortresses and bridges were built. The reinforcement of the routes connected to Trajan's pass (Trac. "Succi", lat. "Succorum claustra" and "Succorum angustiae"), which is a key point for the defense of Thrace and the lands around the capital of the Eastern Roman Empire - Constantinople, is particularly active.

Probably at some time during the 4th century AD, here began the reconstruction, according to Roman standards, of the most important direct main route connecting "Via Militaris" with the Western Balkans, the Adriatic coast and Rome. It provides a direct connection between



Map of the Roman roads from the territory of modern day Bulgaria according to Madzharov

the newly created capital of the Eastern Roman Empire – Constantinople, the city of Adrianople and the territory of the Western Roman Empire.

This new Roman road starts in a southerly direction, diverging perpendicularly from the "Via Militaris" immediately after entering Trajan's pass into the Upper Thracian valley. It descends along the southern slope of Golak to the river Maritsa and crosses it via a stone bridge, which was located between today's town of Belovo and the village of Momina Klisura. Here, a road junction connecting three routes was formed. The most important of these, which is one of the main Balkan Roman roads, leads southwest.[1] At first it runs along

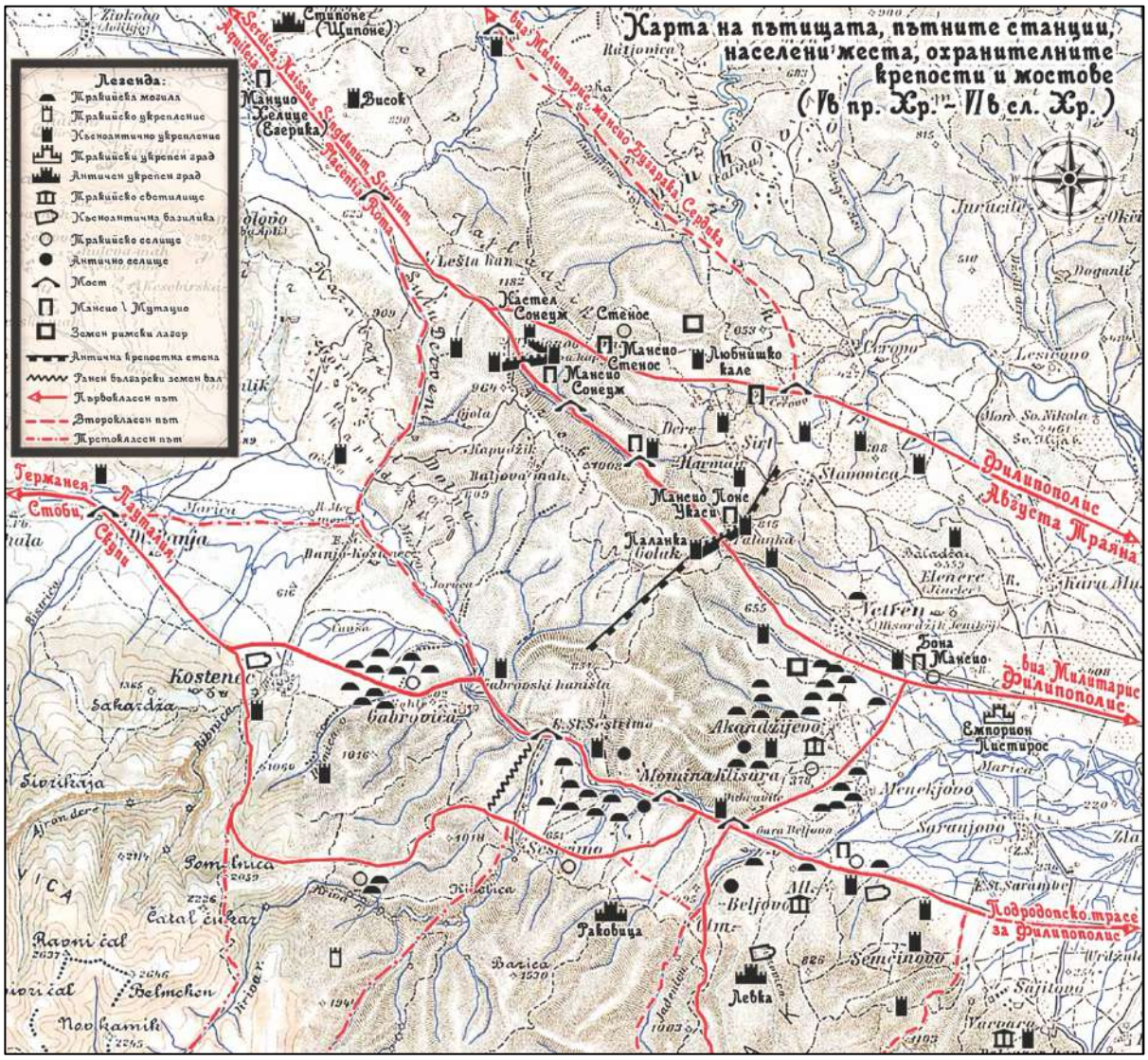


The eastern side of Trajan's Pass according to an engraving by Thomas Allom, published in Constantinople and the scenery of the seven churches of Asia Minor, 1833

the river to the north-west, then turns left from the passage through the land of the village of Sestrimo, where in the past there was a preserved section of its stone pavement. It is precisely this route, further crossing the Musala section of Rila, that is the beginning of Constantinople's most direct road connection through "Via Militaris" to the Dalmatian coast and the rest of the southern parts of central Europe and from there with the Italian peninsula. This road was constructed in order to avoid the serious detour through Serdica (Sofia), from where a road connection with Via Egnatsia departs perpendicular to "Via Militaris". It is for this reason that the second route in question took shape in its final form much later, probably in the 5th - 6th century AD.

After the bridge over Maritsa near the present-day town of Belovo, the main road to the southwest crosses Rila directly to descend into the Kostensko-Dolnobanska valley and reaches the present-day town of Dolna Banya. But here a later developed backup route appears, which from the bridge continues along the entire length of the Momino-Klissurski pass, to the eastern end of the same valley, where it turns west towards Dolna Banya.

The junction at the bridge and the three main roads leading from it, including this one, are not marked in ancient maps and guidebooks. Only the Podrodopski road to Filipopol (now Plovdiv) is marked with the names of its road stations in the section after another stone bridge crossing the Maritsa River near the village of Boshulya to the present-day town of Pazardzhik. The main route leading west to Ulpia Pautalia (now Kyustendil), which deviates to the left from the Momino-Klissurski pass through the village of Sestrimo, rises up into the mountain where it passes through the area "Pomochena Polyana" and descends through today's "Vili Kostenets" in The Kostenetsko-Dolnobanski valley near an ancient city (incorrectly identified



Map of the roads and road infrastructure from the ancient period in the area of Trajan’s Pass according to Tuleshkov

by Ireček as the one known from the springs of Kostantia) near the present-day village of Kostenets. After that, it passes through the area of nowadays town of Dolna Banya, where there was a Roman settlement with a mineral bath. Further on, it enters Sredna Gora along the Gutsalska River, turns below Drenova Mogila (as a guard fortress is positioned on the height covered by this turn) to exit into the Samokov Valley. From here the road passes below the so-called "Shishmanovo Kale" (an ancient fortified settlement with a monastery, destroyed in the 6th - 7th centuries, which grew into a medieval town, the predecessor of Samokov), continuing through the plain to pass over the Escus river (Latin Oescus, modern day Iskar) via a bridge, which was reconstructed during the Ottoman era as a stone-wooden bridge. The road then continues to the "Klisurski Pass", the eastern entrance of which is guarded by the fortress today called "Tsari mali grad", and after passing it, continues to the west in the lowland below the fortified Roman city of Germaneia (modern-day Separeva Banya) to reach the nearby mansion near the modern-day village of Cherven Breg.



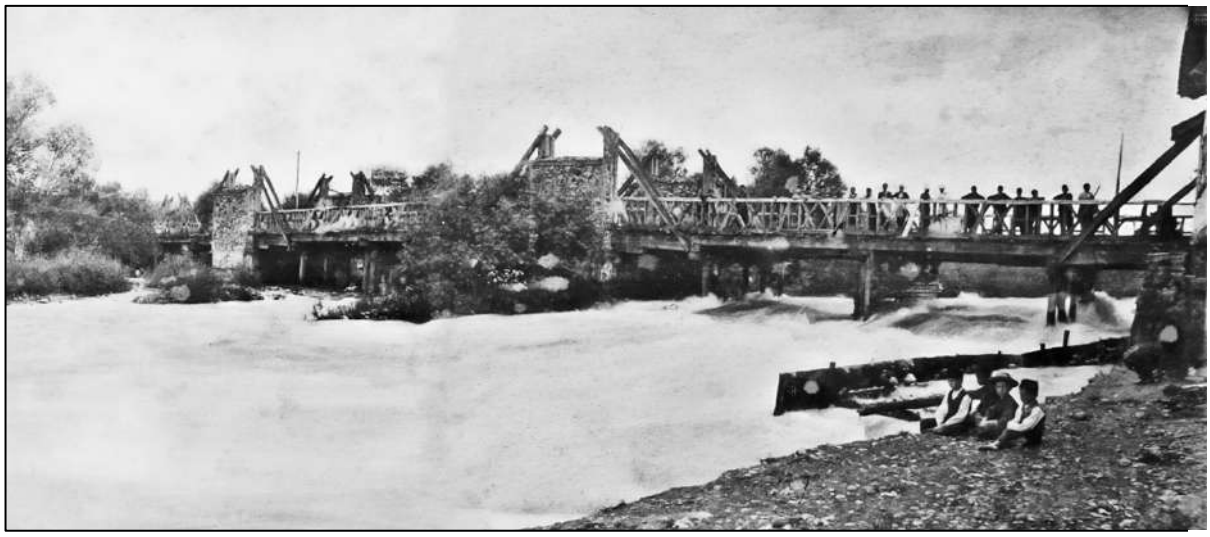
Remains of the Roman road from the Pomochena Polyana area in the Musala section of the Rila mountains, between the lands of the villages of Sestrimo and Kostenets



Remains of the section of the Roman road in the lands of the villages of Kostenets and Dolna Banya



The road in its section passing through the village of Dolna Banya, according to a photo from the period 1881-1884



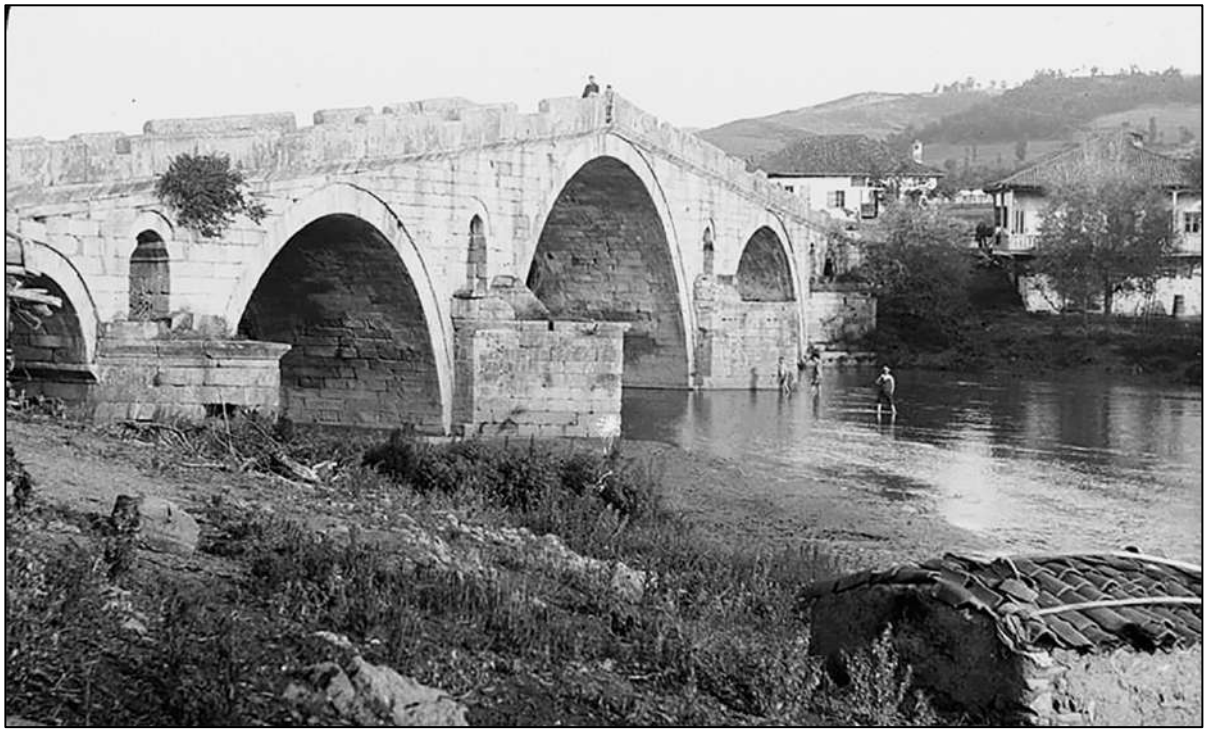
The bridge over the Iskar river near the town of Samokov according to a photo from the 1880

Somewhere here the road splits, but both of its roughly parallel routes finally merge right before the town of Ulpia Pautalia. The first of them continues directly through the Konyavska mountain, at first through the mountainous parts of the historical-geographical region of Razmetanitsa, where it goes through the lands of the modern-day villages of Blatino (station Gorno Yahinovo), Golyamo selo, Novoselyane, Korkina, Gorna Koznitsa, Tavlichevo, and in this area merges with the secondary road coming from Serdika, through Radomir, Dolna Rakovica and Klenovik (old Kolош). After the merger of the two roads, their generalized route continues to the area of the present-day villages of Gorna Graštitsa and Dolna Graštitsa, where a square-plan mansion with very thick outer walls, protected by a cylindrical tower, was excavated on its land, and then the road crosses the river Strymon via a large granite bridge entering the Kyustendil valley.

The second route, known to the local population as "Drachkio pat" and "Drumo", descends after Cherven breg to the south along the left bank of the Dzherman river and crosses it via a bridge (now Arakchiiski bridge, on the outskirts of Dupnitsa). Another, long-lost bridge over the Bobovdolska river follows, and from there, entering the Razmetanitsa area, another of the few mansias explored to date is reached, near the Drumo hamlet in the village of Dolna Koznitsa. Somewhere here was a junction with the transverse secondary road from Serdika, continuing south to Heraclea Sintika (near the modern-day village of Rupite) and "Via Egnatia". The next larger river - Strymon (present-day Struma) is also crossed via a bridge in the area below Nevestino in the area of present-day’s Chetirtsi village, which connected the neighborhoods on both banks of the ancient city to reach along the coast to Ulpia Pautalia. After the destruction of this ancient city, its bridge also seems to have ceased to exist. Only in 1469/70, due to the importance of the road as a main trade route for the Ottoman Empire, a new facility - the stone "Kadi (Kadin) bridge" - was built a little further up the river. After this



The stone-wood "Arakchiiski Bridge" over the Dzherman River near the city of Dupnitsa according to an archive photo



The Ottoman "Kadi ("Kadin") Bridge" from 1469/70, over the river Strymon (modern-day Struma) by the village of Nevestino near the town of Kyustendil

Roman city, the path of the two parallel routes, which merged into one in the Kyustendil valley, continues to two other large Roman cities in the territory of modern-day North Macedonia - along a detour to the south to Stobi (Stobis, at the mouth of the Erion River (now Tsrna river)) in the river Achios (modern Vardar) and another westward to Skupi (now Skopje), from which respectively it continues from Stobi along Achios to Thessaloniki and Via Egnatia, and through it to the port of the city of Dyrachium (modern Drač), and from Skopje directly to the Adriatic military port south of the city of Lissus (Italian Alessio, modern Leža, Albania). From here there was also a continuation along the Dalmatian coast to the main city of the province of Dalmatia - Salona (Colonia Martia Julia Salona , now the village of Solin three kilometers from Spalatum, now Split) to reach Aquileia, where it connects with the "Via Postumia". From these two ports - Dyrachium and Lissus, ships crossed the Adriatic Sea to the port of Brindizium (modern-day Brindisi) and then through the road system of the territory of the Apennine Peninsula, a connection was made mainly to Rome.

The importance of this road from Trajan Pass, known later as Dalmatian, Bosnian or Adriatic, is indicated by several large Roman bridge structures along its route, such as the seven-arched Arakchiiski bridge over the river Dzherman near Dupnica, the two neighboring bridges over the Strymon near Pautalia, the five-arched bridge with wide arches over the Pchinya River near the village of Sredno Koinare south of Kumanovo and the large "Stone Bridge" in Skopje over the Aksii (Vardar) River, crossing it shortly before reaching the archaeological site of the ancient city of Skupi on the left bank. It seems that this part of the road in the section near Pautalia was modernized in the Late Antiquity after its destruction during the barbarian invasions of the 5th century, during the large-scale fortress and road constructions of Emperor Justinian I the Great. Procopius of Caesarea mentions its fortress system as one of those restored in the region. At that time, a



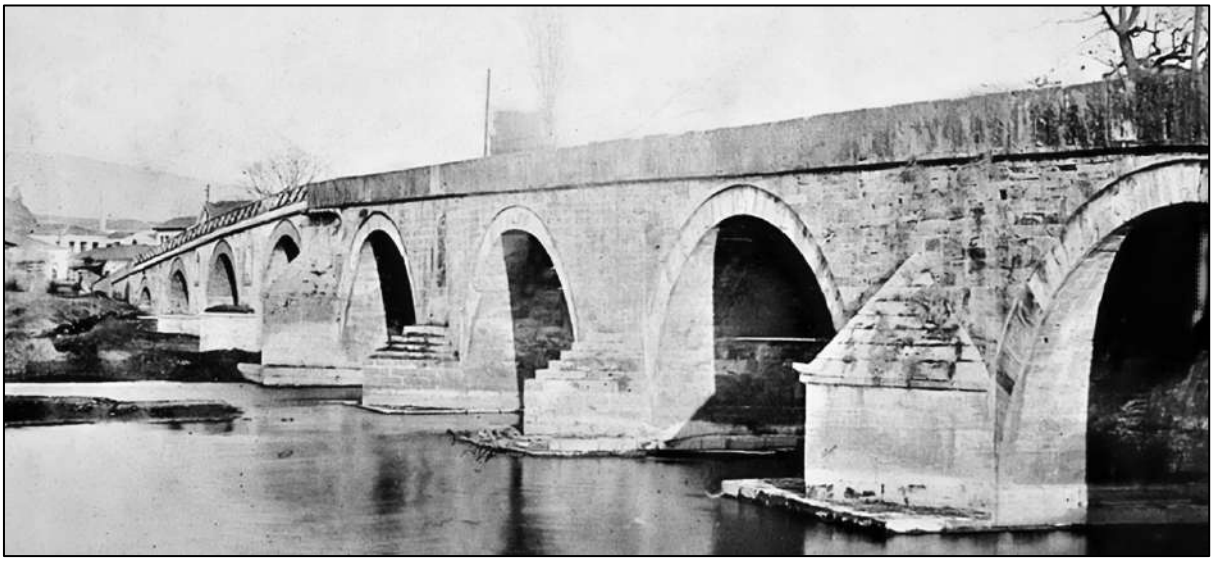


The Roman bridge over the Pchinya River in the area "Blatiyata" near the village of Sredno Konyare, Kumanovo, North Macedonia. Views according to photos from 1915 and modern-day

number of fortifications were rebuilt along this route, for example at Germaneia (greek Γερμανία), along with six more fortifications along the road to Rila and to the southwest near modern-day Dupnitsa, as well as in the direction of Pautalia, along the two parallel routes.

It is then that the road becomes a significant artery and is therefore not marked on the Poitinger map (IV century). Only the Serdika - Pautalia road is marked on it, without its continuation further south along the Struma River through Skaptopara (Blagoevgrad) and Heraclea Sintika (near the village of Rupite, Petrichko), to Via Egnatia, which will appear on the maps only after 4th – 5th century, when modernization was clearly carried out, turning

this road into a first-class one, judging by its wide multi-layered profile covered with stone slabs, of which sections have been preserved, for example in the "Stronski rid" and "Pobuchenik" localities (about 600m in length) between Krupnik and Kresna, where the road passed over the Kresna gorge, as well as in other places.

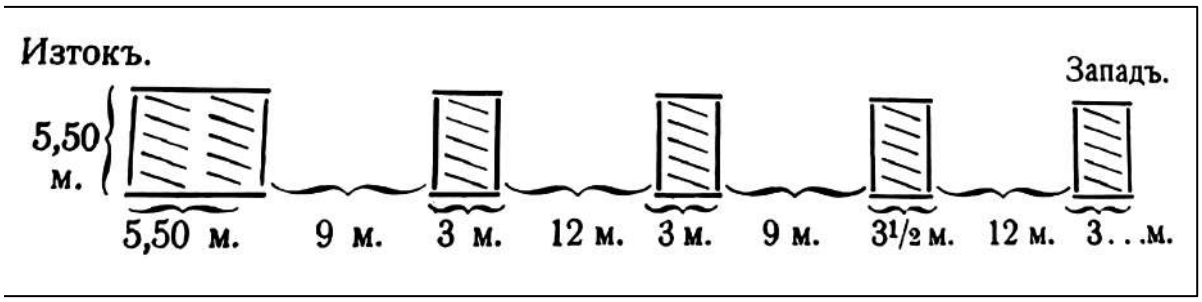




"Stone Bridge" over the Vardar River in Skopje, North Macedonia. Photo views from 1876 - 1890 and modern-day

About the bridge built with granite blocks over the Strymon before the road reaches Ulpia Pautalia, Yordan Ivanov writes: *"There was a 70-meter long granite bridge on the Struma, the foundations of which are still preserved from the sides (shore banks-N.T. note) and the supports (river beds - N.T. note), half submerged in a dry riverbed. These bridge remains of large carved granite slabs are still called by the local population "Raklite" (i.e. bridge abutments - N.T. note). Due to the constant and unnoticed filling of the local left river bank from the slopes of the Konyavska mountain, the Struma moved further west, so the bridge remained dry and its material was lifted".*[2]

It becomes clear that when, in 1530, after Cherven breg, Benedikt Kurpešić passed *"through a mountain and reached a nice stone bridge near a village called Oslokostanitsa* (Turk. Oslokostija, dn. Slokoštitsa-N.T. note)", he meant precisely the one that was apparently still well preserved and maintained at the beginning of the 16th century.[3] And it is after the road descends from the mountain that it crosses the Struma river via said bridge and then before Kyustendil it reaches the village of Slokoshtitsa, which was then positioned northeast of the modern-day village near the present-day cemetery park, where the ruins of one of its medieval churches - "St. Archangel Michael" are located.[4] Yordan Ivanov made a sketch and concrete measurements of this undoubtedly ancient and medieval structure, which after analysis defines it as a five, or if there were any small relief openings in the shores - six or seven arch bridge. According to him, it is oriented in the East-West direction, but the geographically correct direction is North-East (left bank) - South-West (right bank). Due to the changes of the river bed and the continuous cultivation of the land, especially on the right bank, the remains of the south-western end of the bridge had already disappeared before the end of the 19th century as a result of the aforementioned plundering for building material. Today, there may be something surviving of the foundations below ground level. On both banks, there are still scattered granite blocks, several of which are quite large in length, exceeding 100-150 cm.

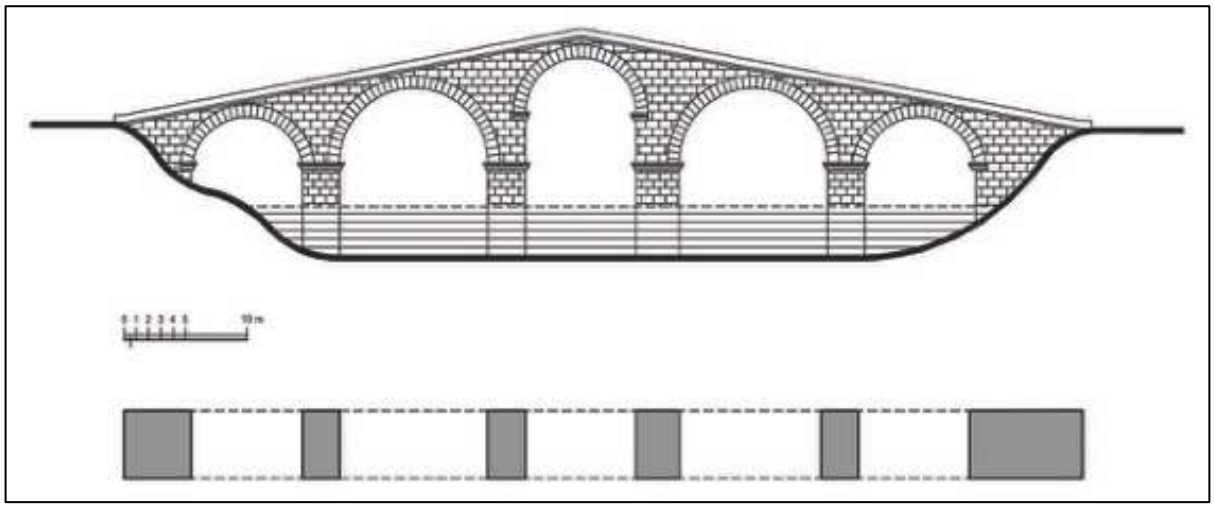


Plan of the Roman bridge over the Struma river near the town of Kyustendil according to Yordan Zahariev

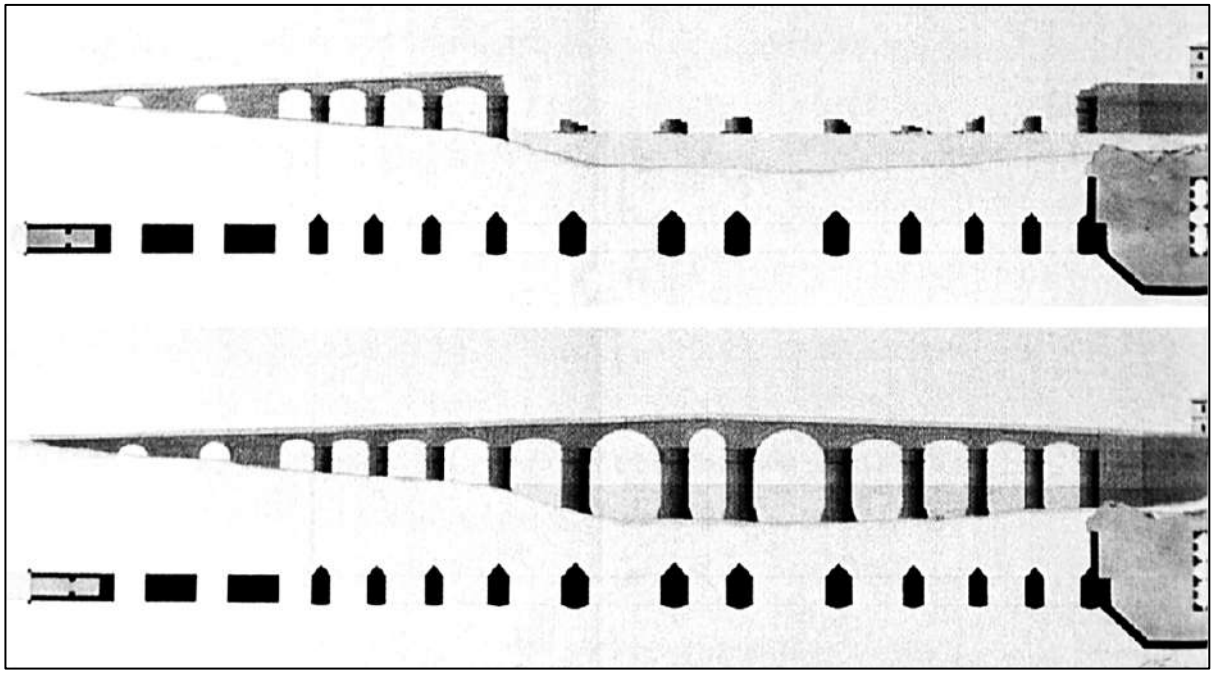
This is why Ivanov photographed only the left bank embankment and four river embankments after it. As the left-bank breakwater is shown as quite short and ending in a vertical side-wall, and the right-bank breakwater is missing from the drawing, the actual length of the structure and its total number of arches remain unknown. However, it can be said with confidence that this is a typical symmetrical Roman humpback bridge where the inclined roadways meet at the highest point above the 9m wide arch on both sides of which there are vaults higher than 12m. And this means that after the last - south-western embankment at a distance of another 9 m followed the opposite embankment. However, since this bank is a little lower, it was correspondingly longer than the opposite one.

Indeed, in the most common typology of Roman humpback bridges, the largest of the arches is placed in the center of the structure. But in the case near Kyustendil, we see a 9m wide vault framed on both sides by larger 12m vaults. Such is, for example, the only completely uniform solution known to me so far- that of the "Ponte Alconetar" over the Tegus River on the route of Via de la Plata, today in the Extremadura region of Spain. It is also built of granite and is characterized by the use of segmental vaults, and since these were used mostly in the 1st - 2nd centuries, the bridge dates back to the time of the great builders of road infrastructure - the emperors Marcus Ulpius Trajan (98 - 117) and Publius Aelius Trajan Hadrian (117 – 138). Moreover, the fact that the main imperial architect of Trajan and Hadrian was the brilliant Syrian Apollodorus of Damascus (c. 50s - c. 130s), in whose work

segmental arches are often present, is the reason to assume that "Ponte Alconetar" is most likely among his large-scale projects. It is unlikely that this happened in Hadrian's time, for as Dio Cassius (c. 200 - c. 230) wrote in his History of Rome, the emperor hated him for his free speech and therefore exiled him and subsequently ordered him killed.[5] But still there are researchers who believe that the architect worked for him not only during the first years of his reign, but even that there was no murder at all.



The Ancient bridge over the Strymon river (present day Struma) near the village of Dolna Grashtitsa, next to Pautalia (now Kyustendil). Reconstruction plan by Nikolay Tuleshkov based on photographing of the abutsment by Yordan Ivanov

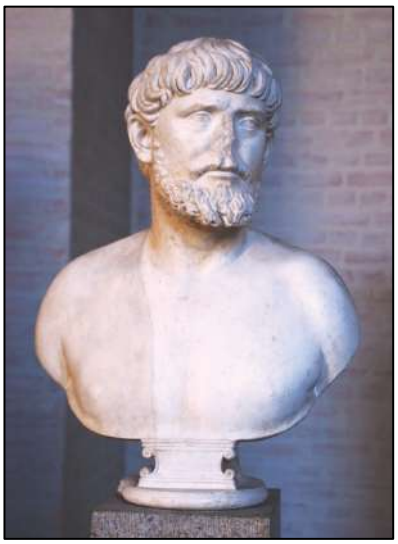


The Alconetar Bridge over the Tagus River along the route of Via de la Plata in the region of Estramadura, Spain. Plans by Fernando Rodrigues from 1797





The Alconetar Bridge over the Tagus River along the route of Via de la Plata in the region of Estramadura, Spain. Aerial photograph from 1956, Views from an engraving by Marquis Alexandre de Laborde from the period 1792 - 1798, published in: Voyage pittoresque et historique de l'Espagne. Tome I, Paris, 1811, and in its condition before being moved to a new location





Bust of the Syrian architect Apollodorus of Damascus from the Hypotheque in Munich, Germany, and a plaster cast of a fragment of Trajan's Column in Rome, depicting the consecration by Emperor Trajan of Trajan's Bridge over the Danube River (at the back of the image), where the patrician in the background to the left of the altar is most likely Apollodorus

In addition, it should be noted that Emperor Trajan built a significant part of the road network on modern-day Bulgarian territory - Via Militaris, Via Istrum, Via Traiana, improved with public buildings and fortifications the cities of Augusta Traiana (now Stara Zagora), Nikopolis ad Istrum, Nicopolis ad Nestum, Philippopolis (now Plovdiv) and others. In this territory, he also built his famous huge stone-wooden bridge over the Danube at Zhelezni vrata, west of Vidin, where in the upper part of each of the embankments there are two rows of imposts one above the other, as is the case with the "Alconetar" bridge In Spain.

Some planning similarities between the Ponte Alconetar and the Struma Bridge should be noted. For example, the width of the narrow central arch in both facilities is the same - respectively 8.92 m in the first and approx. 9m for the second. The pair of arches surrounding the central arch of "Alconetar" has spans of 16.72 and 16.74m, and the widths of the next pair are 12.03m and 11.93m, respectively.[6] Due to the modest length of the bridge over the Struma, the widths of the first pair arches are about 12m each.

Ivanov photographed the abutments without showing the presence of breakwaters, but perhaps if there were any they were very low, as is the case at Alconetar, according to Fernando Rodríguez's architectural survey and reconstruction (1797), and therefore buried in sediments rendering them invisible.[7] All the similarities, together with the unique layout specific only to these two facilities, suggest that the authorship of the two bridges probably belongs to the same architect of Emperor Trajan. There is a strong possibility that it was the Syrian Apollodorus of Damascus, one of the greatest Roman architects. However, we do not have any historical evidence that the construction itself was managed by him on the spot.

In order to justify this assumption, however, the fact that in 106 AD Emperor Trajan declared the Thracian settlement of Pautalia as an urban center in the province of Thrace (later

Inner Dacia) under the name "Ulpia Pautalia", which according to Yordan Ivanov translates as "Spring city", originating from the Latin word "pauta" meaning "hot spring". Precisely because of its healing mineral waters, it was already known for its baths. Archeologically and documentally, it has been established that during the time of Emperor Trajan, as well as under his successor Emperor Hadrian, the city reached its greatest prosperity. It was then, according to an inscription found in 135, that his civil basilica was built.

Everything presented so far, and especially the significant architectural coincidences with the unique Alconetar bridge, allows us to state the hypothesis that the bridge over the Struma river at Ulpia Pautalia dates from the beginning of the 2nd century, when, undoubtedly due to the newly acquired high city status, the road connections were modernized along with their infrastructure.

NOTES

1. The text on „The ruins of the old bridge" (Развалини-ты на старийтъ мостъ) over Maritsa is written here by Zahariev, who notices them here circa the middle of the 19th century. (Zahariev, S. Geographic, Historic and Statistical description of Tatari-Pazardzhik Kaaza. Vienna, 1870, p-p. 25).
   The travelogue of Quiclet (1657) makes it evident that this is an old, undoubtedly Roman bridge. He describes how, together with his companions, after leaving Trajan's Pass in Thrace, they immediately turned right towards Cuzdervent, crossed Maritsa "through a ford in their carts at a destroyed bridge" and from there they continued to Filipopol (modern day Plovdiv) along the Podrodopski road, through Malko Belovo (Quiclet, M., The journey of M. Quiclet to Constantinople and back. - In: French travelogues on the Balkans in the 15th - 18th centuries, 1975, p-p 222)
2. Ivanov, J., North Macedonia (Съверна Македония), Historical research. S., 1906, p-p 20
3. German and Austrian travelogues on the Balkans. S., 1979, p-p 150. This is proven by the text, which explains how they reached the bridge by crossing "one mountain" and not through the valley of the Struma River.
4. In 1530, it was the village of Slokoštitsa that was the first village on the road on the right bank of the river after the "Granite Bridge" before Kyustendil, not the "Kadin Bridge", as some authors believe, which is more distant and, moreover, after Cherven breg the road crossing it goes along the valley of the Struma River, and not through the Konyavska mountain.
5. In some historical studies, the thesis is developed that Apollodorus was not executed, but continued his work under the reign of Publius Aelius Trajan Hadrian, but it is unknown exactly until when. In any case, based on architectural analysis, the Pantheon in Rome is considered to be the work of Apollodorus, created in the period 118 - 126, which means that he worked for the new emperor for at least nine years after his accession to the throne.
6. Dimensions are by Benedikt Seidl for Wikimedia Commons based on the drawings of arch. Fernando Rodríguez from 1797, but perhaps the photography and engravings of the bridge made under the supervision of Marquis Alexandre de Laborde in the period 1792-1798 were also used (See : Voyage pittoresque et historique de l'Espagne. Tome I, Paris, 1811).
7. The original drawings of arch. Fernando Rodriguez from 1797 are in the archives of the Gabinete de Dibujos de la Real Academia de Bellas Artes de San Fernando in Madrid, Spain (signatures A-3638 and A-3639).

SURVEY OF DIFFERENT MODELLING APPROACHES FOR HISTORIC MASONRY ARCH BRIDGES

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Abstract: *An attempt has been made in the report to give a brief description of the existing state of the problem defined above, in the light of its relevance and modern scientific research around the world. According to the authors of the report, the debate on this topic is important for our country, due to its great importance for our cultural-historical architectural heritage, examples of which can be seen in the centuries-old Bulgarian examples, namely: Kadin Bridge, Svilengrad Bridge, Devil's Bridge and Koljo Ficheto Bridge*

Key words: *masonry arch bridges, deteriorations, damage, failure patterns; load-bearing capacity*, *MFE*

1. Introduction

1.1. Description of the existing state of the problem, previous scientific research on the topic of the project and relevance of the scientific issues

At the moment in Bulgaria, the problem has not been developed in the necessary depth and there are no modern historical and scientific-technical studies in this area. Currently, several stone bridges in Bulgaria have been studied on the subject and published in: Chapter Bulgaria in "Handbook of International Bridge Engineering (HIBE2014), published by the American publishing house "Taylor&Francis Group" in 2014 (Fig. 1a and Fig.2b).

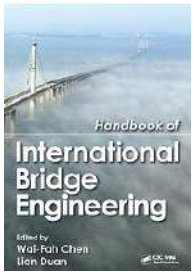


Fig. 1a. Handbook of International Bridge Engineering

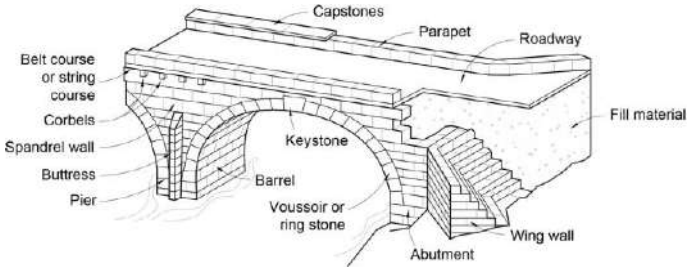


Fig. 1b. Constitutive elements of the masonry bridge

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Several stone bridges significant for our centuries-old history have been studied from a constructive point of view: Kadin Bridge near Kyustendil (Fig. 2), Svilengrad Bridge (Fig. 3), Devil's Bridge on the Arda River (Fig. 4). near the village of Ardino. The project for the rehabilitation of the Koljo Ficheto bridge near Byala-Rusensko was studied, which will be soon in realization (Fig. 5). The results of these studies have been published in several conferences at home and abroad. Other similar surveys of our famous stone Roman bridges filled with massive stonework, built in the Rhodope Mountains, at the beginning of the Christian calendar, are forthcoming. The accumulated experience during their survey will make it possible to form a whole direction in the methods for determining the stressed and deformed state of historical arched bridges of massive stone masonry, under load from their own weight and earthquake, using the finite element method.



Fig. 2. Kadin bridge



Fig. 3. Svilengrad Bridge



Fig. 4. The Devil's Bridge



Fig. 5. Kolyo Ficheto bridge

Papers dealing with the application of FEM for determining the stressed and deformed state of existing structures of Stone Historical Arch Bridges(SHAB) have been analyzed. An attempt is made to create a classification for assessing the impact of cyclic loading on the safety of the structures of (SHAB) in a three-level scale: non-threatening and directly threatening the safety of the structures, conditions that bring the structures to an emergency state. A goal is purposefully formulated to create a scientific product in which: the genesis, development and current state of the problem related to the joint behavior between different types of stone structural elements in (SHAB) are investigated: namely: stone abutments, stone pillars, stone bridge arch and stone parapets under self-weight and earthquake loads using the finite element method. In order to solve the problems defined above, the authors of the report carry out the corresponding acquaintance with the various types of historical arched bridges in our country, filled with massive stone masonry, which have become symbols of great historical and cultural value for our country. An architectural and structural survey and description of these samples was undertaken. A classification of the damage in these bridges has been done [4,5]. Modern European normative documents related to the study of the various types of (SHAB) in our country, filled with massive stone masonry, have been studied and analyzed. A specially designed masonry calculation software that takes into account the characteristics of stone block

masonry elements for historic arch bridges has been extensively studied. Such a program was created by a collective in Genoa and is called: TREMURI. Development started after 1999 after the catastrophic earthquake in Laquila, Italy, Greece, Turkey and Lamatrice[4,5].

2. Results obtained from the above-mentioned actions of the author's collective

2.1. A general synthesis of the studied foreign and our experience

Masonry arch bridges are found in many European countries, most notably in Italy, the United Kingdom, Portugal, Spain, France, Germany, Switzerland, and also quite a few in Belgium. Not to be overlooked are the stone railway bridges in the Czech Republic, Slovakia, Hungary, Romania, Bulgaria, Greece, Serbia and Turkey. Most of these structures were built either in the 19th century or in the early decades of the 20th century. The available materials and construction methods used to build these bridges are no longer effective today. However, the legacy of this type of bridge can be functional enough and have great cultural, even aesthetic value. The arched shape contributes to a balanced appearance and appeals to the human spirit. Yet, as traffic loads develop and due to structural aging and possible deterioration, the load-carrying capacity of these arches needs to be checked accordingly.

Various methods have been developed to assess the structural features of vaulted arches[1-17]. Some of them allow reporting defects found during monitoring. The destruction of masonry arches is characterized by its sudden appearance. Either the inner masonry layer peels away from the rest of the vault near the center of the arch, or a 4-joint mechanism forms in these places with reduced cross-sections due to cracking. The latter mechanism can be seen in highly visible cracking. Load capacity assessment can be done in a variety of ways depending on the degree of accuracy required. Design traffic loads are usually considered a major consideration in the event that the structure does not meet current requirements. In these cases, analysis to define the vulnerability of stone bridges to brittle failure, governed by the ratio of performance to demand, represents an effective method. Vertical traffic loads can be supplemented by horizontal loads independent of traffic or wind and generated by a sudden rise in the river level below the bridge. The latter include wave impact and debris impact on bridge piers. However, in this report, computational methods are the main focus of our future research. So far, the fastest assessment has been made using the MEXE method developed in the United Kingdom[15]. Several equations give a conservative value for the load capacity. A more refined method is hard block analysis. The mechanism of destruction is a matter of a subsequent second stage, of the behavior of the structure, since the destruction occurs after the formation of 4 joints between the massive masonry blocks. In addition, this method also allows the behavior of degraded areas to be accounted for. However, this method certainly gives less conservative results.

However, the failure mechanism cannot include 3-D effects in wider arched arches. Therefore, FE analysis can be used, provided that a non-linear material and geometric behavior model is included. For material nonlinearity, Mohr–Coulomb or alternatively Drucker–Prager laws are closest to real behaviors. Finally, those parts where failure occurs can be analyzed in more detail by simulating each element of the masonry, meaning the natural stone blocks or bricks and mortar. The characteristics of these materials are quite different and this in-depth type of analysis leads to the analysis of more complex critical behavior. Examples of vaulted arches will be considered to illustrate these different methods. They also include the Kolyo Ficheto bridge near Byala-Rusensko. One of the

main research goals for this bridge is to determine whether the effects of turbulent, water waves are more damaging than the effects of debris. In many other cases, the latter analysis seems more critical. A parametric study will complement this result.

2.2. Brief review of known methods for assessing the load-bearing capacity of stone bridges

2.2.1. Historical methods

Old studies, such as those of La Hire (1695, 1712) and Couplet (1729–1730), included theoretical analysis and experimental work on the pressure line in the arch, especially in arch collapse. Then Gregory (1697) deepened the shape of the arch in a chain bridge, as the most suitable solution for taking the weight of its own form. Heyman (1982[6]) considered this last observation in more detail, concluding that it can be interpreted as defining the lower bound of the plasticity theorem. Navier (1833) introduced a third rule for the middle third of the bridge cross-section, which was applied to masonry arches by Rankin (1898), so that the line of pressure was constrained to lie in the middle third of the arch to avoid tensile stress. Barlow (1846) and Fuller (1875) worked on graphical solutions for determining the pressure line in an arched structure. Castigliono (1879) used the minimum strain energy theorem to develop elastic methods, and for masonry arches he performed an iterative analysis quite similar to modern, non-linear finite element model (FEM) analysis (no stress/non-linear procedure, tension shear part of the arch). Further work developed by Rankine (1898) analyzed: the role of infill in masonry arches and recorded the first geometric/mechanical observations, including that higher infills were particularly likely for semi-circular or elliptical arches which would probably otherwise be unstable construction time; or to give the greatest possible security for the hydrostatic arch, the infill must be constructed of solid masonry up to the level of the crown of the extrados and the importance of the square lateral connections in the infill to avoid damage.

2.2.2. Modern methods

The first successive studies[1-17] on arch bridges were carried out by Pippard (1948, 1951[11]), Heyman (1969, 1972, 1976, 1980, 1982[6]), Withey (1982) and Tellet (1983): these studies made a significant contribution to our knowledge of the use of elastic destruction methods and mechanisms. Pippard also considers evaluation methods: the arch is held elastically until the first joint or crack occurs. It is then destroyed by a mechanism of four articulated devices. He sees that after the appearance of the first joint there is a considerable reserve of strength in the arch before it collapses. The elastic method improves the preliminary assessment of masonry arches: the basic assumption is for an arch with two joints of horizontal forces keeping the arch in equilibrium. Alfred John Sutton Pippard's (1948[11]) approach uses the partial derivative of the strain energy-U, with respect to a force, which is equal to the displacement in the direction of the force (Castigliano theorems; Castigliano 1879); the annulus is treated as a two-hinged parabolic arch with a secant variation of Ij/I0 secα, where I0 is the moment of inertia of the area at the crown. The axial force and shear force terms in the strain energy equation are ignored in this theory. Therefore, the strain energy is assumed to be completely dependent on the bending response of the arch. In this line of thought, it is necessary to introduce into the research the Military Engineering Experimental Establishment (MEXE)[15], method in the form of nomograms and is currently recommended by the UK Department of Transport in its departmental standard (Department of Transport , 2001). The MEXE method is a long-established system for assessing the load-carrying capacity of the masonry arch[15]. In recent years, it has been the subject of testing where some shortcomings have been

identified. There is now a growing consensus that the current version of MEXE overestimates the load capacity of short-span bridges, but for spans over 12 m it becomes increasingly conservative. This method was based on the two-hinged elastic analysis by Pippard[11], which was then calibrated with both field and laboratory tests in the 1930s (Oliveira et al., 2010[10). The method was most predominately used in World War II as a way to quickly classify the load-carrying capacity of older masonry arch bridges. However, since that time, the MEXE method has still been used as a way to load-rate masonry arch bridges. The modified axle load (MAL) depends equally on the arch and backfill thickness, although the ring thickness has significantly more influence on the arch behavior than does the backfill. The modification factors are introduced without taking account of the arch geometry; the backfill depth, ring thickness, and even the mortar thickness could have differing influences on arches with different geometries.

2.2.3. Empirical rules

Over the past several decades, numerous methods have been used to evaluate the load-carrying capacity of masonry arch bridges, using tools ranging from easy-to-use geometric rules to the most sophisticated finite element software. The first and simplest approach deals with empirical rules based on geometric relations coming from the proportions in the construction of the elements of the arch (span, arrow, thickness, width and height of the pillars, etc.). There is not always mechanical confirmation of these rules; however, they have been widely applied in the past with real-life arches. A summary of these methods is presented in Table 1[1].

Date	*Author*	*Deep arch*	*Shallow arch*
15th century	Alberti	$t=s/10$	-
1714	Agutier, $s>10$ cm	$t=0.32+s/15$	-
1777	Perronet	$t=0.325+0.0035\ s$	$t=0.325+0.0694\rho$
1809	Gauthey, $s<16$ m	$t=0.33+s/48$	-
1809	Gauthey, $16<s<32$ m	$t=s/24$	-
1809	Gauthey, $s>32$ m	$t=0.37+s/48$	-
1809	Sganzin	$t=0.325+0.3472\ s$	-
1845	Dejardin	$t=0.30+0.045\ s$	$t=0.30+0.025\ s$
1854	L'Eveille	$t=0.333+0.033\ s$	$t=0.33+0.033\ s^{1/2}$
1862	Rankine	$t=0.19R^{1/2}$	-
1870	Dupuit	$t=0.20\ s^{1/2}$	$t=0.15\ s^{1/2}$
1885	Croizette-Desnoyers	$t=0.15+0.20\rho^{1/2}$	-
1885	Lesquiller	$t=0.10+0.20\ s^{1/2}$	$t=0.10+0.20\ s^{1/2}$
1914	Sejourne	$t=0.15+0.15\ s^{1/2}$	-

s=span; R=radius of the circle passing through the crown and intrados springing; ρ=curvature radius

Table 1. Empirical Rules for Crow arch thickness(MEXE,1952)

2.2.4. A classic solution

The classical solution of arched, masonry bridges is mainly based on Heyman's theory (Heyman, 1997, 1982, 1996)[6], which simplifies the calculation of the ultimate load of the masonry arch by making the following assumptions: (i) the masonry members have an infinite compressive strength, (ii) the masonry units behave as a rigid body, (iii) the joints do not transmit stress, and (iv) the masonry units do not slide at the joints. As a result of these assumptions, the limiting plasticity theorems can be applied to determine the ultimate load of a masonry arch. Plasticity theories include two theorems: (i) an upper limit or mechanical solution; and (ii) a lower bound or equilibrium solution. The mechanism method is based on upper limit plastic analysis (Heyman, 1982[6]): The failure loads of the masonry arch can be determined by analyzing the arch as a mechanism instead of an elastic structure. The effects of joints on the failure load of masonry arches are analyzed, suggesting that possible locations of hinge points should be identified. The forces and

stresses in the indeterminate structure must then be calculated. The joint locations must then be adjusted based on the calculations and the process continues iteratively until the joint locations are stabilized (Livesley, 1978). Compressive force analysis is based on plastic analysis with a lower bound: Hayman[6] recommended determining the smallest possible arch thickness that will fit the line of pressure and with the assumed joint locations. This thickness is then compared to the actual thickness of the arch. It considers the ratio of the two values to be a geometric safety factor for the arc. From his work he developed a method he called: the "rapid analysis" method. It is based on an arc with dimensionless parameter inputs and a point load P. The equation is based on failure occurring at the joints of each of the springs that realize it under tension from a moving load and at the crown.

2.2.5. FEM analysis

Three-dimensional (3D) nonlinear FEM analysis offers the ability to model the entire structure by verifying the structural behavior of a well-defined and precise structure. All loads affecting the structure can be modeled.

2.2.6. Structural modeling

Another complex subject in masonry is the selection of an appropriate model to represent the structure. According to the homogeneous material hypothesis, the following types of models can be applied:

2.2.6.1. With Concentrated Masses

This is a rough approximation of the geometry of the structure, but may be sufficient to determine the structural dynamic response (if material nonlinearity and the resultant effects of the actual geometry of the structure are included). Obviously, this type of model cannot be used to predict local or global collapse mechanisms or failure rates of individual structural components. **With beams and columns**: This defines the behavior of the system in more detail than the previous approach. It is possible to determine the sequential formation of the failure mechanism model in both static and dynamic aspects.

2.2.6.2. Macro elements

This approach considers the structure as a whole of wall panels, each of which is a recognizable and complete part of the building. It may also coincide with a recognizable part of it architecturally and functionally (for example: the facade, apse or chapels); usually formed by several panels and horizontal elements connected to each other so that they represent a single structural part, even if it is joined, and not independent of the whole structure. In terms of FEM element types, the models can be distinguished according to the following details: 2D or 3D alternatives: two-dimensional (2D) or three-dimensional approaches are available, adopting a one-dimensional frame, two-dimensional shell or three-dimensional brick elements, or a combination of these. Shell elements produce faster and more controllable patterns due to the presence of fewer joints than brick elements. On the contrary, the model with brick elements allows the visualization of the stress evolution inside the structure. Nevertheless, the results obtained in both types of analysis are similar, both in terms of structural deformations and stress distribution.

2.2.6.3. Mixing

By increasing the elementary elements, the reliability of the result is strongly influenced by the solution of the approximation problem; therefore, using a dense mesh may not be the better option. The most appropriate mesh size derives from engineering

judgment, taking into account both the dimensions of the structure under study and the refinement of the expected results.

2.3. Classification of damages in masonry bridges

This section presents some details on the damage classification of a stone masonry bridge. A clear and innovative way is described in "Sustainable Bridges" (2007)[14]. In particular, the general classification of bridge structure failures is shown in Fig. 6.

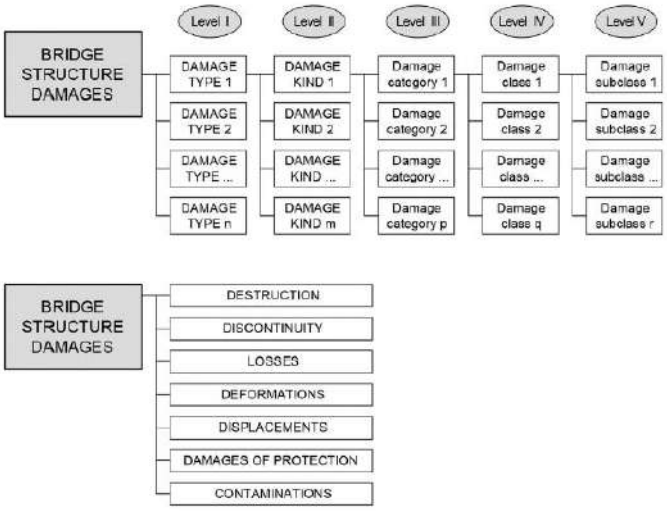


Fig. 6. Bridge structure damage classification (Sustainable bridges, 2007[13]).

The classification according to the location of the damage is depicted also in this book[14]: Fig.7; the fault break classification is also illustrated, in Fig 8; the classification for the type of losses is highlighted in Fig 9; the classification according to the type of deformation is shown also in Fig.10; the classification according to the type of displacement is shown in Fig.11. Finally, some figures deal with the description of common causes of failure and the type of contaminants, respectively.

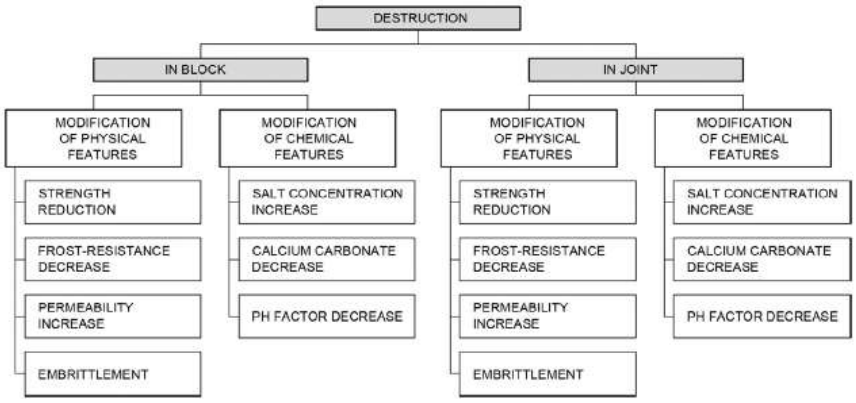


Fig.7. Classification according to the damage localization (Sustainable bridges, 2007[14])

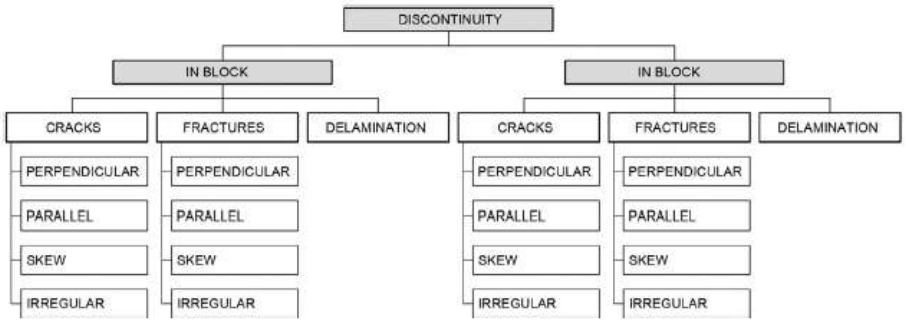


Fig. 8. Classification for damage discontinuity (Sustainable bridges, 2007[14])

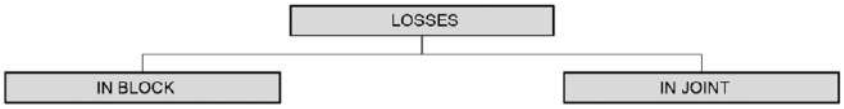


Fig. 9. Classification for losses type (Sustainable bridges, 2007[14])

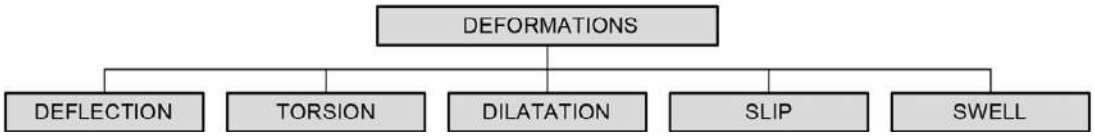


Fig. 10. Classification for damage deformations type (Sustainable bridges, 2007[14])

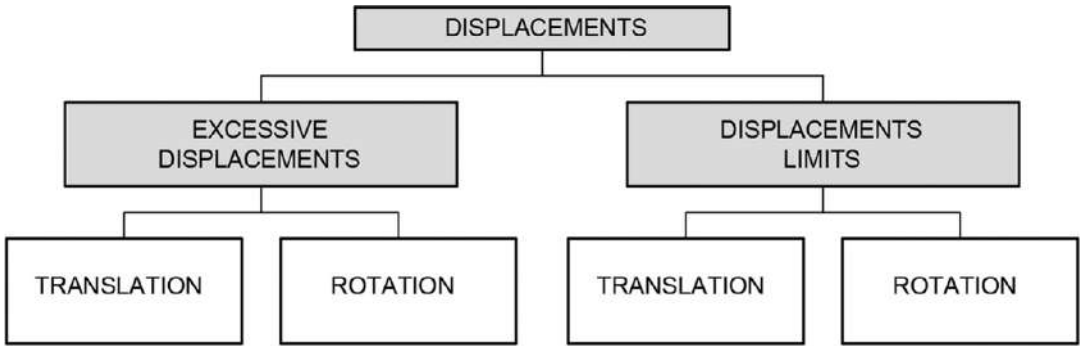


Fig. 11. Classification according to the displacement type (Sustainable bridges, 2007[14])

2.3.1. Frequent failures in masonry arch bridges

Structural defects generally fall into the following categories: - Construction; - Long-term load; - Transient load - Environment -A combination of all the above types of defects can usually be found in existing masonry bridges. Modern traffic loads, heavier than those of the past, could cause an older bridge to show serious problems, while well-maintained masonry arches that are not subjected to heavy loads are likely to be among the most durable structures.

2.3.2. Undermining the foundations

One of the most common reasons for the destruction of masonry arch bridges is the undermining of foundations. However, this type of damage is hidden in the river bed, some elements do worse damage, such as: increase in the flow speed in the river (eg for

environmental reasons) and local disturbance of the flow due to the construction of the pillars. Cleaning problems can be avoided by adding deep footings connected to the existing structure.

3. Some remarks about another modelling approaches for masonry arch bridges according [19]

3.1. Plastic Limit Analysis

Application of the plastic theory is possible for masonry bridges provided that some assumptions have been fulfilled, as stated by Heyman [**20**]. The assumptions' that needed to be fulfilled are: no tensile strength in the arch; the masonry in the arch is incompressible; and sliding between masonry units cannot occur. One of the benefits of this procedure is that one needs not to know the initial stress state in the bridge, only a few material characteristics are needed, and only the ultimate state of the bridge is elaborated. In order to obtain the correct load factor in plastic analysis, several conditions have to be fulfilled. First of all, equilibrium criterium has to be satisfied, meaning that internal bending moments must be equated by the external load. Secondly, the yield conditions have to be satisfied, meaning that no point in the arch should have a moment that is larger than the plastic moment ($M \leq |M_p|$). Considering other internal variables this basically means that the stresses in the material must be everywhere less than or equal to the material strength (e.g. shear, crushing and tensile strength limits must all be respected). At the final stage (collapse), the arch will transform into a mechanism. And finally, the energy which is being dissipated has to be positive, which means that the work done by the forces at the boundary at the plastic hinge has to be negative. In order to obtain the correct load factor in plastic analysis, several conditions have to be fulfilled. First of all, equilibrium criterion has to be satisfied, meaning that internal bending moments must be equated by the external load. At the final stage (collapse), the arch will transform into a mechanism. Regarding masonry arch structures it is necessary to determine the thrust line at each cross-section. If the trust line is within the masonry arch this indicates that it is tension-free. On the other hand, the possible location of the plastic hinges has to be determined. Several authors developed different numerical solutions based on this method [20,21]. Numerous researchers used this method to study stone bridges (collapse mechanism and limit behavior) exposed to dead and live loads, and at first, only vertical loads were considered. Clemente et al. 1995 [19] concluded that if only dead loads are acting on the bridge the safety check is a function of the arch thickness. The work was extended to the dynamic behavior of stone arch bridges and seismic actions [19]. Here the comparison is done between the expected Peak Ground Acceleration (PGA) with the acceleration value necessary to turn the structure into a mechanism. As various assumptions are considered here (impact between the stone blocks is neglected, infinite strength of the material is considered) this represents only a first approximation of dynamic response. Alexakis and Makris [19] concluded that directions of the rapture in the arch once it translated into a mechanism have a direct impact on the location of the imminent hinges and the minimum thickness of the arch. The main advantages of this method are its simplicity and easy application by engineers in practice, the solution is straight forward, meaning that it can be easily checked, and it is very fast, meaning that it is economically feasible. The drawback is connected to several assumptions. First of all, the compressive strength is always a finite and never an infinite value. To overcome this issue the thrust line was replaced by the trust zone [19], which is of sufficient depth to carry the load at each cross-section. This has led to its application in practice and being adopted by the Department of Transport in the USA as the primary means for the assessment of arch structures. The trust zone refers to a

specific cross-section having adequate depth to withstand the applied loads. Further developments were done by Taylor and Mallinder [19] with the introduction of the parabolic stress-strain constitutive relationship for local crushing of masonry at locations of hinges. Brencich and Di Francesco [19] introduced an elastoplastic iterative analysis, with no tensile restraints. Obtaining the ultimate strain represents the collapse conditions. This procedure lies in the conservative part due to the assumptions that have been made and the fact that it is a 2D model. The involvement of the spandrel wall and the arch barrel-fill interaction were not taken into account, and it is well known that they have a beneficial effect on the behavior of masonry arch structures. The proposed method was elaborated on several examples including a railway bridge with six spanes know as Cantalupo viaduct (Fig. 12) and good agreement was noted [19]. This method was able to take into account the interaction between the spans of a multi-span masonry arch bridge.

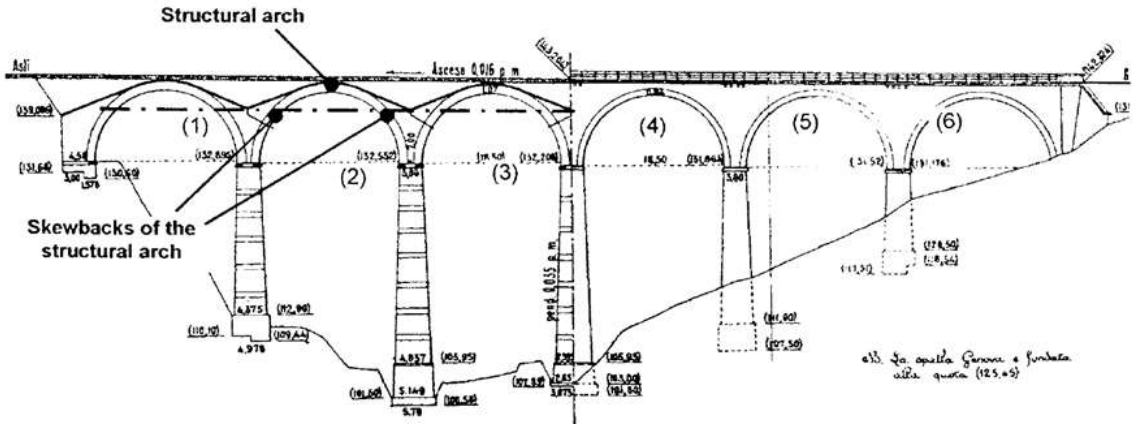


Fig. 12. Cantalupo viaduct on the Genova-Ovada-Aqui Terme line at km 13.549 [19]

3.2. Finite element method

De Felice (2009) [19] for the first time employed the non-linear beam elements with a fiber-cross section in the assessment of the load-carrying capacity of multi-span masonry arch bridges (Fig. 13). The arch is discretized into straight-lined beams and each beam is subdivided into longitudinal fibres, and their stress-strain behavior is defined in relation to the material properties obtained by experiments. The non-linear constitutive relation of the beam section is derived by integration of the stress-strain relation of the fibres [19], as well as taking into account the effective ductility capacity of masonry under compression.

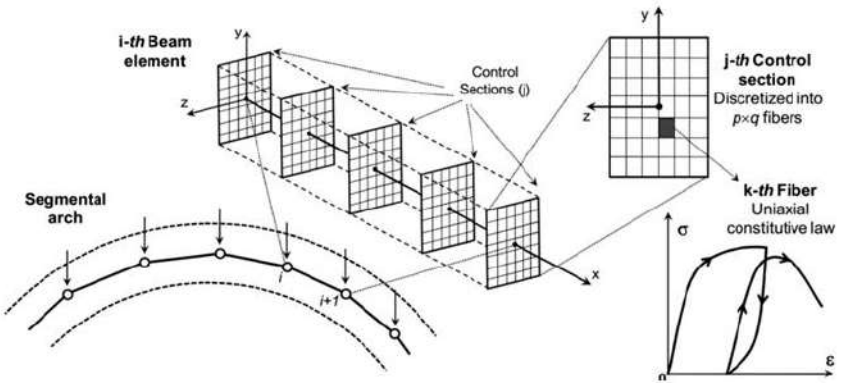


Fig. 13. Fibre beam model [24]

This method was used for an assessment of two and three-span arch bridges and compared with the results obtained by the limit analysis. Excellent agreement was obtained for the obtained level of the load-bearing capacity and the failure mechanism. Taking into account the limited ductility capacity a reduction in the maximum load was observed, once again indicating that the limit analysis is too conservative. It has been shown that the computational time is low, while the interaction among the spans is enabled. This method proved to be an excellent tool for a preliminary assessment of railway structures and showed its benefits regarding computational time and simplicity. De Santis and de Felice (2013) [19] assessed 50 large-span railway brick masonry bridges in Italy. This method was further applied to study its dynamic behavior [19]. The method was applied to an existing seven-span railway masonry viaduct for determination of its seismic capacity (Fig. 14).

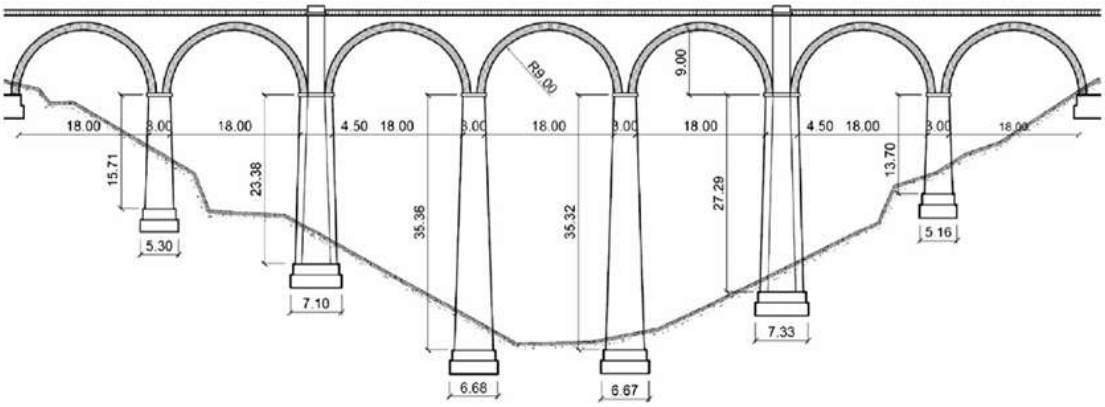


Fig. 14. Ronciglione viaduct [19]

It was observed that the application of this methodology was beneficial for computational time and can be applied for preliminary assessment of the seismic capacity of multi-span masonry arch bridges. The application of 2D and 3D elements for modeling of masonry arch bridges enables a better description of the geometry, infill-material characteristics as well as spandrel walls. In this respect, the number of the input parameters increases, difficulty in the correct definition of the boundary conditions arises and the computational time highly increases. As well, calibration is highly dependent on numerous uncertainties. The development of 2D models was mainly connected to the adequate representation of the in-fill material and the contribution of the spandrel walls in the load-carrying capacity and the structural response.

4. Conclusions

The history of the bridges of Bulgaria is both rich and interesting. Engineers and designers have always been required to solve technical and engineering problems while at the same time producing structures that were in keeping with the every city's architectural context and tradition in Bulgaria. A number of assessment methods are available and are reviewed briefly in [11,12,18,19-23]. For any method to accurately predict the load carrying capacity, the most important factor is the allowable tensile and compressive strengths of the masonry. From the review it is apparent that the ratio of the adopted tensile stress to compressive stress for prediction of the load ratings as well as ultimate carrying capacities of the bridges are differing even by same investigators. For the assessment of an existing masonry arch bridge the procedure will be in future proposed[1-

23]. Here is the place to note the merit of Prof. Van Bogart Philippe from Ghent, Belgium, who did an extremely great service to the Bulgarian engineers by preparing a brilliant solution for them on the bearing capacity of the Devil's Bridge over the Arda River in Bulgaria[18](Fig.15a,b,c).

a) 

Fig. 15a. Dyavolski most

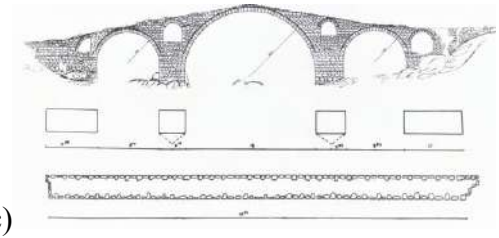
b)  c) 

Fig. 15b,c) General view during flood in Dec.2021, from the side of,(Photo courtesy, Yanko Petkov); b) schemes of Devil bridge

"Devil bridge" is an arch bridge over the Arda River (Fig.15a,b,c.). It is situated in a narrow gorge near to Ardino in the Rodopi Mountains and is a part of the ancient Roman road connecting the lowlands of Thrace with the north Aegean Sea cost trough the Makaza pass. The bridge was constructed in the period of 1515 – 1518 by the by the famous local builder "Usta" Dimitar form Nedelino. The bridge was proclaimed a monument of the culture in 1984. The structure is a close-spandrel deck-arch stone bridge with a total length of 65.70 m and includes three main spans of 9.00 m + 18.20 m + 8.50 m. Additional four spans help to capture the capacity of the spring high level of the river water. They have varying spans from 1.85 to 3.52 m. The middle arch rises 8.20 m above the ground. All arches are approximately semicircular shaped and have a total width of 3.55 m. The bridge deck is 3.40 m wide and has symmetrical longitudinal slopes of 18‰. The stone parapet has dimensions of 200x120 mm. The main arch piers have dimensions of 3.30x3.35 m and 3.66x3.35 m. The pier and the abutment are bedded on the solid gneiss rocks. The bridge structure survived one of the strongest earthquakes in Bulgaria at 1909 with a magnitude of 5.9 on the Richter scale and epicentre in Yambol (HIBE 2014, W.F.Chen and L. Duan).

The basic model consists of nonlinear material and is modeled by using plate elemenets, as in Fig.16. Also the piers were modeled with system of vertical and horizontal intersecting plates, which have detect whether such a type of model would be able to simulate the real behavior of the pier. Apart from this a solid non linear model was made including 0,2x0,2m stones of f_c variable values up to 8 MPa and f_t=0,1Mpa.

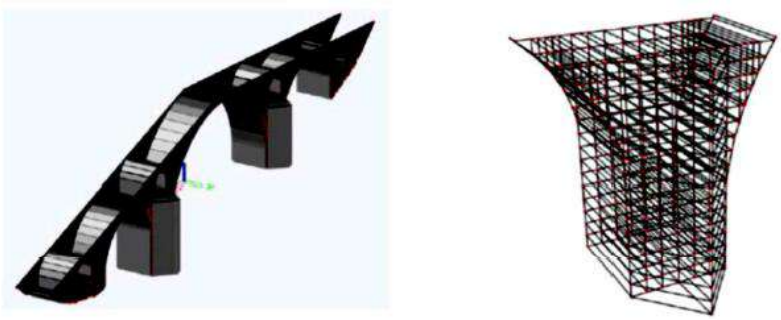


Fig. 16. Plate model overview (left) detail pier (right)

The plated model of Fig.16 demonstrates that the horizontal failure load, due to wave impact does not continue to increase with the compression strength (see graph of Fig. 17). As this process continues the pear may collapse either by overturning as it rotates, or sliding through shear.

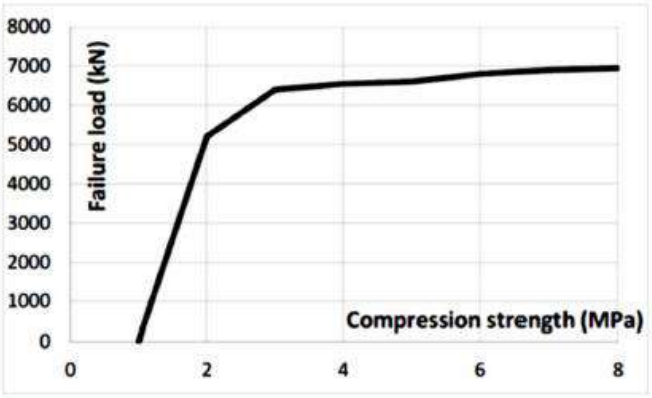


Fig. 17. *Failure Load as a function of compression strength*

According [18] the plate model allows to verify these situation by manual calculation. The external forces can be recalculated to a critical moment M_{cr}, corresponding to the overturning conditions, which occurs at excessive compression and cracking. The graph of Fig. 18 exhibit both manually determined and simulated values of the bending moment to be practically identical. Hence this type of simulation leads to the conclusion that failure is due to excessive bending.

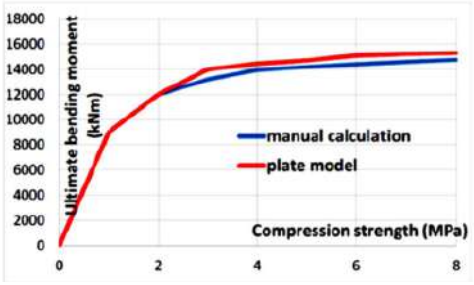


Fig. 18. Failure bending moment as a function of compression strength.

According [16], in addition if manual calculation of shear stress is made through the use of the formula: $\tau = c - \mu\sigma$, taking c=0,2 MPa and μ= 0,3. If the load reaches 3-times the nominal values, two lines can be drawn as in the graph of Fig. 19. The upper one is found by the model of the plate elements, whereas the lower line is calculated by hand. This

graph demonstrates the plate model is incapable of detecting corrected failure by shear. This is definitely due to the use of the Mohr-Coulomb criterion.

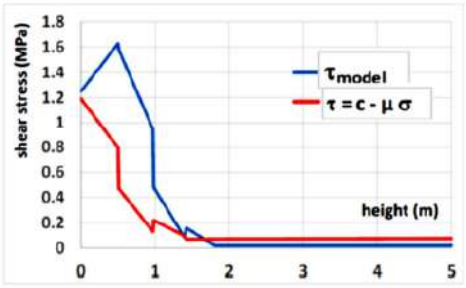


Fig. 19. Shear stress from plate model and calculated by hand

According[18] , therefore two types of models (Fig.20), with solid 3D-models have been assembled. Several materials laws have been implemented. The result are compared with the outcome of the plate model. The first modification is shown in [18] as Fig. 21, by the macro model with solid element. This gives a failure force that is 25% lower than it was found with the firs method. A further improvement is the consideration of strain softening. This reduces the failure force by another 28% or up 1,47 times the nominal force. Finally, brittle failure is also taken into account and collapse horizontal force of 5,6 m wave impact, further decreases to 40% of the value found with the plate model. As a consequences, according[18], this more elaborate simulations clearly indicate the failure of the pier, subjected to lateral load is due to brittle failure by shear. Therefore the conclusion that care should be taken to repair the masonry joints is strongly advisable.

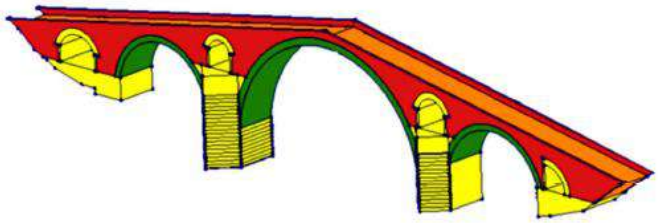


Fig.20. 3-D macro model

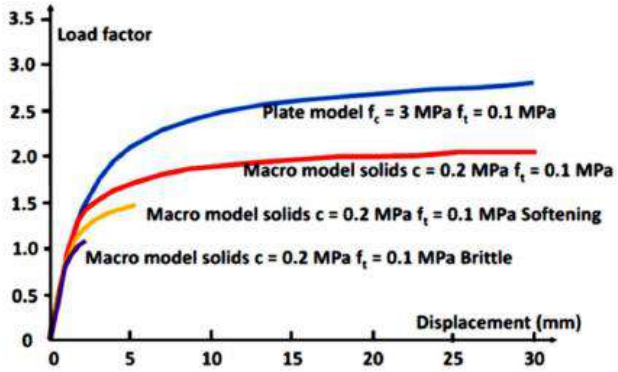


Fig. 21. Failure load factor and displacement of various models

Further refining, according [18], of the simulation went through implementing a meso-model, including interface layers and softening , as well as brittle fracture detection. The results are rather similar to the macro-model, although the failure load is slighted lower, and failure occurs by brittle fracture due to shear. Fig.22 displays the last

converged step of the numerical process, including the crack in the tensile part, the reduced cross section and the skew pattern which leads to shear failure.

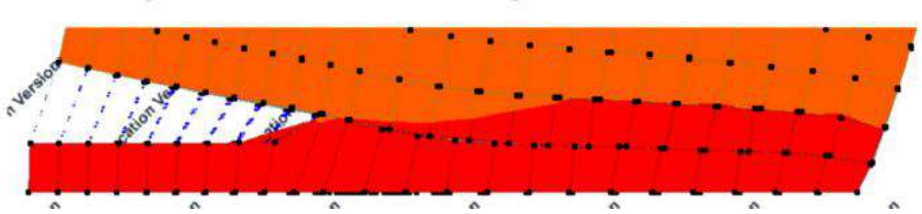


Fig. 22. Cracking before shear failure at the pier base

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PROBLEMS BY RESTORING, STRENGTHENING AND REHABILITATION OF THE SUPERSTRUCTURE OF THE STEEL BRIDGE OVER THE ISKAR RIVER NEAR VILLAGE KUNINO

Peter Velev [1], Ivan Yakimov [2]

Abstract: *The bridge over Iskar river near the village of Kunino is built up in 1936 following the idea of an Initiative Committee, with the funds and the voluntarily work of Kunino's villagers. It replaced the unsafe ford across the river that often took casualties. The exploration before the project of rehabilitation found that the piers and the abutments made of rectangular masonry are in good shape and the foundation did not reveal any defects during the 75 years long exploitation. As about the supreme structure, which consisted of hinged riveted steel trusses with lower road slab – 3 hung x 22 m and 2 double-cantilevered x 31 meters. The end pair of "hung" cantilevers on the left-bank abutment was hardly damaged by a transport accident and "temporarily" replaced by HEB-beams supported on a "temporary" pier. Due to the hard corrosion of the verticals and diagonals of the truss, the rotten cover of the "temporary" supreme structure at the left-bank abutment and most of all the due to the construction of water power plant "Kunino" an urgent reconstruction and rehabilitation of the bridge became necessary fully with the funds of the investor of the WPP. This paper treats the specifics of the design and the accomplishment of the bridge reconstruction.*

Key words: *restoring, strengthening , rehabilitation , old steel bridge*

1. Introduction

1.1. A brief history of the facility.

This bridge was built on the idea of an Initiative Committee from the village of Kunino with the financial ones funds and with the personal labor of the people of the village back in 1935. In two memoirs about the village of Kunino there are valuable and curious facts about the construction of the bridge [1]. It replaced the precarious ford across the river, which often claimed human casualties. The initiative committee has chosen the type of bridge after looking at several ready-made facilities. Along with this, a construction manager was hired, as well as several craftsmen who, together with local craftsmen, built the lower construction of the bridge. The metal superstructure was delivered and installed a few years later.

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Fig. 1. View of the bridge from the left bank

In 1984 during the construction of the new route of the Mezdra railway line - Cherven Bryag, the superstructure in the extreme left bank opening of the bridge was destroyed when the "Kraz" mobile crane passed by. A temporary bridging was made with the construction of one additional pillar from old rail traverse units without a reliable foundation. On the left bank abutment, the temporary pier and the first pier, double "T" steel beams are arranged, on which lie wooden sleepers serving as a roadway. Subsequently, the temporary pillar was additionally clad in a reinforced concrete shell, and a reinforced concrete road slab was built on the traverses (Fig. 1).

1.2. Brief description of the structural scheme of the bridge before the reconstruction.

Lower construction - the foundations were not revealed when photographing the structure, therefore, it can only be assumed that flat foundations were completed, on which pillars and piers of square masonry were built.

Upper building-the scheme of the top structure is gerber beam-two parallel Fifthful vertical steel noveled farms with the road below, with openings 21.92+22.60+22.09+22.60+21.92 (Fig. 2). The transverse axle distance between the farms is 405cm. Two pairs of farms are more height-260 cm-and longer length-31.64m, and three pairs of farms are 190 cm high and respectively smaller length-for the end fields 17.40m and 13.05m for the middle field. The longitudinal farms at the lower farm connections are connected to each other by transverse farms located below the road plate. Together with the stability vertical brackets, they form transverse P-shaped frames (Fig. 10). The road plate is reinforced concrete, supported by longitudinal steel beams without connection between steel and concrete. The road plate is without drains, without waterproofing and without curbs, with joints in the "gerbers joints" without constructive shaping. Under the entire length of the bridge, a horizontal stability farm has has been developed with crossed diagonals. The construction is propped up in six pairs of tangential camps, as at the survey has not been established, which are stationary.

1.3. Issues that require recovery, reinforcement and remediation on the top structure.

The facility is located on a municipal road IV-13411 between the village of Kunino and Stoyanovtsi village. The bridge was mainly used by agricultural machines and animals (cows, buffalo sheep and goats), as a result of which, along with their constructive features

(drainage, joints, nitted constituent rods) lower - unprotected part of the metal Construction has been subjected to the aggressive impact of stuck mud, enriched with nitrates. In the main part, verticals and diagonals at the lower key plates were heavily corroded and even three of them completely interrupted (Fig. 3 and Fig. 4). The characteristic of the nitly structural elements swollen corrosion (Fig. 5 and Fig. 6) was also observed. Several of the verticals and diagonals were severely distorted as a result of hits from agricultural machinery (Fig. 7). Some of the pressed elements had been Dilled (Fig. 8).



Fig. 3. Dashed vertical corrosion in lower belt



Fig. 4. Interrupted section of a diagonal



Fig. 5. Swelling



Fig. 6. Swelling corrosion in girdle



Fig. 7. Impact marks



Фиг. 8. Deplaned upper

In connection with the construction of the "Kunino" MWPP - the facility turned out to be the only one connection of the construction site with the on-site concrete unit, as well as with the republican road network. It was necessary to transport over 40,000m3 of concrete, 460t of reinforcement, formwork, technological equipment and construction machinery (some mobile cranes weighing over 60t).

1.4. Conclusions of the prepared expertise

The investor commissioned the preparation of a technical expertise on the state of the bridge, for the need for emergency repair of the facility and the possibilities for its implementation, which in conclusion states:

− the condition of the two abutments of the bridge, with the exception of their sub-bearing platforms, is good. Repair works are needed to shape the transition plates, expansion joints, slopes and rehabilitate the camp sites;

- all pillars, including their camp sites, are in a visible lot good condition;

− the temporary additional pillar at the left bank weir, made in violation of all building regulations, should be completely demolished;
− renovation and strengthening of the main supports is necessary;
− it is necessary to restore the damaged superstructure in the final opening;
− the road plate is in good condition.

In conclusion, the expertise found that the bridge can be restored, strengthened and rehabilitated on the basis of a relevant project.

2. Exposure

2.1. Establishing the mechanical and characteristics and chemical composition of the input materials

To establish the mechanical characteristics and chemical composition of steel Specimens were removed and tested. Determination of mechanical characteristics was done at the Independent Construction Laboratory "Infrastructure" EOOD. The results are as follows:
− yield strength Re=230/291 MPa;
− tensile strength Rm=371/372 MPa;
− relative elongation after destruction A=36.0/36.5%.
The results showed that the steel met the requirements of S235.
The determination of the chemical composition was done in the Testing Laboratory of materials and calibration at the Institute of Metallurgy at the BAS.
The results are in the following:
− carbon content C 0.026%±0.005;
− silicon content Si ≤0.004%;
− manganese content Mn 0.260%±0.004;
− phosphorus content P 0.035%±0.003;
− sulfur content S 0.031%±0.003. Chemical analysis led to the conclusion that the steel was weldable.

2.2. Selection of calculation scheme

Due to the peculiarities of the construction details, the calculations were carried out with continuous belts with either rigid or hinged filler rods. Done there were also several variants with the discrepancies between the system and construction axes. With separate detailed calculation schemes, the bearing capacity (critical load) of the component pressed diagonals, as well as of the upper and lower girders of the main girders, was determined.

2.3. Problems in determining the stress state of the structure

In broken bars, even after their restoration, there is no shearing effort from the structure's own weight. A similar effect is observed in the rehabilitation of corroded bars when parts of the section are removed and new ones are added in their place. When reinforcing filler bars or upper and lower girders in the added part of the cross-section, stresses from the weight of the structure are also not obtained. The exact static solution is obtained after repeated calculation of the structure under different static schemes, loads and different sections of the elements. An essential feature is that it is necessary to collect stresses at points and not shear forces at sections. The sequence of restoration and rehabilitation is also important, which connects the static task with the organization and execution of the construction. Particular attention was paid to the technological order when welding elements under mechanical tension to avoid excessive heating, so that the E modulus of the steel would drop and, as a result, uncontrolled displacements would occur.

2.4. Problems in drawing up the details of restoration and reinforcement of the bars and nodes.

When restoring and strengthening the rods and nodes, the aim was to preserve them in to the maximum degree the existing structural and nodal axes (Fig. 9). Swelling corrosion is a problem in riveted constructions, which is why special attention was paid to the watertightness of the composite sections. The swelling corrosion was removed by chemical and mechanical interventions, after which the joints were filled with polyurethane paste. Practically all the work on the restoration and strengthening of the metal structure was carried out without stopping traffic on the bridge.

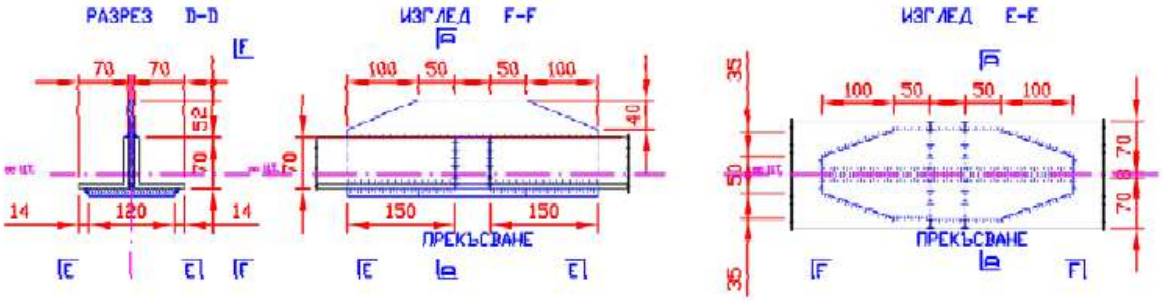


Fig. 9. Reinforcement of a rod composed of two closely spaced corner profiles

2.5. Restoration and reconstruction of the road slab and abutments

To protect the lower parts of the longitudinal girders from atmospheric water and aggressive mud, consoles and boards were formed, as a continuation of the road plate (Fig. 10). Reinforcement from the road plate passed through the "old new concrete" joint, which made the connection to the board and the console. The expansion joints of the road slab, ensuring their water tightness. To prevent the ingress of water through guard ballast walls were made monolithic transition plates and the expansion joints were formed. The drains are installed in the axis of the bridge in about 11m. The reinforced concrete pavement was completed with welded meshes Ø5, 20/20cm.

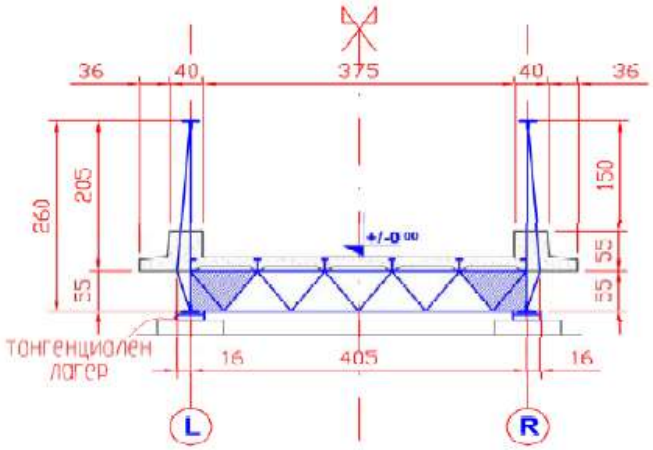


Fig. 10. Cross-section after the reconstruction

2.6. Problems in the joint work of steel and concrete in boards

The existing road slab resting on the cross trusses worked quite independently of the metal structure. After the reconstruction, the boards with transverse section 40/35cm

already cover the filling rods of the longitudinal trusses on both sides, preventing their free movement from impacts, including from temperature differences (with solar radiation, the upper belt has a temperature of 50C°, at air temperature, on board and in the lower belt 35 C°). This effect was taken into account when sizing the metal structure. As a result of under estimating the problem of relation to the reinforced concrete board, cracks appeared around some diagonals.

2.7. Restoration of the structure in the damaged field

The restored superstructure consists of two end longitudinal beams - steel profiles HEB 600 and two middle longitudinal beams – steel profiles HEB 550. Dowels from U profiles are mounted on the upper girdle of the beams. The end longitudinal beams are located in the structural axes of the existing two longitudinal bar structures. Above the left bank weir, the longitudinal beams are carried by a cross beam - steel profile HEB 600, resting on two rubber bearings. Above the end support post the reinforced longitudinal nodal plates of the rod structure serve as longitudinal beams, and the reinforced transverse truss serves as a support for the middle longitudinal beams. A reinforced concrete plate is cast on the longitudinal beams, which forms a steel-reinforced concrete section in this opening by means of dowels (Fig. 11)



Fig. 11. Connection of the end longitudinal beams with the verticals and the diagonals of the longitudinal trusses during road slab reinforcement

3. Conclusions

Even with very serious damage, old bridges can be repaired and should be preserved, if only for their historical value. Reconstruction and rehabilitation projects, as well as their implementation, require high competence and responsibility. Even in very difficult situations, such technological solutions could be found that reinforcement and rehabilitation can be carried out without stopping traffic.

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CONCRETE TIED ARCHES FOR MEDIUM SPAN BRIDGES FROM LAST CENTURY

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***Abstract**: In the past century, in Belgium, concrete tied arch bridges have been built regularly to cross small rivers or roads. This type of structure was used for crossing spans, varying from 30 to 60 m. The examples being presented demonstrate that these structures can no longer be built in an economical manner, but also confirm their heritage value. The shape and characteristics of five types of such bridges are discussed in detail. Most of these bridges are well documented, principally by drawings. It becomes clear that none are identical. Many of them may be demolished in future. Because of the architectural, technical and cultural value of these structures, they should be protected and maintained, since they testify of the skills that previous generations were able to use. Indeed, the arch shape appeals to the human spirit through the balance and harmony it emanates.*

***Key words**: Concrete arch bridge, arch composition, bridge preservation, concrete hinges, rocker bearings, concrete hangers.*

1. Introduction

In the past century, in Belgium, concrete tied arch bridges have been built regularly to cross small rivers or roads. This type of structure was used for crossing spans, varying from 30 to 60 m. Today, bridges with this span would probably use precast prestressed concrete beams with an overlying concrete slab deck, as shown in Fig. 1. Thanks to the wide possibilities of extensive prefabrication, this is a fast and economical alternative. However, the superstructure of such bridges has little character and offers a rather low-profile appearance, without much added value. Designers do try to develop alternatives where some research is done, but these are only isolated attempts. One exception now is the use of extradosed posttensioning as in the Bergwijk Bridge [1], located to the South of Ghent, exhibited in Fig. 2.

But it can hardly be said that these forms can compete with the arch shape [2]. The latter has a harmonious appearance and appeals to the human spirit through its elegance and visibility. The classic arch is located below the roadway. This shape is not interesting in mainly flat landscapes. For this reason, the tied arch is used, where one or two arches are placed outside the horizontal clearance of the roadway. Traffic then drives over a bridge

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deck that is suspended from the arches and serves as a tensile element that connects the two springs of the arches. The actual tensile elements are the edges of the bridge deck, mostly called the lower chord. A balance exists between the required stiffness of the arches and the lower chords. The sum of these bending stiffness values remains constant, but may be distributed among the arch and the chord in various ways. The following 5 examples of medium span bridges illustrate the quality and value of concrete tied arches.



Fig. 1. Precast prestressed girder bridge



Fig. 2. Extradosed posttensioned bridge

2. Albert 1st Bridge

This bridge allows to a small road, connecting the municipality of Schaerbeek to the town of Vilvoorde, to cross important railway lines from Brussels to the North and East of the country. It connects the 'Rampe du Lion' to the Zenobe Gramme lane and is located to the North of Brussels. It was built between 1923 and 1925 to replace a metal bridge from the beginning of the 20th century, of which it reused the abutments and the two central double piers, in coated masonry and blue stone. The classically inspired style of these reuses has also permeated certain new parts (piles, corbels, parapet). It is a tied arch reinforced concrete bridge, with two sections of three spans of unequal lengths (from 26.4 to 34.4 m), whose six arches relieve the two main lateral beams of the continuous deck. The bridge is exhibited in Fig. 3, seen from the North-West side. The whole is supported, by a system of rollers and joints, on the old supports and on four pairs of new square piers. The sidewalks are supported by ogee corbels and provided with a thin solid concrete parapet. The splitting of the central piers of the previous bridge allowed the construction of an earth ramp towards the railway tracks.

A remarkable characteristic is the cross-section of the lateral arches, which has an octagonal shape, which is rarely found. The octagon has overall width and height of 0.70 m. The value of the arch rise to the span varies from 0.174 for the 34.4 m spans to 0.189 for the smaller 26.4 m spans. This implies the arches cannot be linked by an upper bracing

and must have sufficient lateral stiffness to prevent any lateral buckling. The hangers, connecting the arches to the lower chord girders are in RC as well. These hangers have variable width from 0.5 m at the arch connection to 0.7 m, which corresponds to the 0.7 * 1.28 m high cross section of the lower chords. The hangers have lower stiffness in the plane of the lateral arch, corresponding to a width of 0.30 m. Hence, the hangers effectively contribute to the lateral stiffness. The runway slab is just 0.15 m thick and is supported by crossbeams of 0.4 m width, located at each hanger and on both sides of the solid arch springs. The static schema is rather peculiar, since each group of 2 spans is continuous. Both group ends allow horizontal displacements, since these are supported by roller bearings. The internal supports are plain mortar.



Fig. 3. Albert bridge

This bridge is reasonably well documented, since 4 original drawings dating from 1922 were found. Unfortunately, no calculations documents could be retrieved. The first drawing, is a general overview with some of the main dimensions. The second drawing combines reinforcement data and concrete dimensions of the Western group of 3 arches, whereas the third drawing gives the same information for the East side group. Finally, the 4th drawing contains all details, including reinforcements and overall dimensions. The arch cross section and spring are exhibited in Fig. 4.

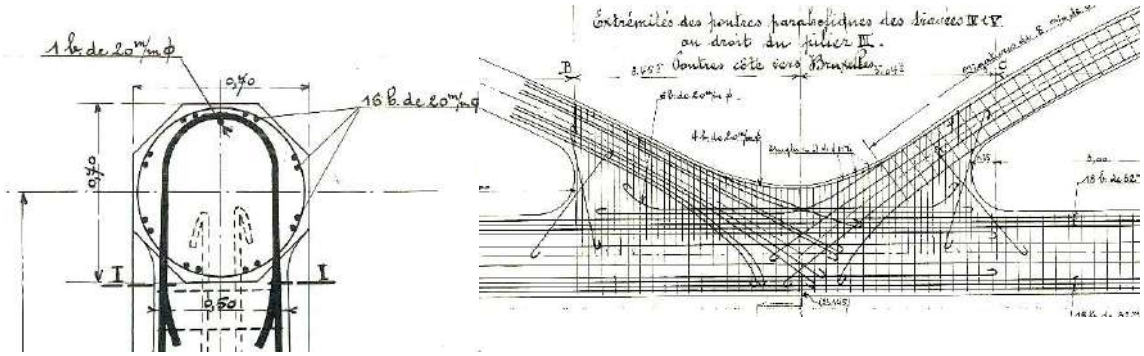


Fig. 4. Details of arch from original drawings

Presently, the bridge is in a poor state. Some repairs were already carried out in 2008 to prevent concrete fragments from coming loose. This poses a danger to the trains running under the bridge. The crossbeams and deck in particular seem to suffer from the

deterioration of the concrete. In 2013, the movement of trucks and buses was banned on the bridge. As a result, some bus journeys had to be adjusted. Since 2021, attempts have been made to abolish the bridge's heritage protection [3]. The owner maintains his intention to replace the bridge with a light modern metal structure. In the opinion of many observers, the damage and poor state of the structure is due to insufficient maintenance and care. This is the most effective way to endanger heritage structures.

3. River Emu bridge

This bridge, completed in 1940, carries the Bass Highway across river Emu at some 2 km from the town of Burnie, state of Tasmania, Australia. This small bridge has a span of 32.92 m, the arch rise being 6.06 m. Except near the arch springs, the lower chord measures 0.508 m by 1.291 m. Twelve hangers of 0.305*0.508 m connect the arches to the lower chords, the 2 outward ones being integrated with the arch springs. The largest stiffness of the hangers is in the out-of-plane sense. In 2010, while preparing some refurbishments [4], the question appeared whether an upper wind bracing would have to be added. This proved to be unnecessary, since the lateral stiffness is sufficient. Maintenance and some strengthening of the bridge was carried out in 2019. A global picture of River Emu Bridge is exhibited in Fig. 5.



Figure 5 : River Emu Bridge

Almost complete information was found about this bridge, with 8 original drawings. The first drawing shows the location and overall view, which is depicted in Fig.6. The lower chord continues beyond the river piers in a short 5.064 m span. This introduces an end bending moment at the arch springs. According to this drawing, the rock bed is deeper for one pier, whereas its counterpart required just a shallow foundation.

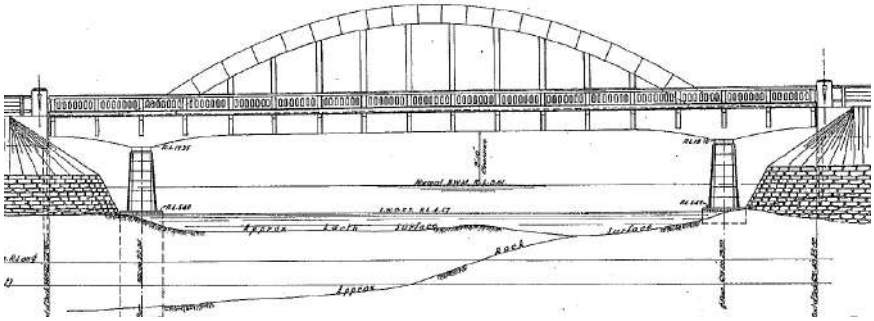


Figure 5 : General drawing River Emu Bridge

A particular characteristic is the apparent central hinge in the arch centre. This is rarely found in concrete structures. What is more, it cannot be observed from outside and probably it does not provide any additional structural value.

Drawings 2 to 4 show detailed concrete dimensions as well as reinforcements of the arches, lower chords and hangers, of the arch springs, of the small corbelling spans. Drawings 5 to 8 concern a central cross section and details of the cross beam reinforcements, reinforcements of the bridge deck slab, details of the central hinge and of the rocking bearings. Fig. 6 exhibits the detail of the hinge, whereas Fig. 7 concerns the bearing.

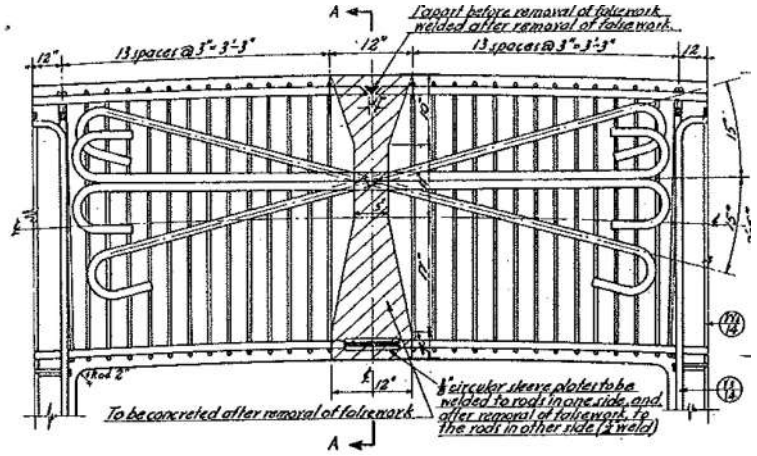


Figure 6 : Arch central hinge

In reality, the central arch joint, has acted as a hinge during erection only. The drawing clarifies the main rebars are joined after building the arch, thus making it continuous. Probably, some uncertainty existed for closing the arch top and this temporary device was considered helpful.

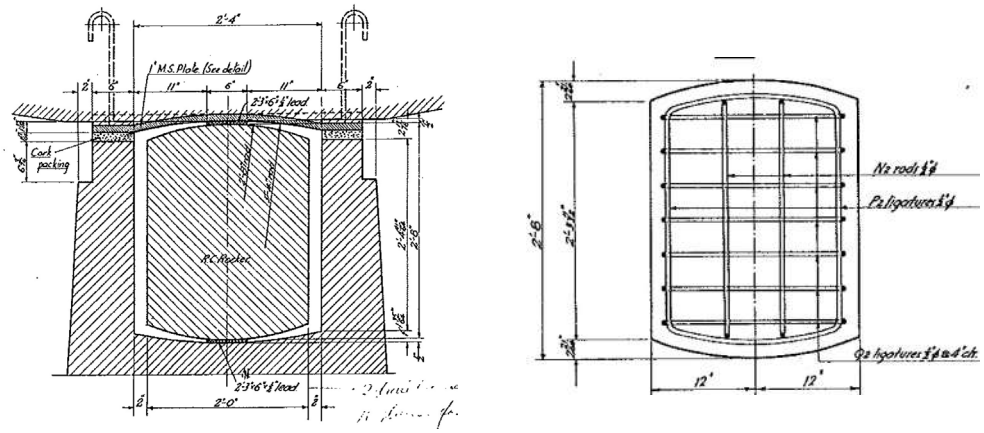


Figure 7 : Rocker bearing

The rocker bearing is entirely of RC, which is rarely found. Mostly these roller or rocker bearings were made of cast steel. It is unsure how this bearing could be designed, since the mostly used Hertz formulas for local contact apply only to steel. Yet, the contact area itself consists of lead sheets, which proves that the contact stress is really low. Nevertheless, this is a rather unique arrangement.

4. Bridges across river Lys

River Lys is part of the fluvial connection between rivers Seine and Scheldt and is being adapted to 4500 tons vessels. The final aim is to divert traffic of goods from the road to inland shipping. Before 1960 river Lys had only a minor importance for shipping. Many of the bridges, crossing this river date from the period between WW1 and WW2. Twelve of these bridges are tied arches, of which 10 are of concrete. Obviously, more of these bridges have existed. Apart from the 2 remaining ones, all steel arch bridges have been replaced within the framework of the Seine-Scheldt project. The idea of building concrete ties arches must have been popular during that time. Some of these bridges have a symbolic meaning, such as the bridge in Oeselgem across the original course of the river, which was abandoned after straightening, called four war fronts bridge [5] [6]. River Lys had an important defence role during the early days of WW1 and WW2. This bridge was dynamited both times and reconstructed. Fig. 8 exhibits one of the finest examples of Kuurnebridge in Harelbeke.



Figure 8 : Kuurnebridge across River Lys

All of the concrete arch bridges have steel hangers, connecting the arches with the lower chords. This does require regular maintenance, especially at the connection with the concrete parts. As displayed in Fig. 9, water can ingress into the joint, corrosion is inevitable and would require a more elaborate arrangement than the simple disposition with fitting tubes, adopted in these bridges.



Figure 9 : Fitting tubes arrangement of hanger connections

No original drawings of these bridges have been found. Therefore it is impossible to comment on further details. Yet, at some of these bridges, near the springs, the arch takes opposite curvature, compared to its normal part. This changes the sign of the outward force due to curvature and requires additional stirrups. However, it does avoid the intersection of the arch and lower chord to be located beyond the bearings, which results in edge moments of the structure.

5. Bridges of railway line Brussels to Antwerp

Similar as for river crossings, the section between Mechelen and Antwerp of railway lines is crossed by concrete arch bridges, dating from the period of 1930 to 1939. Presently still 5 of these bridges exist, 4 of them being of concrete. However, at least one of them may be demolished in future and replaced by a small tunnel [7]. This is the 34.10 m span bridge Spoorweglei in Sint-Katelijne-Waver, exhibited in Fig. 10 with drawing. It is mainly used for access to platforms and thus has become a pedestrian bridge.

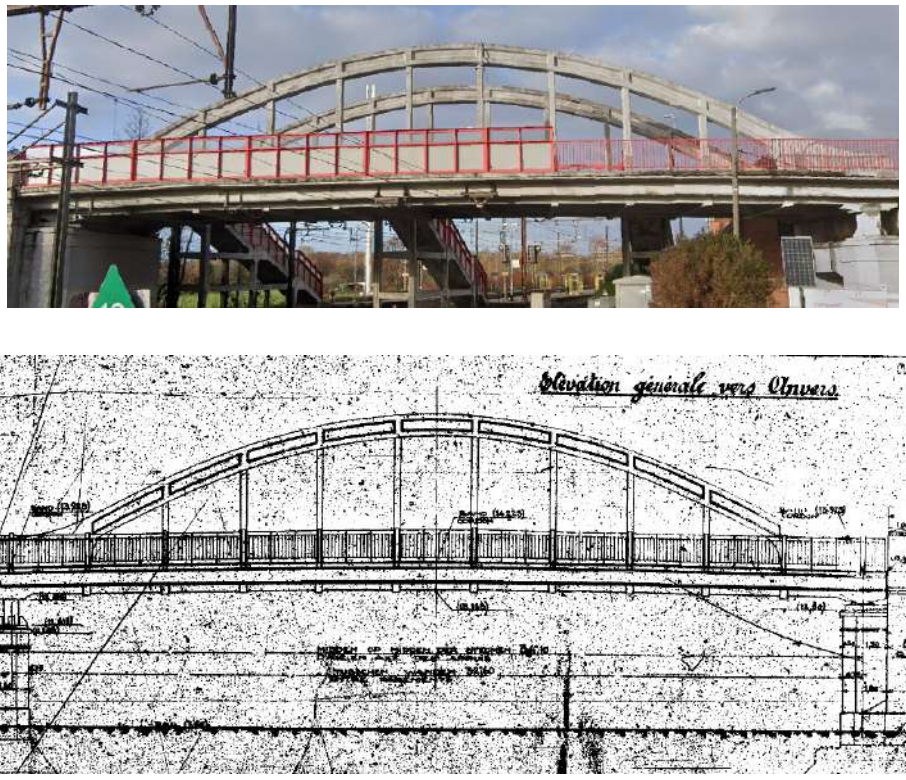


Figure 10 : Bridge at Sint-Katelijne-Waver with drawing overview

Only a few drawings of this bridge were found. The information consists of general views as in Fig. 10 as well as some concrete dimensions steel reinforcement drawings. These were sufficient to build this bridge.

Both the picture and the drawing show a particular development of the arches. These do not have a rectangular cross section but are I-shaped, except for those locations where the concrete hangers are connected. The overall width of the arch equals 0.6 m and is reduced to 0.425 m along most of its development. Apart from some concern to save concrete material, it does not become clear why this arrangement was made. However, it seems to have been applied systematically in this area, since the other bridges at Kontich and Hove also have this special arch cross section. Fig. 11 shows one arch of the bridge at Kontich.



Figure 11 : Bridge with I-cross section of arches at Kontich

6. Bridge at Dendermonde

The largest bridge of 60 m span of these concrete tied arches is found at the town of Dendermonde. It allows the crossing of the road from this town in the direction of Brussels, with the railway tracks of the local station. The bridge is exhibited in Fig. 12.



Figure 12 : Bridge at Dendermonde station

A vast amount of documentation, dating from 1932, was found in the 20 drawings concerning the whole station project, of which 7 relate to this bridge. Archives show no design document. Similar to the bridges of section 5, the arches do not have a prismatic cross section. The concept has been expanded by creating a double arch, connected by small short columns, which coincide with the location of the concrete hangers. The total height of one arch equals 1.85 m, yet both branches are 0.9 m wide and 0.55 m high. However, a small edging of 0.20 m width exists inside the opening. This increases to some extent the stiffness of the columns. The cross section of the separate arch branches and the edging are exhibited in Fig. 13.

The steel bearings are of fixed and rocker type. This allows free longitudinal displacement. The types of bearings are characteristic of almost 90 year old bridges. They are rarely found in modern building, because of the elaborate design and craftsmanship. Today, these bearings are highly valued. Both types are exhibited in Fig. 14.

This bridge seems to be a highlight in interwar period concrete arch building. It is well maintained and will probably continue to serve in many years to come. All particularities of this type of structure are coming together in this rather large bridge. It has no disturbing upper wind bracing, since the lateral resistance of the arches and the contribution of the concrete hangers provides sufficient lateral stiffness.

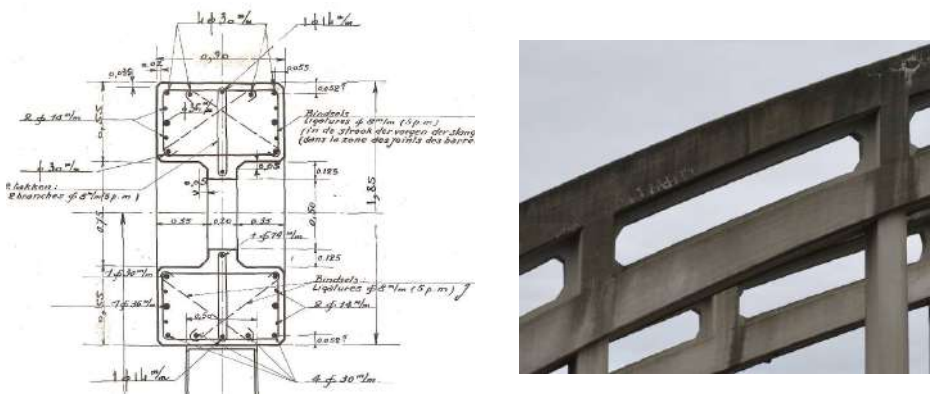


Figure 13 : Arch cross section and edging



Figure 14 : Dendermonde bridge bearings

7. Concrete hangers

A particularly vulnerable part of these bridges are the RC-hangers. In view of their low stiffness the principle stress is tensile, caused by the normal force. Since only 4 of the discussed bridges are fully documented, the following quantities could be determined, as mentioned in Table 1. $A_{hangers}$ is the cross-section of the reinforcement rebars in mm², $B_{hangers}$ is the concrete cross-section area in m², N_{Ed} is the hangers force in kN for ULS, including dead weight and the load due to LM 1, N_{Rd} is the ULS-capacity of the hanger in kN for ULS, σ_c is the concrete tensile stress (MPa) in SLS and the last column indicates whether the concrete is cracked or not.

Table 1 Hangers forces and concrete stress

Bridge	$A_{hangers}$	B_{hanger}	N_{Ed}	N_{Rd}	σ_c	Cracked
Albert	5428	.0.15	750	651	1.557	NO
Emu	3547	0.155	741	775	1.96	YES
SK Waver	4241	0.10	750	713	2.074	YES
Dendermonde	7364	0.30	852	1031	1.123	NO

If the concrete strength today would correspond to class f_{ck} = 25 MPa, which is probably a rather low estimate, the value of f_{ctk} = 1.8 MPa. Whether the hangers have cracked concrete is confirmed if the concrete SLS-stress exceeds 1.8 MPa.

Both the Albert bridge and the bridge at Dendermonde have uncracked concrete hangers, whereas for Emu-bridge and the SK-Waver bridge the hangers must be cracked. Obviously, at the time of the design and construction of these bridges the LM 1 was not in

use. Therefore, these bridges do not comply with the conditions of EN 1991-2, which can be easily understood. However, both bridges with cracked hangers do almost comply with the ULS-requirements. Nevertheless, whether the hangers are steel or RC, they are a main concern in these concrete tied arch bridges.

Conclusion

Arch bridges are now almost exclusively designed for larger spans. The examples given show that arch bridges for moderate spans were frequently designed in the past century. The choice of concrete as a construction material is very logical, because arches are mainly compressed parts. It has also been shown that these arches can take different shapes, either with rectangular cross section, or octagonal and even doubled. This shows that designers could make different choices. It is regrettable that these structures are gradually disappearing, due to new functions, heavier loads or a lack of maintenance. Arch bridges remain aesthetic structures with a powerful shape that add value to the environment.

As far as the present study was able to retrieve the relevant data, the RC-hangers seem to be an important point of attention. Once the concrete is cracked, the deterioration process, due to chloride ingress from de-icing salt, as well as carbonation can start. Therefore, during maintenance, the hangers as well as the connections require special care. In case of steel hangers, the example of section 4 clearly demonstrate the weakness of the hanger connections.

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ABOUT THE 55 YEARS HISTORY OF ZDIAKOV BRIDGE IN CZECH REPUBLIC

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Продължение на втора част:

Abstract: Ждяковският мост след повече от 55 години история продължава да представлява един от най-големите и красиви мостове в Чехия. През 2017 г. се навършват 50 години от пускането му в експлоатация и същевременно с приключена точно след този период успешна реконструкция. Ждяковският мост е изпълнен от стомана като двуставна подпряна дъгова конструкция и все още се класира сред най-големите дъгови и красиви мостове по света.

Key words: *arch bridge, 330m span over Zdiakov dam*

6.4. Подновяване на ревизионната база през 1999 г

Много преди появата на пукнатини в стеблото на главната греда на моста бе известно, че ревизионните пасарелки на моста са напълно нефункционални. За проверка на основните греди на мостовото платно е използвана мобилна платформа. През 1999 г., тогавашното Министерство на транспорта и съобщенията отпуска средства за производството и инсталирането на ново оборудване за проверка на моста. Оборудването за ревизионни прегледи се състои от две платформи, които минават под пътната настилка на моста от опората до горната част на свода и две платформи, които минават по горния пояс на гредите на свода, (фиг. 20 и фиг. 21). Подмяната на инспекцията мостове е извършено от Hutní montage Ostrava през 1999 г.





Фиг.20. Инспекционен пешеходен мост под пътната настилка на моста
Фиг.21. Обозначение на ревизионна количка

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7. Подготовка на реконструкцията:

7.1 Диагностично изследване

През април 2008 г. Проектантската организация, Pontex spol. s.r.o., въз основа на поръчката на ŘSD, беше извършено диагностично изследване на моста - Žďákov [21, 23], което беше основата за актуализиране и уточняване на проекта за ремонт на моста от 1999 г. Резултатите от редовните инспекции на Žďákov мост многократно сигнализира за сериозни щети и нефункциониране на опорите на моста, деградация на бетона в горните части на опорите, корозия на крайните напречни греди, нефункционално отводняване на моста и начална локална корозия на стоманената конструкция. Извършеното през 2008 г., диагностично обследване включва описание на състоянието и дефектите на отделните части на моста и препоръки за ремонт. За стоманените части на носещата конструкция обикновено се препоръчва, само полагането на ново антикорозионно покритие, с изключение на мостовите преходни фуги, които изискват подмяна с нови ламелни мостови затвори. За бетонните части на носещата конструкция и долната конструкция е препоръчително да се извърши допълнителна диагностика и въз основа на нейния резултат да се санира бетонната повърхност. Необходима е пълна подмяна на горната част на моста. Оригиналният дренаж дълго време не е функционирал. *По-специално, надлъжните стоманени греди под пътната настилка на моста са напълно унищожени от корозия.* Това бе и основната причина за корозията на стоманените елементи, които се намират в близост до нея. За елементи на мостово оборудване, т.е. парапети и дренаж, се изисква подмяна, ремонт на инспекционно оборудване и въвеждане в експлоатация на инспекционни пасарелки. Въз основа на резултата от извънредния оглед е посочено състоянието на конструкцията, в следната сигнатура: V – лошо, коефициент на състояние на конструкцията $\alpha = 0,6$ и товароносимост $Vn = 19$ t, $Vr = 61$ t, $Ve = 163$ t. Състоянието на конструкцията е засегнато от лошо състояние на затворите на моста и опората на горната част

7.1.1. Допълнителното диагностично изследване

То е отново изготвено от фирма Pontex spol. s r.o., през октомври 2011 г. в обем от 269 страници [24]. Основата е технически доклад, изготвен на базата на извънреден оглед на избрани части от моста и предложение за ремонт на отделни части от моста. За стоманените части на носещата конструкция предложението за ремонт в общи линии съвпада с предложението от предишното диагностично изследване. За подпорите е предложено разрушаване на горната част на подпорите, а за пилоните и основите на свода е добавен метод за рехабилитация. Въз основа на резултата от проверка на плочата на мостовата настилка отдолу, където бе открито проникването на влага през нея, за мостовата настилка беше предложено да се премахне напълно стоманобетонната плоча по дължината на главните греди на ширина от около 2,0 m.

Освен това бе предложен график за ремонт в три етапа на изпълнение: в рамките на 2 години; в рамките на 5 години; и в рамките на 10 години. Друга важна част от допълнителното диагностично изследване беше изследването на състоянието на съществуващата PKO и корозионна атака, което беше изготвено от инж. Ondřej Kugler, сертифициран инспектор FROSIO [25, 26]. Целта на корозионното изследване бе да се определи действителното състояние на антикорозионната защита и възможните загуби от корозия на отделни елементи на моста Žďákov. Въз основа на установяването на фактическото състояние на стоманената конструкция са предложени мерки за ремонт или възстановяване на антикорозионната защита на отделни елементи. Проведеното

изследване бе високо професионално. Докладът предлага поетапност на ремонтите за защита от корозия: 1 етап – изпълнение до 1 година: затваряне на моста. 2-ри етап - изпълнение в рамките на 2 години: ремонт на ПКО на главните греди на мостовата конструкция и на напречните греди, локални ремонти в кръстовища на усилватели с камерни греди 3-ти етап – изпълнение до 10 години: пълно възстановяване на ПКО на всички стоманени елементи на моста.

В доклада не са отчетени загуби от корозия на стоманената конструкция, с изключение на затворите на моста. Обследването на вътрешността на камерите на сводовите греди не е извършено цялостно. Към доклада на инж.Куглер е изготвена чрез експертна оценка от автора на тази публикация, в която той изразява съгласието си със съдържанието и изложените заключения. Други части от допълнителното диагностично изследване бяха тестове на свойствата на материалите, тест за устойчивост на повърхността на бетонни сондажи срещу действието на вода и размразяващи агенти и определяне на степента на карбонизация и тест на концентрацията на хлоридови йони.

7.2. Проектна документация

7.2.1. Документация на задание за изпълнение на строежа

Документацията за заданието за строителство (ДЗС) е изготвена от Pontex spol. s r.o., през ноември 2011 г. въз основа на част от оригиналната проектна документация, статическото преизчисляване от 1999 г., диагностично проучване от 2008 г. и допълнително диагностично проучване от 2011 г. В преамбюла се посочват причините за ремонта на моста-Žďákov и допълнително се подчертава, че при диагностичното обследване на съоръжението, нешата ще бъдат прецизирани, което вероятно ще доведе до промени по време на строителството. Строежът на моста е разделена на четири относително обособени части в ДЗС: - ремонт и саниране на долната конструкция - ремонт на горната част на моста и мостовата настилка, включително подмяна на капаци на моста - ремонт на свода на ПКО и стойки - мостово оборудване. Освен това съдържанието на дейностите по предстоящата реконструкция е изредено по ключова дума: -Ремонт и рехабилитация на инфраструктурата; - Разрушаване на горната част на подпорите; - Нови части на опорите, - Цялостно бетониране на повърхността на опори и крила с дебелина от 100 до 150 mm; - Цялостно бетониране на повърхността на шахтите на пилоните с дебелина от 100 до 150 mm; Разрушаване на тавана на нишата и новото му изграждане върху главите на пилоните; - Рехабилитация на камери за дренаж в главата и подножието на пилона; - Пълно бетониране на основите на пилон и конзоли под сводовата фуга с дебелина около 130 mm; - Издърпване на крайните части на скобите под съединението на арката с вертикални пръти за предварително напрягане; Рехабилитация на основите за наклон. Ремонт на горната част и пътната настилката на моста: Демонтаж на горната част на пътната плоча на моста, включително парапета и корнизната греда; - Разрушаване на част от пътната плоча върху главните греди на ширина 2,0 m по цялата дължина на моста и последващото й възстановяване; - Инжектиране на пукнатини в пътната плоча, Поддържане на главните греди на мостовата конструкция пред опорите. Ремонт на пространството зад опората: изкоп на почвата, последващо насипване, преходен клин и ново пътно платно. Нов път и изолационен слой (хидроизолация на пътя и проходима изолация на тротоар и двуслоен път). Нови мостови щори (многоламелни водоустойчиви щори с намален шум). Ремонт на стоманената конструкция на мостовата настилка (усилване на местата на корозионно отслабване, пробиване на всички затворени ребра по стеблото на главната греда, евентуално разширяване на отвора, премахване на дублиращи листове, крайни напречни греди). Цялостно възстановяване на РКО стоманената конструкция на

мостовата настилка (сухо абразивно бластиране до ниво на чистота Sa 2½ и четирислойно покритие с дебелина 360 μm.

Ремонт на РКО арка и стойки

Цялостно възстановяване на ПКО по външната повърхност на свода, включително фугата под свода. Локално възстановяване на ПКО по вътрешната повърхност на свода. Пълна реставрация на стойки РКО включително пета и горна става.

Мостово оборудване

Нови корнизи и парапети. Нов дренаж на моста (дренажи, вертикални и надлъжни водостоци, водостоци при пилони). Ремонт на съществуващото инспекционно оборудване (колички за инспекция на арка и мостова настилка). Регулиране на люкове в мостовата настилка и входове към камерите на арките.

7.2.2. Документация за изпълнение на строежа

Не е изготвена документация за изпълнение (РДС) преди началото на строителството, поради ограничение във времето. В хода на строителството е допълнена и уточнена документацията по заданието за строеж въз основа на установеното актуално състояние. По този начин RDS беше постепенно завършен по време на текущата реконструкция. Издадени са специални технически условия за качество [28] за реконструкцията на моста Žďákov, които допълват глава 1 „Общи“ от техническите условия за качество на пътните конструкции с разпоредбата, че документацията за изпълнение на строителството ще бъде извършена въз основа на документацията на ДЗС, допълнена с констатациите от други проучвания, които ще бъдат извършени в рамките на изпълнението на строителството и допълва, респ. разширява резултатите от диагностичното проучване, извършено като част от документацията за обществената поръчка. Същевременно се изготви технически регламент за отделните строителни дейности. В края на реконструкцията е изготвен обобщен заключителен доклад, който се състои от 15 частични окончателни доклада.

8. Протичане на реконструкцията 2015 – 2017 г.

Реконструкцията на моста Žďák стартира официално на 8 юни 2015 г. с участието на представители на ŘSD CR и при голям интерес от страна на медиите [30]. През 2015 г. е изпълнена само част от работата от приетия график поради недостатъчна подготовка на проекта. Извършено е разрушаване на пътното платно и изолационен слой, разрушаване на горната част на опора на левия бряг и премахване на опора на моста, изграждане на нова горна част на опора на левия бряг и преходната му зона.В края на строителния сезон върху откритата бетонна плоча на моста беше оформено временно платно за еднопосочно движение със светлинна сигнализация и временно бе преодолян процепът на мястото на бъдещото затваряне на моста на левия бряг. Временното движение по моста - Žďakov беше пуснато на 19 декември 2015 г., фиг. 22.



Фиг.22. Настройка за временна работа



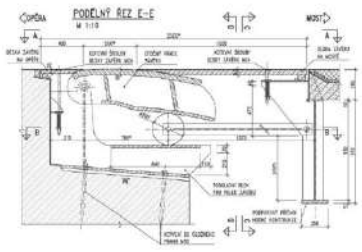
Фиг.23. Ограничено двупосочно движение по моста

Освен гореизложеното, на части усилено се допълваше документацията за изпълнение. Основните проблеми бяха пътното платно и изолационния слой на мостовата настилка, мостовите затвори, модификация на настилката, антикорозионна защита на стоманени конструкции, проектиране на нови корнизни греди и парапети, рехабилитация на опори и дренаж на преходните зони, рехабилитация на пилони и дренаж на моста. Този процес също така включваше технологични и изпълнителни разпоредби (TePř), които бяха обработени от подизпълнители.Процесът на коментиране и одобрение за някои TePř беше труден и следователно дълъг. През зимата на 2016 г. продължи работата по документацията за изпълнение. Фактите, открити в началото на реконструкцията, вече могат да бъдат отчетени тук. Най-важното решение произтича от резултата от теста за разрушаване на плочата на мостовата настилка, извършен през 2015 г., когато част от плочата на мостовата настилка на площ от около 2 m^2 беше съборена и по време на което беше разрушена се установи, че бетонът и армировката на мостовата плоча, съединителните елементи и горния фланец на стоманените напречни греди са в много добро състояние, за разлика от резултатите от допълнителното диагностично изследване от 2011 г. Поради това беше решено да не се кърти ст.бет. плоча по ДЗС. Временната операция беше отменена на 30.3.2016 г. и мостът Žďákov беше напълно затворен. През 2016 г. са извършени по-голямата част от строителните работи [36, 37, 41]. Разрушена е горната част на дясната опора и впоследствие е изградена нова дясна опора, монтирани са двете мостови опори, изградена е преходна зона зад дясната опора, монтирани са бордюри, хидроизолация на мостовата настилка. направена е плоча и пътно платно, демонтирани са корнизните греди заедно с парапета, изработени са и са монтирани нови корнизни греди и парапети, направена е бетонова плоча под тротоарите, при което възстановяване на ПКО на главните греди на мостовата настилка, на стойките и арките бе вече започнато.
На 20 септември 2016 г. затварянето приключи и мостът беше пуснат в ограничено двупосочно движение(фиг. 23). През 2017 г. продължи работата под мостовата настилка и по долната конструкция, докато мостът беше в експлоатация.Изпълнен е ПКО на сводовете, укрепването на сводовете и стълбовете, рехабилитирани са бетонните повърхности на пилоните, основите на стълбовете, подпорите и долната страна на мостовата настилка, изолацията на проходите на тротоарите и направени улуци, направено е ново отводняване на моста и парапетите на ПКО, които са повредени при работа по моста.Реконструкцията е завършена на 30 септември 2017 г.

8.1. Съдържание на реконструкцията

Основната цел на реконструкцията беше подмяна на мостови затвори, възстановяване на водоустойчива изолация на пътната плоча, подмяна на корнизни греди и парапети, възстановяване на стоманената конструкция РКО и рехабилитация на бетонни части на моста [31, 32, 33, 34, 35, 42].

Затворите на моста бяха в окаяно състояние дълго време, и ограничаваха трафика по моста [22]. По време на създаването на моста Žďákov (1967 г.) не е имало налични капачки за мост, които да са водонепропускливи при необходимото деформиране при разширението. Оригиналното затваряне на моста беше с нетипичен дизайн. Това беше стоманено цилиндрично поддържано мостово затваряне, което позволява общо изместване при разширение от приблизително 300 mm(Фиг.24). Движението на крайния напречен елемент в надлъжна посока от затоплянето или охлаждането на основната носеща конструкция се предава на цилиндър, който се движи върху стоманената плоча на основния блок. Подвижният цилиндър поддържа въртящия се елемент на затвора, чийто горен фланец има формата на цилиндрична повърхност. Затворната пластина се движи по цилиндричната повърхност, която е неподвижно поддържана в другия край на мястото на крайната напречна част. Тази конструкция осигурява плавно нивелиране на пътеката за движение. Затварянето на моста не беше водоустойчиво и това беше причината за пълното му разрушаване след приблизително 45 години експлоатация(Фиг.25) Единственото решение на това условие е да се заменят съществуващите затварящи устройства на моста с нови водоустойчиви затварящи устройства с нисък шум. Това също беше свързано със сравнително значителния ремонт на повредените части на двете опори (фиг. 26).







Фиг. 24. Оригинален цилиндричен мостови затвор **Фиг. 25.** Премахнат мостови затвор **Фиг.26.** Изглед на опората

Водоустойчивата изолация върху пътната настилка показа дефекти, изразяващи се в течове, видими при гледане на плоча отдолу(Фиг. 27). На някои места по бетонната плоча се забелязват инфилтрати и локална корозия на стоманени елементи. Носещата бетонна армировка не е засегната от корозия. При ремонта е направена нова пръскана изолация.



Фиг.27. Изглед на пътната плоча отдолу



Фиг.28. Състоянието на пилона преди реконструкцията

Оригиналните корнизи и парапети образуваха една структурна единица.Поради корозия на някои елементи от корнизната греда и парапетите, беше решено да се извърши цялостна подмяна на корнизните греди и парапетите.Съществуващият РКО на стоманените части на моста (дъги, включително укрепване на свода, напречни греди и главни греди на мостовата конструкция) вече беше деградирала до такава степен, че вече не гарантираше дългосрочна антикорозионна защита.По време на реконструкцията е извършена цялостна реставрация на ПКО, с изключение на вътрешните пространства на арките на камерите и напречните усилватели на камерите в ставите на свода. Бетонните пилони, останалите части от подпорите и основите под колоните от двете страни показват значителни дефекти по повърхността си, които не застрашават статическата функция на моста, но е необходимо да се извърши цялостна рехабилитация на повърхностите им [38]. Малкото покритие на армираните сглобяеми пилони е причина за тяхната корозия и откъсване на покриващия слой(фиг. 28).

8.2. Общо състояние на моста преди реконструкция

Състоянието на всички носещи стоманени и бетонни части на моста - Žďákov по отношение на тяхната носимоспособност беше много добро, с изключение на опорите на моста.Това означава, че товароносимостта и експлоатационният живот на моста-Žďákov (с изключение на преходните фуги на моста) след почти 50 години експлоатация са били същите като през 1967 г., когато мостът е пуснат в експлоатация.*Това е почти уникален факт и е резултат от висококачествено проектиране, отговорно производство и изграждане на моста, както и адекватна дългогодишна поддръжка.*Текущото строително състояние на целия мост се отрази неблагоприятно само на строителното състояние на опорите на моста. По време на реконструкцията бяха извършени дейностите, описани накратко в параграф 8.1, които преустановиха започналите деградационни процеси върху стоманобетонните елементи на моста и възстановиха първоначалната им функция. От гледна точка на терминологията би могло да се говори по-скоро за ремонт, отколкото за реконструкция на моста.

8.3. Подробно описание на реконструкцията

Реконструкцията започна със затваряне на движението по моста за всички автомобили и пешеходци.Движението на автомобили се насочваше по обходни маршрути от около 20 км?.Преди затварянето на моста бяха обсъдени и изпълнени мерки за инженеринг на трафика (DIO).

8.3.1. Мост и пътна настилка

В обхвата на целия разпределителен план първо бяха разрушени пътната настилка, бордюрите, пътната настилка на тротоарите и изолацията на плочата на мостовото платно(Фиг. 29). Асфалтовата настилка на пътя с дебелина около 80 mm беше фрезована. Поради променливата дебелина на пътя, той беше непрекъснато проверяван, за да се избегне фрезоване на плочата на мостовата настилка. Последните около 10 до 20 mm бяха отстранени доста внимателно. Оригиналната водоустойчива изолация, която беше направена от разтопени асфалтови ленти, беше внимателно отстранена със строителна машина с нож. Премахването на асфалтовите ленти беше доста трудно. Остатъците от изолацията, които не са отстранени машинно, са почистени на ръка.

Усилието беше оправдано с цел да не се повреди повърхността на бетонната плоча от мостовата конструкция. По този начин бетонната плоча бе открита по цялата ширина между основните греди на мостовата конструкция. Оригиналните каменни бордюри бяха складирани за повторна употреба на реконструирания мост. В двата края, където бетонната плоча на мостовата конструкция граничи със стеблото на главната греда на моста, бетонната плоча на моста бе ограничена от горния фланш на ГВП U 180, заварен към стеблото на главната греда на моста(Фиг. 30 и Фиг. 31).

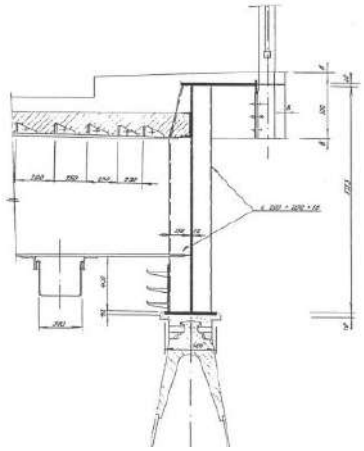


Фиг.29. Бетонна плоча вурху на мостовата конструкциа след отстраняване на пътната настилка изолация



Фиг.30. Завършване на бетонната плоча до главната греда до главната греда на моста

По време на реконструкцията бетонът под тротоара на това място е разбит ръчно с леки къртачни чукове, след което повърхността е почистена с водоструйка. Над колоните под споменатия U напречен профил, от вътрешната страна на стената на основната греда, има вертикално ребро от ъглов профил ʟ 200×16(Фиг. 31). При преустройството се установява, че U напречното сечение не е заварено към стената с непрекъсната заварка, поради което на места от под нарушената хидроизолация под настилката прониква влага във вътрешното пространство на триъгълното ребро(Фиг.32). Това бе причината за възникване на сериозни дефекти през 1998 г. По време на реконструкцията на моста – Žďákov, бе обърнато необходимото внимание на този детайл. Песъкоструените стоманени части (фланец с U-профил, стеблото на основната греда на моста под горния фланец) бяха обработени с инхибитор за корозията. Съединението между U-профила и стената на главната греда беше внимателно запечатано по цялата дължина на моста с епоксиден уплътнител Sikagard 186 със сгъстител Stellmittel T 5% от теглото. Запечатването се извършва с помощта на шпатула.





Фиг.31. Напречно сечение на главната греда **Фиг.32**. Детайл на бетонната плоча от пътната конструкция над пилона при главната греда

В последствие под настилката се оформя контрафалц под наклон 4% от еднокомпонентната смес REDPATCH 1 с дебелина от 5 до 50 mm(Фиг.33), върху която е нанесена пръскана изолация. Изолацията е извършена по протежение на стеблото до горния фланш на главната греда на моста.Тази *мярка предотвратява проникването на влага във вътрешното пространство на триъгълното ребро,* заварено към вътрешното стебло на основната греда на мостовата конструкция, *дори и при бъдещо повреда на правопроходната водоустойчива изолация на настилката.* На ситуацията от 1998 г. не биваше да и се даде възможност да се повтори в бъдеще.





Фиг.33. Контрафалц под тротуара **Фиг.34.** Разрушена пътна плоча

В ДЗС се приема, че плочата над пътната конструкция може да бъде отстранена на ивици с ширина около 2,0 m от главната греда на моста от двете страни по цялата дължина на моста. Причината за това разрушаване и подмяна с нова плоча са резултатите от допълнително диагностично изследване, виж гл. 7. Когато се гледа плочата на моста отдолу, се вижда локалната корозия на кръстовището на бетонната плоча със стоманените фланци на основната греда на моста и напречните елементи(Фиг. 27). Тези факти предизвикаха тревоги за състоянието на бетонната плоча и стоманените фланци на напречните греди. За да се определи действителното състояние, плочата над мостовата конструкция бе локално разрушена в горната главна греда на мостовата конструкция близо до т.н.опора „Милевска“ с площ от приблизително 2,0 m^2, (Фиг. 34), което не можа да бъде извършено по време на диагностичното изследване. Установено е, че бетонът е с много добро качество,

армировката не е корозирала по никакъв начин, горният фланш на напречната греда също не показват корозия. Качеството на бетона е определено чрез сондажите (Фиг. 35). Освен това е извършено повърхностно сканиране на пет избрани зони, за да се определи армировката в бетонната плоча(Фиг. 36).

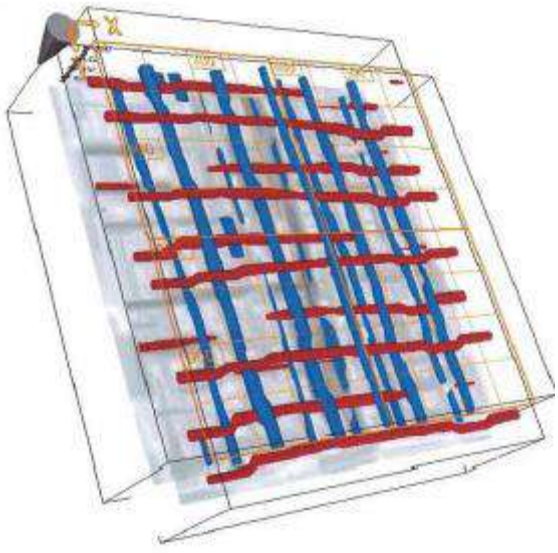


Фиг. 35. Отвор в ст. бет. плоча



Фиг. 36. Търсене на армировка с радар

В полето между напречните греди, 8 pcs/m надлъжна армировка са монтирани в долния слой на бетона; над напречните греди същият брой надлъжна армировка са в горния слой(Фиг. 37). Напречната армировка на срязване се състои от 4 pcs/m [38].



Фиг.37. Пример за компютърна обработка на радарното измерване



Фиг.38. Почистване с водоструйка

Затова беше решено, че мостовата настилка е в добро състояние, че не е атакувана от сол и че няма да се отстранява. Това беше значителна промяна по време на реконструкцията, но напълно оправдана и със сигурност в полза на цялата конструкция на моста - Žďákov. Освен това беше извършено цялостно почистване на горната повърхност на бетонната плоча на мостовата настилка с водна струя при налягане от 2100 бара(Фиг. 38), което беше определено на базата на тест за бластиране. Премахнати са стърчащите стоманени елементи от бетона. Това са напречни неносещи стоманени пръти близо до горната повърхност на бетонната плоча, които очевидно са били

използвани за издърпване на бетона по време на полагането му. Останалите стоманени пръти са песъкоструени до метален блясък и снабдени с пасивиращо покритие SIKAGARD 186. Зоните, изпъкнали над околната повърхност на плочата, са обработени чрез шлайфане до максимална дебелина от 15 mm, като се вземе предвид покритието на армировката. Неравностите и вдлъбнатините под нивото на околната повърхност с дебелина над 5 mm бяха изравнени до нивото на повърхността на плочата с помощта на епоксидна смес SIKAGARD 186, напълнена със силициев пясък. Ремонтите извършени с полимербетон се намазват със SIKAGARD 186 и се посипват със силикатен пясък фракция 0,0/0,8 mm. В оста на водостоците настилката има надлъжно корито, което е пресечната точка на напречния наклон на пътното платно и контрафалца под тротуара. Оста на коритото е разположена на разстояние 1,0 m от стената на главната греда на мостовата настилка(фиг. 39). Между първоначалните дренажи в бетонната плоча на мостовата настилка са пробити отвори за по-късен монтаж на дренажни тръби на изолацията DN 50 mm(Фиг. 40). Осовото разстояние между дренажните тръби на изолацията произтича от надлъжния профил на коритото. Надлъжният наклон на коритото е минимален в горната част на дъгата, тъй като цялата настилка на моста е в дъга с голям радиус, а в горната част допирателната към дъгата е хоризонтална. Следователно аксиалното разстояние на дренажните тръби близо до горната част на арката е най-малко. След завършване на защитата на изолацията на пътното платно, жлебът с ширина около 200 mm и дебелина около 35 mm е запълнен с дренажен пластичен бетон(фиг.41). Така създадената обработка може да осигури оттичането на водата от повърхността на изолацията в надлъжна посока към дренажи или дренажни тръби от некорозионна стомана А4.



Фиг.39. Жлебове по оста на моста



Фиг.40. Дренажна тръба в жлеба



Фиг.41. Жлебове, пълни с с дренажен пластбетон

В средата на участъка на моста има входове към ревизионния мост на арките от двете страни на пътното платно.Входовете са покрити със стоманен капак(Фиг. 42 и Фиг. 43).



Фиг.42. Вход към ревизионната пасарелка на дъгата



Фиг.43. Стоманен люк в пътното платно

Върху главните греди на моста са поставени две ставни устройства, които са разположени над първата вертикална линия зад пилона на „Příbram“ и над последната вертикална линия пред пилона „Milevský“. Съединенията намаляват напрежението върху главните греди от вертикален огъващ момент. В бетонната плоча на моста преди реконструкцията във фугите се появяваха малки и тънки като косми пукнатини.По време на реконструкцията е отстранена пътна плоча с дължина 1100 mm и ширина около 200 mm. Полученото пространство е покрито с нова стоманена конструкция, която е заварена към горния фланш на главния носач на мостовата настилка(Фиг. 44). Това решение предотвратява появата на опънни напрежения в настилката на плочата в непосредствена близост до ставните усройства в главната греда на мостовата конструкция.



Фиг.44. Детайл в плочата при ставата на главната мостова греда

Върху бетонната плоча на мостовата настилка след водоструенето, изравняване на повърхността, изграждане на контрафалц и монтаж на дренажни тръби е направена пръскана изолация(фиг. 45), която е изтеглена по стената по двата края до горния фланш на стеблото на главната греда на моста(фиг. 46). На мястото на отводняването, оригиналните дренажи в долната част са запазени и върху тях беше изтеглена пръсканата изолация. Изолацията бе изтеглена и върху маншета на дренажните тръби. Използвана е изолационната система Sikalastic с двойно анкериращ лепилен слой и хидроизолационна полиуретанова мембрана. Адхезивният слой се състои от два слоя епоксидна смола с нисък вискозитет Sikagard 186, поръсена с изсушен силициев пясък с фракция 0,3-0,8 mm. Дебелината на първия слой зависи от абсорбцията на основата, а дебелината на втория слой зависи от грануларността на пясъчната посипка. Действителният изолационен слой е полиуретан, направен от материала Sikalastic 821/821LV с номинална дебелина 2,1 mm, но поне 2,0 mm. Лепилният мост Sikalastic 823 е нанесен върху изолационния слой с разход от около 0,1 kg/m^2, чиято дебелина не е измерена. По време на изпълнението на изолацията е организиран засилен постоянен надзор на изпълнителя и инвеститора. Спазваха се последователно технологичните и изпълнителни предписания за хидроизолация на бетонната плоча на мостовата настилка с пръскана система на полиуретанова основа.



Фиг.45 Полагане на изолация



Фиг.46. Завършване на пръскана изолация до стеблото на главния трегер на моста

Първоначално е изпълнена изолационна защита под настилката на ширина 1,0 m от стеблото на основната греда на мостовата конструкция, т.е. до оста на коритото, като се използват асфалтови ивици. Причината за тази защита беше основно да се осигури достъп на работници, така че движението на хора върху пръсканата изолация на палубната плоча да бъде напълно изключено. Впоследствие е изпълнена защитата на изолацията под пътя с модифициран лят асфалт МА 11 IV PmB 10/40-65 с проектна дебелина 35 mm, минимум 25 mm, максимум 73 mm. Средната дебелина е 42 mm(фиг. 47). Извършено е и изравняване на повърхността на мостовата настилка в слоя лят асфалт. Полагането е извършено ръчно на секции по цялата ширина от около 10 m между ръбовете на бъдещото корито. Монтирани са дървени рамки на местата на водостоците и дренажните тръби. Разписана е последователмостта за движение на превозни средства, транспортиращи асфалтовата смес и за движение на работници по изолацията, което предотвратява разрушаването на изолацията. Тази процедура беше под постоянното наблюдение на техническия надзор на инвеститора.

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МАЛКО ЗА ГОЛЕМИЯ ИНЖ. ЛАЛО РАЙКОВ (1923 -1997)

Васил Пенчев[1]**,**

ВИПроект

Абстракт*: Авторът на доклада ни връща в историята на периода, през който в нашата държава се изграждаха значими и значителни по размери мостове от обикновен и предварително напрегнат стомнобетон, на голяма част от които проектант беше големият български мостови инженер а именно: Лало Райков. Неговата съдба като гражданин на България и голям специалист в областта на стоманобетонните мостове, е толкова необикновена, че си заслужва да бъде преразказана накратко с акцент върху няколко житейски случки от неговия живот. Целта на доклада е запознаването на младото поколение мостоваци с един наистина величав образ на човек, кавалер и джентълмен от недалечната българска проектантска мостовашка школа.*

Ключови думи*: стоманобетон, значими по размери мостове в София*

1. Увод

На пръв поглед тези истории може да не впечатлят някои мои читатели, но искам веднага да ги уверя, че този човек е оставил в мен дълбоки следи в начина ми на мислене (Фиг.1)



Фиг. 1. Лало Райков

1 Випроект

Познанството ми с него беше като втори университет. Той е един от първите завършили новосъздадения още в царско време ИСИ. След 1944 година, инженерно-строителният институт бил прекръстенна ИСИ „Йосиф Сталин“. Един наш много добър инженер, след Девети (1944)избягал в САЩи като му видяли дипломата, помислили, че е завършил с отличие Идеологически Сталински Институт! Милите те!.Лало е учил заедно с този (избягалия), но и през ума му не е минавала идеята да бяга някъде.Беше много известен инженер, но главно бе специалист по мостовете. Неговият патент „Камертон“ бе приложен за първи път на три моста в участъка Пирдоп-Розино (на подбалканския път за Бургас) и икономическият ефектот тези камертони бе осезаем.Но нямаше пари да го патентова в целия свят , а само в соцлагера. Това решение на изключителния българск инженер-мостовак Лало Райков, което е патентвано само в бившия социалистически лагер се нарича „Камертон”. Стълбовете са разрязани в най-горната си част, както е показано на на схемата . Големите хоризонтални сили, които се предават в горната част на високите стълбове, предизвикват огъващи моменти при основите. Каква е разликата между класическото решение и при камертона е видно от следващата фигура(Фиг.2а) и (Фиг.2б).

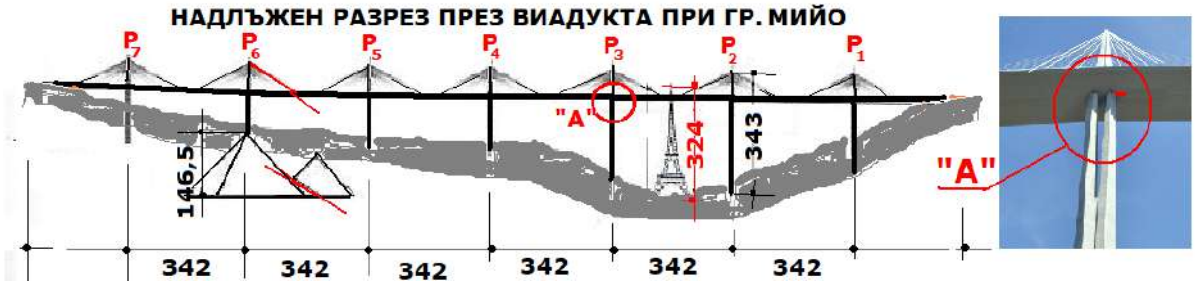


Фиг.2а. Сила Р във върха на колоните



Фиг.2б. „Камертон” колоните

Световно известният арх.Норман Фостър приложи принципа на „камертона“ на виадукта при река Тарн, до градчето Мийо в Южна Франция. Този мост е едно архитектурно - конструктивно бижу, с отвори не по 36 m, а по 342 метра. Най-високият му стълб не е 72 m., а 343 м. и е по-висок от Айфеловата кула.За нея знаем, че през лятото, височината й нараства със 17 см. Конструкторът е инж. Мишел Вирложьо-от строителната фирма „Айфел“, която е изпълнила виадукта (Фиг.3).



Фиг.3. Надлъжен разрез през виадукта при град Мийо

При едно пътуване на Лало с кола до ГДР, сърбите научили, че ще пренощува в Белград и го потърсили в хотела. Лало, неблагонадежден в онова време като фабрикантски син, с перфектен немски, френски и английски език, дълго не го пускаха навън, но сега, както и при други подобни случаи, с него пътуваше Н. Т. - много благонадежден, генерален директор на някакъв „Т…строй“, малко простоват, ама по природа практичен и честен. Много ценеше Лало като инженер и си го беше назначил при себе си. Та сърбите (говоря за изтъкнати сръбски инженери- мостоваци) помолили Лало да го заведат до Баня Лука . Там имало един мост, за който се безпокоели много и предполагали, че проблемът е в лагерите при опорите- не помня подробности. Лало въобще и не помислил, че това е възможно, ама Н.Т. категорично настоял да отидат до там. Какво по- голямо уважение към този известенбългарски- специалист. Между другото, едно немско списание по това време, беше класирало тримата най- известни българи за момента: Борис Христов(всички знаеха тогава Николай Гяуров и повечето не бяха чували за гениалния бас), Дончо Папазов (с пътешествията си и планктона) и Лало Райков (с „камертона“ си)!..

Отиват на моста, Лало прави оглед, прави необходимите изчисления и гарантира с подписа си, че мостът няма проблеми. И разбира се, сърбите искат да платят хонорар за становището. Лало категорично отказва. Н.Т. му споменава, че това е нормална практика и е съгласувал това посещение в Баня Лука с посолството ни в Белград. Но Лало не иска! Тогава Н.Т. взема парите и случаят приключва с големи благодарности. В хотела Н.Т. дава парите на Лало и го успокоява, че това е нормално. Сумата е голяма- пет хиляди долара.

Но Лало си е…Лало! Вечерта Лало поканва сръбските си колеги в бара на хотела. Скъпо уиски, черен хайвер, ядки, скъпо шампанско и някъде след полунощ, нашият плаща сметката на гостите си, ама и на всички посетители на бара. Поръчва още така, че да останат 200—300 долара бакшиш и …толкова. Сръбските му колеги знаели историята на Лало с ж.п. моста при гара Лакатник, на който той е бил проектант, когато е на христовата възраст (1956 г). Мостът паднал при бетониране на сводовете и убил трима работника. Доказало се категорично, че Лало няма вина. Проектът му е бил проверяван и от немски експерти- никаква грешка! Вината е била на един негов възрастен колега (тогава на 55 години), който бил проектант на дървеното скеле на моста. Лало е оправдан от съда, но той отказва свободата, ей така, защото е джентълмен и защото е голям човек! Георги Данаилов увековечил това в пиесата си „Краят остава за вас“.Това малко хора го знаят.

Една вечер, с Лало бяхме сами в ателието му на ул. „Добър юнак“. Аз бях чертожник на неговия „камертон“ тогава.Той ми разказа, как не искал да засегне старият си колега от „Транспроект“ и отивал на работа сутрин рано, към 6 часа, отварял изчисленията му на кофража за моста, проверявал,проверявал така месец, няма и няма грешки. Писнало му и не проверил последните няколко страници, където бил “ключът“ на свода на моста. А там

колегата му сбъркал при сметките и допуснал напрежения в дървената конструкция от 1000 кг/кв.см. при допустими 100! Една фатална нула. Лало ми сподели тази вечер това:
Звънят му по телефона. Казват му от обекта, че при бетонирането, скелето скърца. Лало им рекъл, че щом неговият подпис стои на чертежите, значи всичко е наред. Ама всичко е наред, но дървеното скеле е проектирано от друг и Лало го е подписал като проектант на стоманобетонната част, т.е.- съгласувал формално. Но, когато става белята, през нощта го вземат с кола и го закарват (не е важно къде), настаняват го на една маса, а срещу него прожектори и той не вижда хората отсреща. Започва професионален разпит. Колеги, професори и доценти, задават писмено въпроси, за да не ги разпознае по гласа, следователят ги чете, Лало отговаря. Накрая му заявяват, че няма пропуски в знанията. В съда го оправдават напълно, но той не приема. Чувства вина, че като са му съобщили, че скелето скърца и ако е бил отишъл на място, не е можел да не забележи, че сеченията на гредите и дъските на кофража при „ключа“ на свода са много тънки и опасни. А проектантът на кофража е гледал, ама не видял… Съдба!
То и тъстът на Лало бил с подобна съдба- влиза в затвора заради едни стоманени хангари в Казанлък, където имало една фатална проектантска грешка и единият хангар паднал. Става въпрос за доц. инж. Г.Г., когото помня добре от университета. Лало беше женен за дъщеря му Д. Г.- също превъзходен конструктор - професор).
Когато Лало излиза от затвора предсрочно, единствен, който му подава ръка и го взема на работа, е инж. Георги Атанасов, директор на ЗСК –Кремиковци.Лало и не е помислял да се върне в „Транспроект“. Пускат го предсрочно, защото в затвора прави контрапроект на Сухия док във Варна, проектиран от „Транспроект“ и понижава цената му със 70 милиона лева. Тъстът му вкарва проектите на „Транспроект“ в затвора. Лало прави сметки на ръка с малката сметачна линийка и с пълната си глава със знания. Съставя едни уравнения с много неизвестни, които тъстът му успява да ги занесе в Полша, където има голям изчислителен център, много преди у нас. Решенията доказват правотата на решението, което Лало е направил на ръка и представя проекта си. По време на проектирането в затвора, много му помагат две млади негови колежки от „Транстпоект“ – М.Й. и В.Б. Когато Лало започва работа в ЗСК-Кремиковци, те отиват веднага на работа при него.
След като ми свърши разпределението за три години в проектантската организацияв Ловеч, постъпих в ЗСК- Кремиковци с надеждата да получа софийско жителство. Имах една леля (учителка по химия -Л.К.), която явно много ме е обичала и разправяше, че съм много кадърен и са ме „изтеглили“ в София. Като научих това, идваше ми да потъна от срам – върви разправяй, че не съм лъгъл и не съм се хвалил.
Тогава срещнах този невероятен човек. Инж.Лало Райков беше главен инженер на проектантското бюро към завода. Бързо научих много неща за него и го гледах със страхопочитение. Той ме прие със симпатия и веднага ми даде да проектирам една промишлена сграда. Каза ми: *Правиш архитектурата, конструкцията, ВК-то, вертикалната планировка и само ЕЛ-а и ОВ-то ще ги правят други*. Да, ама той говореше толкова бързо и неразбрано, че помолих груповия ми ръководител П.К, (той беше с мен при поставянето на задачата), за да ми е нещо като преводач. Е, за два месеца свикнах и научих лаловия език. Сигурно съм се справил добре, защото Лало, преди да напусне ЗСК-Кремиковци и да отиде при Н.Т., казал на Георги Атанасов нещо много добро за мен.
Лало ме е хвалил някъде и преди отиването ми в „Главпроект“ (след като той ми беше възложил да направим със спироли подземните паркинги на НДК). И идва той един ден с гръм и трясък при мен в „Главпроект“, за да ме поздрави на новата ми работа и да ми пожелае успех, ама води със себе си и приятеля си арх.Асен Горанов (вече се познавах с него, тъй като той беше известен главпроектец!) Между мен и главния директор има една секретарка - красива и много интелигентна млада жена (М. Р.) и вратите ни май бяха все отворени. Лало говори неразбираемо, но за сметка на това- гръмогласно. Този човек, както и Георги Атанасов, при подобни срещи, пиеха коняк „Плиска“ с ядки и кафе. Това става

по Андроповото време – правеха се проверки по кафенетата и ресторантите, ловяха хора, които са излезли в работно време и ги наказваха, а да се пие на работното място, беше недопустимо. Бях щастлив, че Лало ме е уважил и си казах майната му на правилата. Веднага М.Р., с която много се уважавахме, разбра ме с една дума и мигновено купи и донесе коняк и ядки, а кафето направи самата тя. Главният ми директор, арх. Д.З., млад, амбициозен, принципен, незнам виждал ли беше алкохол, освен като дете, ако са му мажели раните с ракия. Ха сега де! Отивам аз при Д.З. и му викам да дойде и да го запозная с една голяма личност. Той се зарадва,и тръгва с мен. Лало го тупа свойски, казва му да седне при нас и му каза, че голям късмет е извадил с моето назначение. Аз се притеснявам винаги от такива неща, притеснен съм и от коняка на бюрото и. Трябва да призная, че тогава усетих, че този млад ръководител не е тясно скроен. А милата секретарка заключила вратата и достъп до нас нямаше. Успях набързо да обясня на Д.З. кой е Лало Райков. Конякът си вършеше работата и по едно време, Лало каза, че ще ни разкаже една история: Н.Т.,моят шеф, идва при мен преди две седмици и казва, че отиваме до Лондон и ще участваме в един конгрес. За такова пътуване до Лондон съм мечтал от дете. Споделих с него и му казах, че ще запазя две стаи в един стар и изискан хотел до Биг Бенда, за който ми е говорил брат ми, който живее отдавна във Виена. Той се съгласи. Запазих двете стаи. На летището гледам Н.Т., че е много блед, изглежда и притеснен. Питам го какво му е, а той- няма нищо. Пък не е от самолета, летял е доста. В самолета ни хапна, ни пийна. Успокоявам го, а той пак каза, че му няма нищо. Следобед кацнахме и наех едно черно такси, спираме пред хотела, влизаме във огромно фоайе, насреща рецепцията, вляво- насядали спокойни хора, пият чай (не уиски), вдясно - над 20 телефонни кабини. На всяка кабина отгоре пише държавата: САЩ, Канада, Австралия, Нова Зеландия,… България я няма. За един миг спираме на огромния килим, 24 на 24 м., оглеждам със затаен дъх и си казвам дано не е сън. И в този миг, Н.Т. като стрела, пресече килима по диагонал и влезе в кабинката с надпис Канада. Естествено, че се озадачих, не ми каза нищо, не знае дума английски. По едно време един изискан господин от рецепцията идва при мен, набързо се разбрахме, че съм резервирал две стаи и чакам колегата си. Господинът ми каза, че колегата ми явно има проблем. Няколко минути стои без да прави нищо, защото трябва да вдигне слушалката. тогава кабината светва и той може и да ползва телефонните указатели, ако е забравил номера. Видях, че кабинките имат по едно малко удължено прозорче и при Н.Т. е тъмно. Казах, че ще видя, да не е изгоряла крушката. Господинът ми каза, че на рецепцията има контролно табло и няма проблем с крушката. Мамка му, си рекох и в този миг Н.Т. открехна леко вратата на кабинката и ми прави с пръст знак да отида при него. Отидох с бързи стъпки. „Дай ми куфара! Насрах се!“-промълви той, вече не толкова блед. Изтръпнах! Но му казах: „Добре, че стана тука, а не в София, преди да се качим на самолета. Все пак пристигнахме в Лондон!“На това място всички се заливахме от смях, та се наложи Лало няколко пъти да повтаря какво му е казал на Н.Т.

Та този голям човек ми сподели, че на делото за моста при Лакатник, преди още експертите да докажат невинността му, административното, партийното и профсъюзното ръководство на „Транспроект“ не са го защитили. Фабрикантски син, самонадеян, незнам какво си. Ама пропуснали да кажат, че когато се видяло, че ж.п. мостът при Лакатник трябва да е в крива в план(гледано отгоре) и с три свода, питали кой ще се наеме да го проектира, всички си затраяли и само Лало казал, че може. Нали си представяте завистниците как са потривали ръцете след аварията. И Лало се зарекъл, че като излезе от затвора, ще покаже на Транспроект“, че не са били справедливи към него.

Мостовете при Пирдоп-Розино, по проект на „Транспроект“, струваха по 12 милиона. Лало, с гениалния си „камертон“ направи мостовете по 4 милиона! Най- високият стълб беше 72 м. сечението му в план беше 5,0 на 5,0 м. и стените бяха с дебелина 25 см. Най-

горе стълбовете бяха разрязани по средата с фуга от 12 см., отиваща надолу 12,0 м. Истински камертон. 36- метровите предварително напрегнати стоманобетонни греди стъпваха от двете страни на камертона и нямаха връзка помежду си. Така хоризонталните сили на пътното платно караха камертона да трепти и това намаляваше рязко огъващите моменти в основите на опорите. Именно това водеше до голяма икономия на материали и средства. Проектант беше Лало Райков, а му помагаха много известните му колеги Коста Цеков и Жоро Ганчев. Контролите бяха проф. инж. Христо Попов, проф. Христо П. Върбанов и доц. инж. Марчо Минев (по-късно и той стана професор). Чертожниците бяхме аз и двете любими колежки на Лало, за които ви споменах по- горе Работихме всеки ден от 18 до 24 часа, след работа. След два-три месеца. Жоро Ганчев докара от Израел качествен паус. Проектът беше като с ръка непипан!

Аз си купих кола, другите също бяха възнаградени богато за тежкия и отговорен труд, а на Лало му останаха пари: за шлифер!

Мостът на „Цариградско шосе“, на т.н. „Четвърти километър“, проектиран от „Транспроект“, беше от монолитен стоманобетон. Лало обявява на експертен съвет, че ще го направи много по- евтин. Възлагат му го. Прави го също от стоманобетон, но напрегнат на място, след като стоманобетонът добива проектната си якост. И остават пари за още един по- малък мост.

Държавната проектантска организация „Транспроект“ получи няколко удара от бивш свой колега, към който се отнесе несправедливо при описания неприятен инцидент. Един човек, със своите знания и опит, може да е много полезен на обществото и би трябвало да получи някакво признание. Не всичко трябва да оправдаваме с „такова беше времето“. Лошото е, че способните хора у нас много не ги долюбват – посредственитеса много, за съжаление.

WROUGHT IRON CUT NAILS WITH ROSE HEADS ARE STILL IN SERVICE

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Abstract: *The iron cut nails were the most important structural pieces to compose a wooden frame in past. Still there are sacred buildings or residential buildings around, standing more than 200 years, are having wrought ıron cut nails with rose heads for their wooden frame stabilizing masonry system with stone walls.*
At a distance very close to Priene Archeological Ruin Area, there is a church building, written that it was constructed in 1821. Its masonry structural system with stone, has a very well constructed wooden frame on and in the stone wall at both direction, connected the wooden pieces with rose head nails.
It could be easily seen through the damaged parts of wooden linten beams to support the stone walls and also the stone columns in rectangular.
For this church is very close to Priene, it could be thought that this kind of building system could become from Priene construction culture.
In the paper, the visible wrought iron cut nails with rose heads will be presented and will be explained how they could manage to hold the whole structural system in standing up.

Key words*: The wrought iron cut nails with rose heads, wooden linten beams, Priene Archeological Ruin Area, church building, masonry system*

1. Introduction

As it is well-known that whole Türkiye is full with archeological ruins and still the excavations are going on. During the several visits to this sacred building to understand how it is still standing, lots of photos are taken, lolts of measurements are done for building survey, and lots of minutes to watch in silence to listen the true words coming from the whole structure, at its place.

Suddenly, these wrought iron cut nails with rose heads are become more visible over the wooden frame, surrounding the perimeter walls in nearly 150cm in distance till to two arches. And also on the rectangular columns, there are wooden structure as a frame, in 70-80cm in distance till to the top of the frame together with stone.

What is interesting, these nails couldn't be easily taken off from their places even though half of the nails are at the outside. And these nails are a bit long nails, having less section which goes to the end of the nail, straight from the rose head of the nail.

In the paper, some important details will be examined to understand how they support the total structural system of the building, starting from the first meeting time with the details (Fig.1.) [1]. And the shape of a wrought iron cut nail with rose head is very important (Fig.2.).

And also, sometimes the wooden system could be invisible under the plaster, and only the wrought iron nail could be seen to be connected to another iron structural element, to span between the columns, at the end of the arches (Fig.3.) [2].



Fig.1. The connection point of two wooden structural elements at one point where both of them are in 90 degrees position, connected with wrought iron cut nail with rose head (photo taken by Y.K. Aktuglu, on 31st of July, 2022, at first meeting with the building)

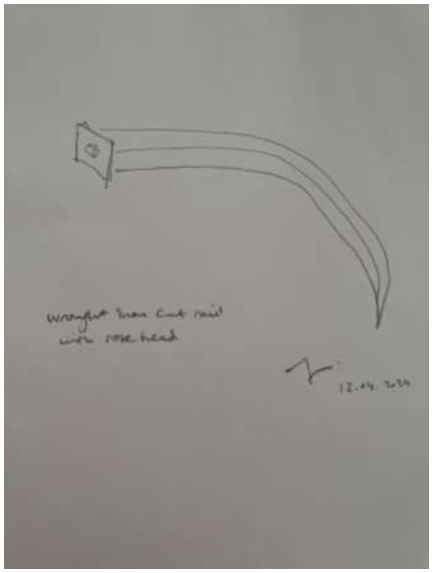


Fig.2. The drawing of Wrought Iron Cut Nail with Rose Head (drawn by Y.K. Aktuglu, on 12.04.2024)



Fig.3. Wrought Cut Nail with Rose Head over the Column (taken by Y.K. Aktuglu, on 25.01.2023)

2. Wrought Iron Cut Nails with Rose Heads, together with Wooden Structure

The structural system of the church building is a masonary one with stone for the perimeter walls and columns inside. But the vital part of this system is the wooden stabilizing structural elements at both directions to reinforce the system with wooden sections, around 6*12cm or more. These wooden frame is inside the wall at both sides of the wall, outside and inside at the same level, and inside of a concrete wells or tubes. Or we could easily say that the wooden structural elements are over concrete layers and and also there is a layer in vertical, where the opposite one is plaster layer, and also top of the wooden structural element, there is another concrete layer. Sometimes, this place is made ready with stones, which are good to shape. These frame goes through the whole building to stabilize the perimeter walls and also the perimeter of the rectangular or square columns. Here are some details(Fig.4.), (Fig.5.),[3].



Fig.4. A detail on the wall (photo taken by Y.K. Aktuglu, on 06.06.2023)



Fig.5. A Iron cut nail on the wooden lintel (photo taken by Y.K. Aktuglu, on 06.06.2023)

At the corners, two pieces of wooden elements are connected with wrought iron cut nails with rose heads. Sometimes they could be easily seen from the surface, sometimes for they are burden inside the connection points, it should be carefully looked to see the iron inside the wood.For the end of these specially designed and forced iron cut nails, the shape of the iron bar in square section first, immediately at the starting point, just under the rose heads, turns into just a point at the end of the bar, and a bit curved to hold the system perfectly (Fig.6.) [3].



Fig.6. Wrought iron cut nail on the corner of the rectangular column (photo taken by Y.K. Aktuglu, on 06.06.2023)

After examining these nails on different parts of the wall and columns, it could be easily said that the real reason why this old church of St.Nicholas, from 1821, near Priene, is still standing is the strength of these nails, coming from their producing procedures and surely also from their shapes to be knocked directly first with wooden lintels, and then with the whole masonary walls and columns with stone[4].

The rose head is rarely on vision(Fig.7.),(Fig.8.), [5]. Mostly rose head is inside the construction as a burden piece.

There are more secrets about how to construct with wrought iron cut nails with rose heads in this old church building, which will be there to be understood whenever it will be ready to know the details, to make manifests build better buildings in future.



Fig.7. The rose head of a wrought cut iron is in view (photo taken by Y.K. Aktuglu, on 19.07.2023)



Fig.8. The rose head of a wrought iron, which is at the real corner of a perimeter wall (photo taken by Y.K. Aktuglu, on 19.07.2023)

3. Conclusion

As a conclusion, WE HAVE TO MAKE MORE AND DEEPER RESEARCHES to understand the situation how the wrought iron cut nails with rose heads support the masonry structure, through wooden frame stabilizing structural elements, over the concrete surface for wooden lintels.

While visiting archeological ruins, we have to look for these kind of nails. More samples, more information will touch directly to our present structural systems for designing future systems for buildings and structures better.

Just after one year later on 31st of July, 2023, when the first time was 31st of July, 2022, that Servet Balcioglu met me meet with this beautiful wealth first time, we checked again all details, more or less everything was same. And my friend got tired a bit(Fig.9.), [6].



Fig.9. At the visit after one year later, everything was same, just my friend got tired a bit (photo taken by Y.K. Aktuglu, on 31.07.2023)

Through these visits, at the same time, another research file has been opened about the St.Nicolas churches around the world, where there is a unique one in Lido, Venice, Italy, where there is another kind of wrought iron cut nails with rose head around St.Nicolas Church in Lido(Fig.10), [7].

It is time to start visiting St.Nicolas again soon, sure first in Türkiye, then in Lido, to continue researches about wrought iron cut nails with rose head and St. Nicholas.



Fig.10. A wrought iron cut nail with rose head, wround St.Nicholas Church, in Lido, Venice, Italy (photo taken by Y.K. Aktuglu, on January.2024))

Acknowledgement

I am very thankful to my classmate from 6.Math.A in BAL, **Servet Balcioglu**, computer engineer, who let me know about this beauty first time in 31.07.2022.
When I met with this church building, I felt that I am inside of a wonderful product, coming from 1812.
Later, I did several visits to this place, to hear the music of the structural elements to show me how they could stay till my time.
Suddenly I took care of the nails, which are the main structural elements to manage to make the church building survive till now.
Sure I have learned a lot.
Thank you very much once more, Servet Balcioglu, for this very wealthy meeting.

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ПРОУЧВАНЕ НА СТРОИТЕЛНАТА ИСТОРИЯ НА ЧИТАЛИЩНАТА СГРАДА С РИТУАЛНА ЗАЛА НА НАРОДНО ЧИТАЛИЩЕ "ОТЕЦ ПАИСИЙ - 1927“ В С. ИЗВОР, ОБЩИНА РАДОМИР

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***Abstract**:*

Chitaliste "Oter Paisiy" in the village of Izvor was founded on January 9, 1927 and played important role in the political history of the district of Radomir during the XXth century. The building of chilaliste "Oter Paisiy", which is located in the center of the village, was opened in 1980. The community cultural center with a ritual hall, which is located in a village, was designed by team of local talented local designers and features the functional program and representative architectural post-war modern characteristics of a significant cultural center in Pernik region.

***Key words:** chitaliste, community cultural center with a ritual hall, post-war modernism, synthesis of arts*

1. Увод

Читалищата възникват в средата на XIX в. като първата българска обществена, просветна и дарителска институция, израз на светските културни и образователни нужди на стремящото се към независимост българското общество, средство за консолидиране на българската национална идентичност.[3] [1,2]

Първите читалища през Възраждането възникват като библиотеки и клубове по тетрална самодейност в частни жилищни сгради.4 След Освобождението започва изграждането на първите читалищни обществени сгради – читалищата във Велико Търново, Свищов, Видин и др, а след началото на XX в. започва масовото изграждане на читалища първо в по-големите градовете, а през 30-те години на XX в. - в по-малките градове и селата. Обогатяването на читалищната мрежа продължава и през втората половина на XX в. като целенасочена политика на социалистическия режим. [4]

Читалищната мрежа в района на Перник стартира с основаването на първите три градски читалища в Брезник (1870), Радомир (1895) и Трън (1895), а през 1898 и 1899 година са основани съответно читалища и в селата Дивля и Сирищник. Читалището в село Извор е основано в периода на масово създаване на читалища

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[3]Читалището в Свищов е учредено в дома на възрожденеца Димитър Начович.

в Радомирско в периода 1926-28 г., когато е приет и закона за читалищата. Така в района на Радомир са основани общо единадесет читалища, или около една десета от общия брой читалища в Пернишка област момента. През първата половина на XX век са основани 63 читалища или повече от половината от съществуващите днес в района на Перник. През последните двадесет години в Пернишка област са основани нови осем читалища, доказателство за жизнеността на читалищната институция като средище за извънучилищно образование, художествена самодейност, съхраняване на автентичния фолклор и др. [1,3]

2. История на читалище "Отец Паисий“ в село Извор.

Читалище "Отец Паисий“ в село Извор е основано официално на 9 януари 1927 г., когато с тайно гласуване е избрано първото читалищно ръководство с председател Методи Георгиев, заместник председател Милан Манов, секретар Манол Михайлов, касиер Димитър Беков, библиотекар Алекси Апостолов, и домакин Александър Николов. През първите петдесет години от своето съществуване читалищната канцелария и библиотека се помещават в различни жилищни сгради в селото. За изнасяне на театралните самодейни представления, основна дейност на читалищния състав, е използван училищният салон. Със средства, събрани от читалището, през 1929 г. училищният салон е оборудван с тетрална завеса, през следващата 1930 г. – със столове за зрителите. През 1938 г. читалището закупува радио-грамофон, а през 1952 г. – кино машина. [5]

Изграждането на читалищната сграда в село Извор е осъществено в контекста на масовото строителство на читалища на територията на община Радомир през периода втората половина на XX в., когато освен нея са построени читалищата в селата Кленовик, Стефаново, Дрен, Копаница, Друган, Долни Раковец, Чуковец, Червена могила, Углярци и Върба - днес квартал на град Радомир. [4]

Построяването на самостоятелна читалищната сграда в с. Извор, е организирано по инициативата и с активната подкрепа на дългогодишния първи секретар на Окръжния комитет на БКП в Перник през периода 1971 – 84 г. - Борис Манов (1922, Извор -2009) - партизанин от Трънския отряд, член на РМС и БКП, генерал-майор от Държавна сигурност, последователно началник на Второ, Трето и Шесто управление и Института за информация и научна организация, депутат в VI, VII и VIII Народно събрание (1971 – 86), награден с ордените „За боева заслуга“, „Народна свобода 1941 – 1944“ II степен, „Червено знаме“ (1957), „9 септември 1944“ (1959).

Биографията на родения в село Извор местен партиен функционер е тясно свързана с политическата история на Перник и Радомир през втората половина на XX в. През 1943 г. е арестуван по политически причини като гимназист и оправдан, през следващата 1944 г. прекъсва следването си в специалност „Агрономство“, за да се включи в партизанското движение. Веднага след политическия преврат започва работа в Държавна сигурност и ръководи последвалите репресивните събития и наказателната акция срещу горяните на местно ниво. През 1946 г. завършва Военно училище „Васил Левски“ в София, през 1956 г. специализиран курс в СССР, а през периода 1962-67 г. като

ръководител на Висшата специална школа „Георги Димитров“следва право в Софийския университет. [6]

През 1966 година в село Извор е издигнат на Паметник на загиналите в Отечествената война 1944-1945 година, който пръв поставя отпечатъка на следвоенния модернизъм в градоустройствената тъкан на селото.

Сградата на читалище "Отец Паисий“, е построена от поделение номер 64920 в Перник на Главно управление „Строителни войски“ с технически ръководител Петър Младенов в периода 1973-1980 година. Открита е официално на 24 май 1980 г. като част от общонационалното честване “13 века България” под патронажа на Людмила Живкова. През 1984 г. към читалището е създаден център "Знаме на мира" за художествено обучение и подготовка на децата от с. Извор и региона за участие в едноименния международен детски форум, също по инициатива на Людмила Живкова. През 1987 г. с помощта на читалищните дейци в сградата на битовия кобинат е открита музейна сбирка, посветена на местното партизанско движение.

На фасадата на сградата е поставена плоча с имената на проектантите и техническия ръководител на обекта.

3. Проектантски колектив.

Проектът за читалище "Отец Паисий“ в село Извор, създаден в периода 1973-1976 година в Проектантска организация – Перник, е поверен на колектив в състав архитект Петър Радев, инженер - конструктор Елисавета Панова, художниците Филип Седефчев и Асен Старейшински.

Архитект Петър Радев (1936 - 2014), който след дипломирането си през 1963 г. в катедра „Жилищни сгради“ при архитект Димитър Цолов в Инженерно-строителния институт София, постъпва на работа в Проектантска организация в град Перник. Автор е на множество градоустройствени разработки и проекти за обществени, жилищни и промишлени сгради и интериори. Сред тях се отличават Профсъюзният дом (1963-64), съвместен проект с архитект Иван Сиврев, отличен с втора премия на наградата „30 години социалистическа революция“ от Общински съвет Перник, самостоятелните реализации на Младежкия дом, дом за весели граждански ритуали и пешеходната зона в Перник, дом-паметник „Радомирска република“, плувен басейн в Благоевград, дом на покойника в Орешак, както и нереализираните идейни проекти за широкоформатно кино, дискотечен комплекс и кафене над реката в Перник. Нестандартните функционални и конструктивни решения и градоустройствен подход на архитект Петър Радев се допълват от художествения синтез в резултат от дългогодишната съвместна работа със скулптора Борис Живков и художниците Филип Седефчев и Мария Горова. [7]

Автор на художествената декорация на ритуалната зала на читалището в с. Извор е значимият творец и носител на орден „Кирил и Методий“ II степен, автор на множество художествено-приложни проекти в района на Перник и в национален мащаб през втората половина на XX в. Филип Седефчев. Седефчев (1935-2021) завършва „Живопис“ в Художествената академия при проф. Ненко

Балкански и през дългогодишния си творческия си път има реализации в разностранни области като изложбения дизайн – национална изложба „Знаме на мира“, ежегодна изложба на Техническо и научно творчество на младежта /ТНТМ/ в Перник и др., на пространствен дизайн - ритуалните зали за бракосъчетания в Перник и Своге, домовете за траурни ритуали в Радомир и село Орешак и др., емблемен дизайн - на център „Знаме на мира“, „Кукери и сурвакари“, ТНТМ и др., монументална стенопис – в Радомир, Орешак, Одеса, приложната графика, дърворезба и дървена пластика – Радомир, Ковачевци, Орешак, стъклописи, каменна пластика, металопластика, живопис. [8]

За един от обектите на художествената декорация в читалището в село Извор, Филип Седефчев работи съвместно с Асен Старейшински (1936 - 1991), носител на почетната годишна награда на СБХ „Александър Жендов” за постижения в приложната графика (1974), орден „Кирил и Методий” III степен – 1975 година и званието „Заслужил художник” (1982), който работи предимно в областта на плаката, но също така създава и множество запазени знаци, филателна графика, дизайн на монети, оформления на книги и илюстрации. През 1961 годиан Старейшински завършва Националната художествена академия, специалност „Плакат” в класа на проф. Александър Поплилов. Дългогодишен член на ръководството, на секция „Приложна графика”; член на Управителния съвет на СБХ; член на Съюза на артистите в България. Последователно е художник в Народния театър, ВИТИЗ, главна дирекция „Български циркове”, държавно издателство „Септември”; Театъра на Българската армия. Негови произведения са представяни във всички значими национални изложби и конкурси в областта на плаката и графичния дизайн, както и в редица международни изложения и представителни изложби на българския плакат по света. [9]

Инженер Елисавета Панова (1945) започва работа в Проектантската организация в Перник след своето дипломиране от Висшия инженерно-строителен институт през 1968 година. След закриването на Проектантската организация през 1992 година започва работа като преподавател по учебните дисциплини "Стоманобетон" и "Строителни материали" в Техникум по строителство "Арх. Йордан Миланов" в гр. Перник. Преподава в техникума до своето пенсиониране. Инженер Панова продължава да работи и към настоящия момент, като проектант по част "Конструкции" на жилищни сгради. Проектант е и на сградата на народно читалище "Иво Войвода - 1928" - село Долна Секирна, община Брезник през 1968 година. Според инженер Панова, читалищните сгради са едни от най-трудните за проектиране, заради големите подпорни разстояния и големите полезни натоварвания.

4. Архитектурна концепция и реализация

Читалищните сгради от различи периоди в страната съвместяват различни функционални програми като най-големите градски читалища разполагат с добре оборудвани акустични зали със стотици места за изпълнения на местния читалищния хор, фолклорен ансамбъл, духов оркестър или симфоничен оркестър или киносалон. Функционалната програма на читалището в село Извор, която

включва две различни по големина конферентни зали, ритуална зала и библиотека го определя като значим културен център в района на Радомир.



Фиг. 1 Изглед от югозапад; 2023 г.



Фиг.2 площадът с перголата и вътрешния двор; 2023 г.

За разлика от по-малките по обем, непретенцозни, често типови проекти на останалите десет селски читалища, изградени през периода 1945-89 година в района на град Радомир, построената по индивидуален проект сграда на читалище с ритуална зала в село Извор се отличава със своите размери и модерна архитектура (фиг. 1 и фиг. 2) и съперничи по естетически качества, техническо изпълнение и функционална програма със сградите на водещите културни институти в Пернишка област от същия период - читалище "Просвещение - 1870" в град Брезник (1969), Младежки дом "Иван Стойнев" в град Земен (1980-те), Пионерски дом (1982) в град Перник, читалище "Гюрга Пинджурова - 1895" в град Трън (1984) и Младежки дом в квартал Мошино на Перник (1985).



Фиг. 3 северозападна фасада и прозорците на зрителната зала; 2023



Фиг. 4 общ изглед на двете тела заедно с площада; 2023

От южната страна на площада, отделен от преминаващата през селото транспортна артерия, в парковата среда на парк „Извор“ е разположен Паметникът на загиналите в борбата против фашизма и капитализма и в Отечествената война (1944-45) с проектанти архитект Георги Стоилов (1929 - 2022) и скулптора Любен Димитров (1904 – 2000). Мемориалът реализира първата градоустройствена намеса в духа на следвоенния модернизъм в село Извор. Читалищната сграда е разположена срещу мемориала (фиг. 3) и несъмнено е съобразена със съществуващия от около десетилетие и половина паметник като допълва и завършва градоустройствения замисъл на мемориала в центъра на селото.

Читалищната сграда, разположена в северната част (фиг. 4) на селския площад, заема доминираща спрямо разположената от изток по-малка по мащаб сграда на кметството. Сградата е умело проектирана като се съобразява със съществуващата денивелация на терена в посока север-юг и разполага с един етаж по-малко към площада.

Читалището е построено на мястото на селската чешма, която е изместена на около 200 м в северозападна посока. Като реминисценция архитект Радев проектира малка чешма състояща се от три призматични обема в югозападния ъгъл на имота.

Настилката на площадното пространство около читалището е изпълнена по индивидуален проект от шахматно подредени мозачени плочи, конгломерат от мраморни блокчета. (фиг. 5 и 6) Около сградата – в партерите на площадното пространство и във вътрешния довр, е засадена храстовидна и дървестна иглолистна растителност.



Фиг. 5 Перголата, ограничаваща вътрешния двор; 2023



Фиг.6 Поглед към читалището от югоизток; 2023

Обемно-пространствената композиция, съставена от призматични обеми, организирани около вътрешен двор (атриум), е позната и от други сгради на Петър Радев. Подчинената на функционалното зониране обемна композиция на читалището е решена като два отделни обема, които на практика функционират отделно. Западната част, която включва две зали – изпълнителска и конферентна, а източната част е заета от ритуална зала, заведение, библиотека и художествени ателиета. Двата обема са отделени композиционно посредством широко 4 м стълбище, осъществяващо прехода между площада пред читалището и задния двор на север, достъпно през сравнително тесен отвор в облицованата с керамични плочки екстериорна стена, играеща ролята на кулиса. От друга страна, двете функционални зони на читалището са обединени чрез вътрешния двор, обособен чрез стоманобетонова пергола пред остъклената югозападна фасада на Г-образното ниско тяло, който осигурява известно уединение на провежданите сватбени церемонии.

Високият обем включва разположени изпълнителска зала за 240 зрители с площ 160 кв. м., достъпна през фоайе с площ 90 кв.м. при главния вход на нивото на площада (фиг. 7 и 8). Основната изпълнителска и кинозала на читалището е оборудвана с кинокабина над входа на залата, повдигната сцена с горна сценична

механизация за декори и окачен таван по индивидуален проект. Залата е обслужвана от две гримьорни със самостоятелни санитарни възли на същото ниво. На полувкопаното долно ниво са разположени репетиционна зала с площ 50 кв.м., както и по-малка зала за около 100 места с площ 125 кв.м., санитарни възли към голямата зала и различни инсталационни помещения .



Фиг. 7 Зрителната зала – поглед към сцената; 2023



Фиг. 8 Зрителната зала - поглед от сцената; 2023

Ниското тяло се състои от полувкопан от запад партер, зает от три художествени ателиета за творческа работа с деца, носещи имената „Другарче“, „Славейче“ и „Изворче“, осветени на север за сметка на денивелацията на терена (фиг. 9 и 10), в които се провеждат школи по народни и класически инструменти, фотоклуб, художествено и приложно изкуство. Етажът, достъпен на нивото на площада пред читалището е зает от зала за радостни граждански ритуали със заведение към нея, конферентна зала и библиотека. Радев проектира специално отделен вход и изход на залата за бракосъчетания, символизиращи съвместният път на младоженците в една и съща посока без връщане назад. Еркерното наддаване на обема на ритуалната зала излиза над полувкопания етаж с детските ателиета е подчертан чрез цветния контраст на фасадите и двете шлицови структурни остъклявания. Втората функционална зона в източното ниско тяло разполага с отделни санитарни възли на партера и инсталационни помещения на приземния етаж.



Фиг. 9 Общ изглед на североизточната фасада; 2023



Фиг.10 Югоизточна фасада на ниското тяло; 2023

Цветната фасадна композиция на читалището в с. Извор, позната и от други сгради на Петър Радев като Младежкия дом в Перник, е базирана на контраста

между пресичането на облицованите в червени керамични плочки, имитиращи тухлена зидария фасади, фасадните повъхности, облицовани с бял мрамор, мазилка или мозайка, както и структурните остъклявания (фиг. 11 и 12).



Фиг. 11 Главният вход на високото тяло с обемно-пластичното пано от триъгълни призми от Филип Седефчев; 2023



Фиг.12 Югоизточна фасада на зрителната зала, увредената облицовка от мраморни плочи от течовете от водосточните тръби; 2023

Масивните хоризонтални покривни бетонови бордове, облицовани с бяла мозайка, подсилват усещането за стабилност на композицията и контрастират с вертикалното членение на прозорците на високия обем от запад (фиг. 3).

Главният вход на читалището, разположен на югоизточния край на сградата е оформен като самостоятелен обем към високото тяло и е маркиран с екстериорно геометрично стилизирано обемно-пространствено пано (фиг. 1 и 11), съставено от матрица от триъгълни призми, облицовано с бял мрамор, което заема фасадната плоскост в дясно от главния вход.

Тържественият вход към ритуалната зала за брачни церемонии е подчертан с типично модернистичен похват чрез силно издадената конзолната козирка, облицована с бяла мозайка.

Сред характерните за творчеството на архитект Петър Радев елементи, познати и от други негови сгради са трапецовидната арка на отвора в свързващата двата функционално самостотелни обема стена-кулиса, увенчана с три декоративни, облицовани в бяло конзоли, както и силно издадените конзолно капчуци.

Цветарниците, проектирани и изпълнени специално за читалищната сграда в Извор, също участват активно в цветовото решение като с цвета и формата си наподобяват с покривните бордове и са разчленени от декоративни полуколони, подобни на декоративните конзоли над трапецетовидната арка към откритото стълбище.

5. Художествен синтез в интериора

Интериорът на сградата, изпълнен по индививидуален проект, е решен в типичния за епохата стил под синергията на различни видове декоративно-приложни и пластични изкуства. Художествената украса на сградата включва модерни стилизации на традиционни форми предимно с фолклорен сюжет под

формата на екстериорни декоративни стенни пана, интериорен стенопис, дърворезба, декоративни стенни облицовки и решетки, окачени тавани, осветление, мебели и др.

Характерен белег за периода на втората половина на XX в. е структурната декоративна мазилка върху окачен таван от рабицова мрежа, изпълнена в коридорите, библиотеката и бившето заведение в читалището. Дъгообразна форма около кристалния полилей е оформена чрез гладка мазилка върху рабицова мрежа.

Друг характерен елемент при оформяне на интериора през втората половина на XX в. е употребата на дървени ламперии и решетки. Ритуалната зала е облицована с дървена ламперия в цвят светъл дъб в комбинация с мокет в маслено зелен цвят, използван и за облицовка на стените. Типичните за този период на строителство декоративни решетки на радиаторите на отоплителната инсталация също са изпълнени от дървени плоскости.

В зрителната зала е изпълнен оригинален окачен таван състоящ се от зелени четирилъчни звезди и бели квадрати и пресечени пирамиди от дървени плоскости по проект на архитект Радев (фиг. 16).



Фиг. 15 Декоративна решетка от стоманени шини от Филип Седефчев; 2023



Фиг.16 Окачен таван в зрителната зала по проект на арх. Петър Радев; 2023

От оригиналното обзавеждане на читалищната сграда, осъществено по индивидуален проект, към настоящия момент са запазени голямата дървена маса, състояща се силно двустранно конзолно издаден дървен плот, подпрян върху ограничена по размер цилиндрична дървена основа по проект на архитект Петър Радев, масивният кристален полилей върху мазилков постамент със сложна криволинейна повърхнина и дърворезбованият герб в ритуалната зала (фиг. 13), декоративните таванни плоскости, висящи от тавана за оформяне на осветлението в бившето заведение (място за почерпка), което днес функционира като хранителен магазин (фиг. 14).



Фиг. 13 Полилеят и конзолната дървена маса по проект на арх, Петър Радев в ритуалната; 2023



Фиг.14 Декоративни дървени елементи на тавана в търговското помещение, по проект на арх. Петър Радев; 2023

Останалата част от художествено - пластична украса на сградата е поверена на художника Филип Седефчев, който работи в екип по един от елементите съвместно с плакатиста Асен Старейшински. Филип Седефчев, който е предпочитаният от архитект Петър Радев декоратор и работи върху множество от проектираните от него сгради, играе основната роля при проектиране и изпълнение на декоративно-пластичните елементи на читалището в село Извор:

- релефно - пластично пано състоящо се от триъгълни призми с различна големина изпълнено в дясно от главния вход с размери 566 х 325 см (фиг. 11);
- декоративна решетка от стоманени шини, наподобяваща народна шевица и монтирана в кръгъл отвор с диаметър 230 см, монтирана във фоайето пред зрителната зала и чрез отвора, в който е монтирана косвено се осветява стълбището водещо към долното ниво (фиг. 15);
- дърворезобован герб на Народна Република България в ритуалната зала (фиг. 13).

Филип Седефчев, който има над 40 реализирани стенописа, работи в съавторство върху стенописа с размери 830х240 см, разположен на височина 245 см от пода на югоизточната стена на ритуалната зала. Стиловите белези на стенописа на тема "Живот", изобразяващ стилизирана млада двойка с фолклорени елементи, подсказват, че водещ автор на проекта, а вероятно и на реализацията , е Асен Старейшенски като това е единственият известен стенопис, изпълнен от утвърдения плакатист (фиг. 17).



Фиг. 17 Стенопис в ритуалната зала на тема "Живот" творба на художниците Филип Седефчев и Асен Старейшенски; 2023



Фиг.18 Вход и изход на ритуалната зала, допълнително маркирани с декорация напомняща арки; 2023

6. Конструкции, конструктивна система

Конструкцията и на двете тела на сградата е рамкова, стоманобетонна. Ограждащите (фасадните) и вътрешните стени са изпълнени от решетъчни тухли. За изпълнение на стоманобетонната конструкция в проекта е предвиден бетон с марка Б150 с призмена якост на натиск 65 kg/cm2.

По данни от инженер Панова конструкцията на сградата не е оразмерявана на сеизмични въздействия - това обяснява и големия еркер с ритуалната зала с дължина 245 cm. За сеизмичното осигуряване на сградата са приложени конструктивните мерки предписани в Правилник за строителство в земетръсни райони от 1964 година. Високите зидове на сценичната кула и зрителната зала са допълнително укрепени с хоризнотални стоманобетонни пояси.

Конструктивните решения следват архитектурния замисъл. Конструктивното решение на декоративната пергола, ограждаща двора пред ниското тяло също изцяло съответства и подкрепя замисъла на архитектурния дизайн. Колоните с многоъгълно напречно сечение са обединени от ригели и свързани с вути с цел рамките да наподобяват на арки.

Двата читалищни обема са покрити с плоски покриви като покривните плочи над зрителната зала и сценичната кула са наклонени с цел осигуряване на наклон за отичане на водата и олекотяване на товара върху тях чрез избягване полагането на бетон за наклон. [10]

При проектирането на сградата е приета регулярна конструктивна схема по височина, при която стоманобетонните елементи преминават непрекъснато от фундамент до покрив. Плочите са гредови като под вътрешните стени са изпълнени носещи греди. Заведението в ниското тяло е покрито със стоманобетонни рамки, развити по късата страна на сградата, разположени през разстояние 205 см една от друга, като стойката към фасадата е изпълнена с по-голямо напречно сечение по естетически съображения. Поради трапецетовидната форма в план на завдението, светлият отвор на рамките варира от 690 см до 570 см. Зрителната зала също е покрита със стоманобетонни рамки, развити по късата страна на залата през разстояние 330 см и светъл отвор 1170 см. Поради наклона в покривна плоча, ригелите са с променливо напречно сечение. Сценичната кула е

покрита със стоманобетонни рамки, развити перпендикулярно на рамките в зрителната зала на 250 см една от друга и светъл отвор 790 см. Ригелите също са с променливо сечение поради наклона на покривна плоча. Подът на зрителната зала, разположена върху по-малката зала (салон за събирания) е изпълнен с наклон за осигуряване на оптимална видимост и е подпрян с колони във вътрешността на партера. Фасадата на партерно ниво на салона за събрания е прибрана навътре с 2,40 м за редуциране на подпорните разстояния, стойките на носещите покривната плоча рамки остават видими. С изместването на фасадата навътре и добавянето на видими колони в салона за събрания, стойките на рамките в зрителната зала се облекчават от товара който, носен от плочата на зрителната зала. Товарът от плочата на зрителната зала на 90% се поема от фасадната стена и видимите колони в салона за събрания.

Сутеренните етажи и на двете тела са изпълнени от бетон до нивото на плочата на първия надземен етаж. Фундирането е изпълнено съобразно геоложките дадености на терена. Геоложкият доклад не е запазен, но по спомени на проектанта инженер Панова, за всяка сграда разработвана в проектантската организация винаги е съставян геоложки доклад. Фундирането е изпълнено чрез плоско , от ивични (под сутеренните стени) и единични фундаменти под стойките на рамките и вътрешните колони. При фундирането е спазено правилото за фундиране на едно ниво.

7. Физическо състояние и намеси в оригиналния интериор и екстериор на сградата.

За сградата на читалището в село Извор, която функционира над 40 години, не е предприеман основен ремонт.

Основните намеси във времето са подмяна и възстановяване на стареещото и липсващо покритие на плоския покрив на ниското тяло, в което функционира постоянно хранителният магазин в пространството на бившето заведение (място за почерпка) към ритуална зала.

Поради недостиг на средства ремонт на високото тяло не е правен. В резултат на това, оригиналният интериор в зрителната зала е силно увреден от течовете, но е възможно да бъде възстановен. Столовете в зрителната зала, които са амортизирани и негодни за употреба, са произведени от българското мебелно предприятие "Тонет", основано през 1926 година, което съществува до днес и е възможно да бъдат заменени с идентични на оригиналните.

Увреден от течове и избили соли е стенописът "Живот" на югоизточната фасада в интериора на ритуалната зала, който се нуждае от консервация. Дървените ламперии и радиаторните решетки се нуждаят от рехабилитация, мокетите са амортизирани и се нуждаят от подмяна.

Листовите настилки в интериора са амортизирани и се нуждаят от подмяна – линелеум в зрителната зала и конферентната зала в ниското тяло, мокет в ритуалната зала и библиотеката, сладкарницата (заведението за почерпка) е с теракотни плочи. Трайните настилки в интерира - мозайката в коридора пред ритуалната зала мраморната настилка пред зрителната зала, както и стълбището

което води към малкия салон под зрителната зала и фоайето пред малкия салон са с мраморна настилка, които са в сравнително добро състояние и следва да бъдат рехабилитирани и запазени.

Оригиналната алуминиевата дограма на структурните остъклявания е запазена с изключение единствено на входната врата на магазина (бившето заведение), няколко единични прозорци със стоманени рамки на библиотеката, конферентната зала и в сутерена са подменени с PVC дограма.

При незавършен ремонт в малката зала за около 100 човека във високото тяло през 90-те години на XX в. с цел да бъде приспособена за дискотека, са демонтирани всички части на интериора до груб строеж - малката сцена, настилката по пода, столовете, облицовките по стените и осветителните тела, така че в момента залата е неизползваема.

Обрушени са мозаечната облицовка на бетонните бордове на покрива, фасадната мазилка, облицовката от керамични плочи и мраморни плочи като в най-голяма степен е увредена североизточната фасада, зоните около водосточните тръби по фасадите във вътрешния двор. Участък от цокъла на фасадата е засегнат от кореновата система на иглолистното дърво, засадено в дясно от главния вход. Мазилката на бетоновата пергола е единственият подновен фасаден материал след изграждането на читалището. Металните казанчета на водостоците/капчуците са силно кородирали.



Фиг. 19 Североизточна фасада със сценичната кула с увредената фасадна облицовка от течовете от водосточната тръба, 2023



Фиг. 20 Североизточната фасада с изнесения обем на ритуалната зала и увредената фасадна облицовка от течащата вода от барбаканите, 2023

Силно обрушената мозечна настилката на площадното пространство около читалището подлежи на ревизия и подмяна. Засадената храстовидна и дървестна иглолистна растителнос, както и чешмата са в добро техническо състояние.

Може да бъде обобщено, че промените по време на експлоатацията на читалището са минимални и напълно обратими. За поддържане и възстановяване на оригиналния обликн на сградата е необходимо привличането на средства по различни програми.

В същото време в страната е актуална програма за финансиране на проекти за въвеждане на мерки за енергийна ефективност на читалищата. Популярните в момента, предлагащи достъпен финансов ресурс програми за енергийна ефективност представляват сериозно предизвикателство пред съхраняването на оригиналния архитектурен облик на обектите от следвоенния модернизъм, тъй като крият риска от обезличаване и загуба на характерни стилови белези в екстериора. Този съществен риск при прилагане на мерките за енергийна ефективност, които са технологично несъвместими със запазването на оригиналните екстериорни фасадни облицовки от втората половина на XX в., е илюстриран с примерите от района на читалищните сгради в град Трън, град Брезник и село Ноевци, които са с изцяло променен екстериорен облик след изпълнение на проекти за енергийна ефективност. [11] [12]

8. Заключение. Оценка, препоръки и предложение за статут.

От основаването си преди почти век народно читалища "Отец Паисий-1927“ в село Извор играе водеща културна роля в района на Радомир и Перник. Читалищната сграда, отркита през 1980 година и разположеният в непосредствена близост мемориал (1966) са основните елементи на модерния градоустройствен облик на селищния център.

Читалището с ритуална зала село Извор се отличава от останалите скромни селски читалищни сгради, изградени през епохата на социализма, и носи типичните стилови и функционални белези на следвоенния модернизъм на представителните градски културни центрове в района и страната. Водеща характеристика на сградата, дело на опитен проектантски екип, включващ имена на водещи художници на местно и национално ниво, е синтезът на изкуствата в интериора и екстериора на сградата.

Значителната архитектурно-строителна, културна и художествена стойност на разположеният извън градски център културен институт, обуславят предложението за предоставяне на защитен статут по Закона за културното наследство на читалище „Отец Паисий -1927“ в село Извор като единична архитектурно-строителна и художествена недвижима културна ценност с категория от местно значение.

В същото време сградата не е качествено поддържана поради липсата на средства в над четиридесет годишния период на експлоатация. Определянето на защитен статут за сградата по Закона за културното наследство на читалището в село Извор и определяне на адекватни режими за опазване, ще подпомогне съхраняването на автентичния облик на сградата и предотврати унищожаването на този ценен представител на следвоенния модернизъм.

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ABOUT THE HISTORY OF STAMBOLOV'S BRIDGE IN VELIKO TARNOVO AFTER 132 YEARS OF SPANNING OVER YANTRA RIVER AND PROJECT FOR STRENGTHENING

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***Abstract**: Stambolov's bridge in Veliko Tarnovo, which used to serve the two-way pedestrian, livestock and later motor traffic, between the railroad station and the town,marked the beginning of the modern bridge construction in Bulgaria. The research focuses on the description of the specific spatial construction of the road facility, which corresponds the architectural idea: joint-rod system with a horizontal upper belt, a lower belt with a curvilinear parabolic outline and a grid of verticals and falling diagonals loaded with tensile forces.*

Key words: *single-span arched, iron riveted bridge, joint, rod, Veliko Tarnovo*

1. Introduction

The proficiency of the architectural and engineering solution of the 130 years old precast wrought iron arch single-span truss bridge in Veliko Tarnovo, named after Stefan Stambolov, which is virtuously embedded in the unique landscape of the old Bulgarian capital, represents a profoundly instructive example for the bridge specialist over the world. (Fig. 1a, b, c). Lightness and elegance of the subtle metal lattice openwork of the bridge structure suggest to the visitors that beyond its routine function it was designed to blend with the surrounding environment. The construction history of the road facility was connected with the government of Stefan Stambolov, Prime Minister of the liberated Bulgarian State (1887-1894), who was born in Veliko Tarnovo. The decision for construction of the European styled bridge of significant size over the Yantra River for connection with the still intended project railway station was resolved by the Minister Council in 1889, about a decade after the Russian-Turkish war (1877-78) that resulted in the liberation of the Bulgarian state. [1] [2].

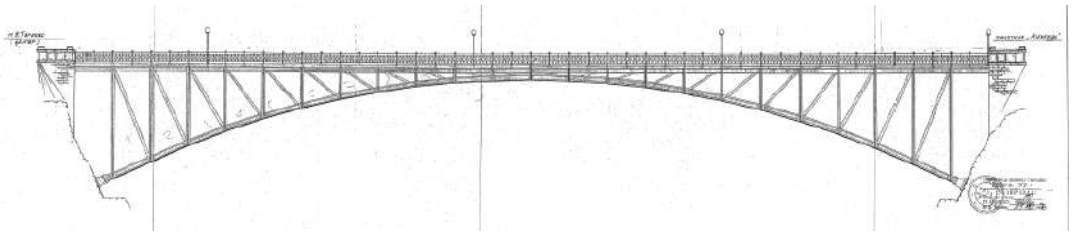
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1a)



1b)



1c)

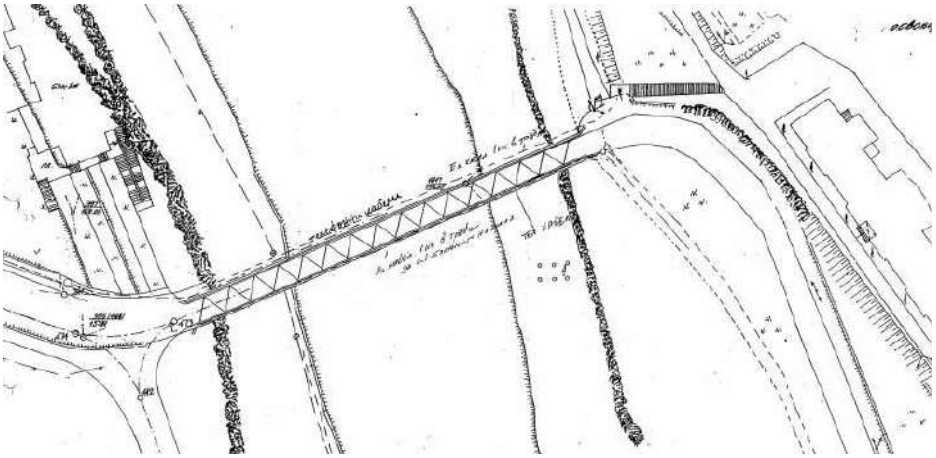
Fig. 1. a) General view of Stambolov's bridge; b) View from the monument of Assenev brothers; c) View from the surface of the river

2. Structural description of the bridge

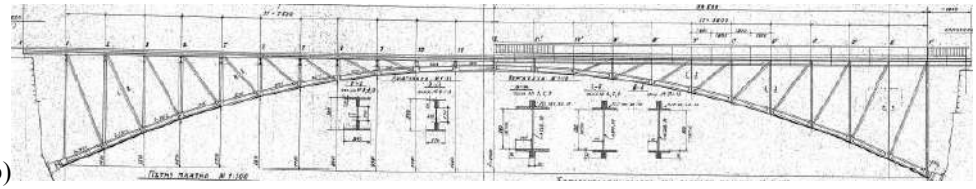
The presented constructive analysis is based on the project for Rehabilitation and Reinforcement of the Stambolov bridge from 1996, contracted by the brunch of "Moststroy" Ltd. in Veliko Tarnovo and designed as subcontractor by the team of structural engineers of "Geotechnika" Ltd., Dragoman with leading engineers ass. prof. eng. Kiril Angelov and eng. Kostadin Tsekov and under the supervision of prof. eng. Peter Staykov. The project was approved by the Municipality of Veliko Tarnovo, but was not executed [4]. The structural analysis of Stambolov's bridge is part of the efforts of the engineering specialists in Bulgaria for investigation, rehabilitation and maintenance of metal bridges from the end of 19th and the beginning of 20th centuries [5] [6] [7] [8] [9]. The metal structure of the bridge is supported on two massive stone masonry abutments, founded on the rocky banks of Yantra River. The main bearing structure of the bridge features upper roadway and consists of 22 fields. The total length of the bridge is 79.20 m. The width of the carriageway between the parapets is 7.0 m (Fig. 2.2a, 2b). The bridge

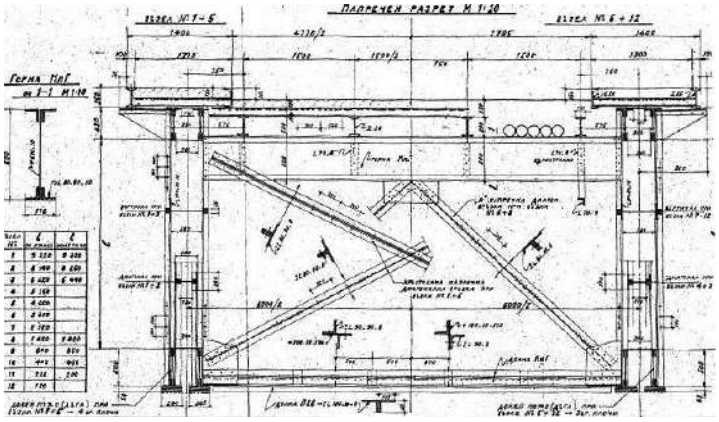
structure consists of combination of two trusses, distanced at 6.00m axially from each other. The overall structural height of the truss is 6.90 m at the abutments and 1.20 m at the key. The cross-section of the upper belt is constructed of two joint T-shaped riveted structural elements, placed axially at a distance of 350 mm from each other. They consist of two L-profiles: 2L 90/90/10 mm for flanges and a strip wrought iron stem with dimension 10/410 mm. (Fig.2b).

2a)



2b)

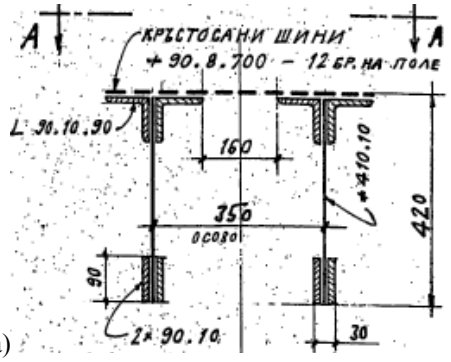




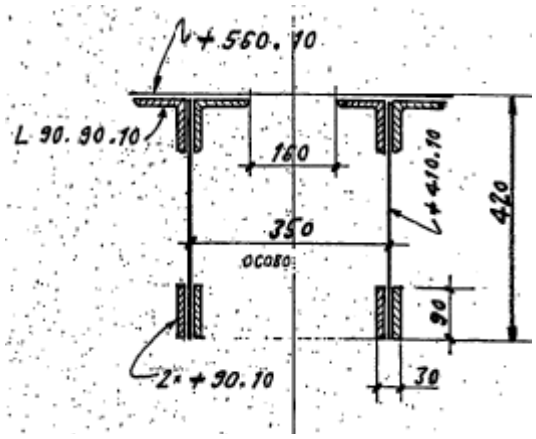
2c)

Fig. 2. a) Technical situation of the bridge [4], b) Longitudinal structural view of the bridge [4], c) Cross-section of the upper belt [4]

The upper surfaces of the truss flanges are additionally reinforced in their free arms range: nodes 0 to 6 with cross rails with a strip steel plate with dimensions 90/8/700 mm (Fig.3a) and nodes 6 to 12 with a strip steel plate with dimensions 10/560 mm (Fig. 3b). The stems are additionally reinforced at their free ends with two strip steel plates with dimensions 2 x 10/90 mm (Fig. 3a and 3b).



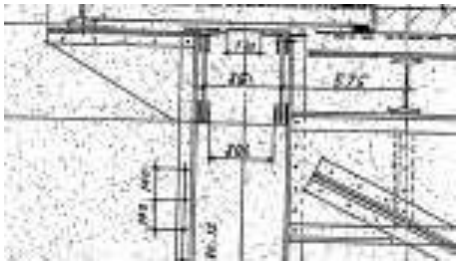
3a)



3b)



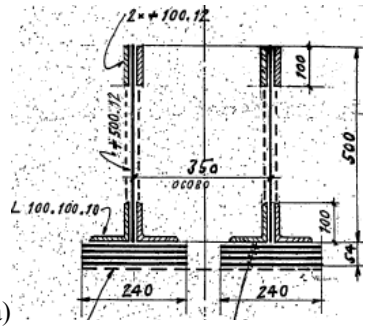
3c)



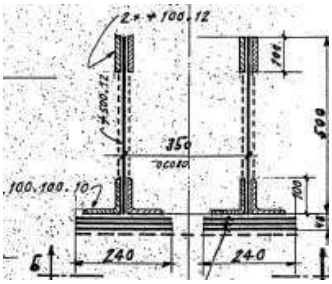
3d)

Fig. 3 a) Upper Belt (nodes 0-6) [4], b) Upper Belt (nodes 6-12) [4], c) View of upper belt (Photo: D. Partov, 2009), d) Upper belt detail [4]

The cross-section of the lower belt is constructed of combination of two "T"-shaped riveted structural elements, placed 350 mm axially from each other, facing upwards. They are executed by the combination of two corner profiles 2L x 100/100/10 for flanges and a strip steel stem with dimensions 500/12mm (Fig. 4b). The stems are additionally reinforced at their free ends with two strip steel plates with dimensions 2 x 12/100mm (Fig. 4a and 4b). The corner profiles of the lower belt between nodes 1 and 5 from the lower end are mounted by four reinforcing wrought iron plates with dimensions 13/240mm. On the corner profiles of the lower belt between nodes 6 and 12 are mounted three wrought iron plates with dimensions 15/240mm.

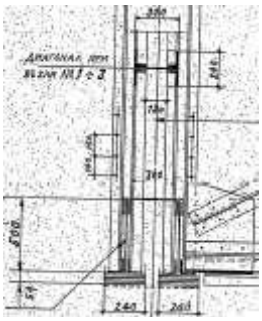


4a)



4b)

4c) 

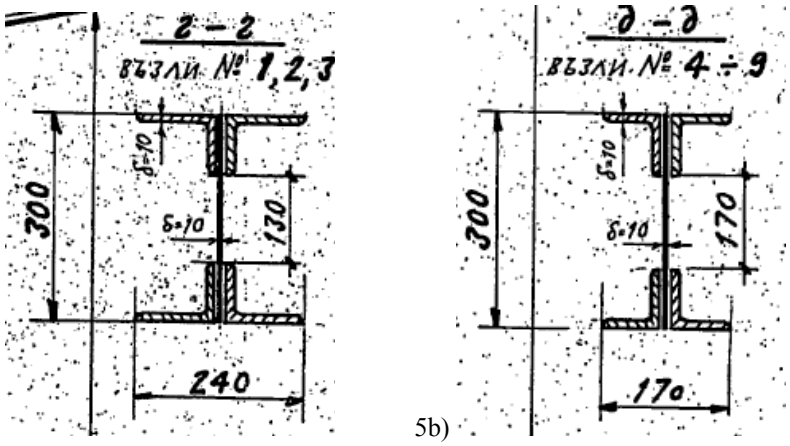
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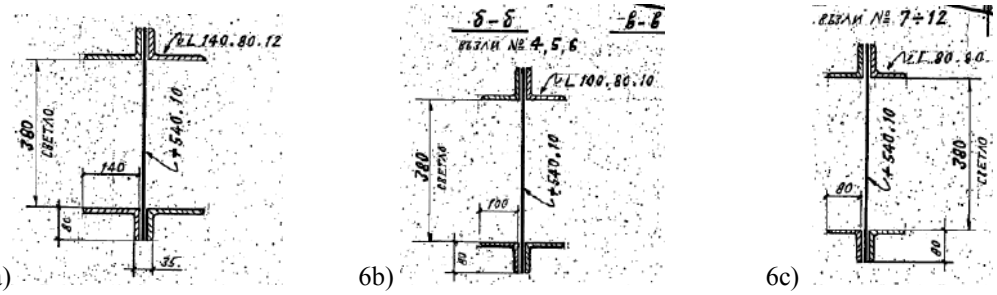
Fig. 4. a) Cross-section of the lower belt between nodes 1-5 [4], b) Cross-section of the lower belt between nodes 6-12 [4], c) View of the lower belt (Photo: D. Partov, 2009), d) Bottom waistband detail [4], e) View of the lower belt (Photo: D. Partov, 2009), f) View of the lower belt (Photo: D. Partov, 2009)

The joint behavior of thus created "U"-shaped cross-sections of vertical load impacts (and horizontal impacts) is realized by combination of two profiles with transverse ribs, positioned in the middle of the truss fields and connected to the bars of the truss grid. (Fig. 4). In ddition, from the side of the flanges between the two adjacent "T"-shaped profiles (as indicated above in the text) a winding connection of strip wrought iron of size 90/8/700 mm cross rails is developed; respectively 12 pieces per field in the upper belt of size 90/8/900mm and 5 pieces per field in the lower belt). The plates that connect the truss diagonals (Fig. 5a, 5b) and verticals (Fig. 6a, 6b) and the upper and the lower belts are of sufficient height for realization of the links without knot. These details are realized with riveted joints. The open box section construction of the two belts enables the verticals (Fig. 6a, 6b and 6c) 2L x 140/80/12 for nodes № 1, 2, 3; 2L x 100/80/10 for nodes № 4, 5, 6 and 2L x 80/60/10 for nodes №7-12 and the diagonals, to be executed as two-part combined profiles with an "I" cross-section. (Fig. 5a, 5b).



5a) 5b)

Fig. 5. a) Cross-section of the diagonals between nodes 1, 2 and 3 [4] b) Cross-section of the diagonals between nodes 4-9 [4]



6a) 6b) 6c)

Fig. 6. a) Cross-section of the verticals between nodes 1-3[4] b) Cross-section of the verticals between nodes 4-6 [4] c) Cross-section of the verticals between nodes 7-12 [4]

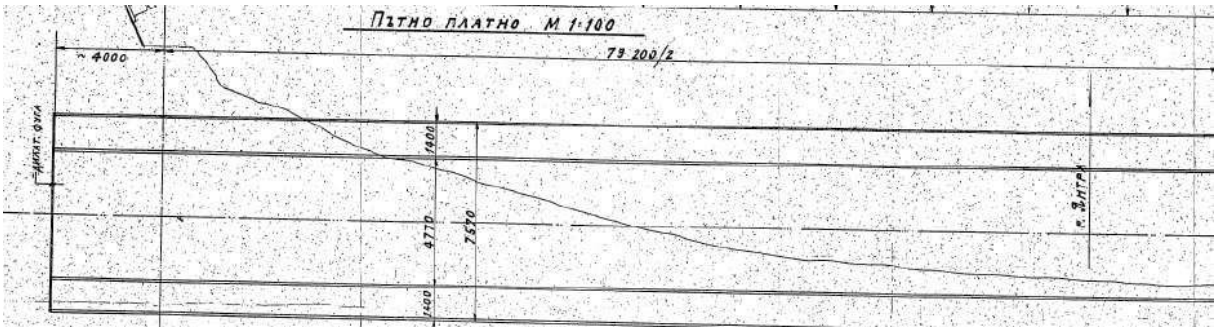
The belts of these diagonals are executed by two unequal angular profiles of size 2L x 70/120/7 mm, fixed by rivets to each other and a connecting stem of 10 mm thick wrought iron strip. The both ends of the truss diagonals, together with their flanges, enter the 300 mm joint between the stems of the upper and lower truss belts. The both ends of the truss verticals are interrupted above the belt stems, the protruding flanges of the verticals on the external side are riveted to the truss belts. Transversal beams of "I"-shaped cross-section, combination of a stem of strip wrought iron of size 10/600 mm and an upper and lower flange of paired angle profiles of 2L x 80/80/10 mm, riveted to each other, are installed between the nodes of the truss upper belt. The transverse beams are of height 600 mm. (Fig. 7a, b, c).



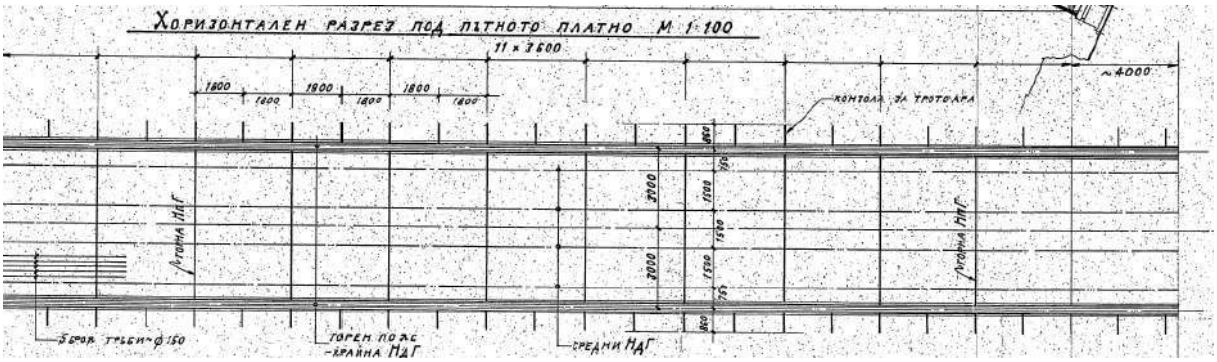
a) b) c)

Fig. 7. Composition of transverse and longitudinal beams: a) bottom view b), condition of the girder of the roadway in 1995, c) Cross section of the main cross beam [4], (Photos: D. Partov, 2009)

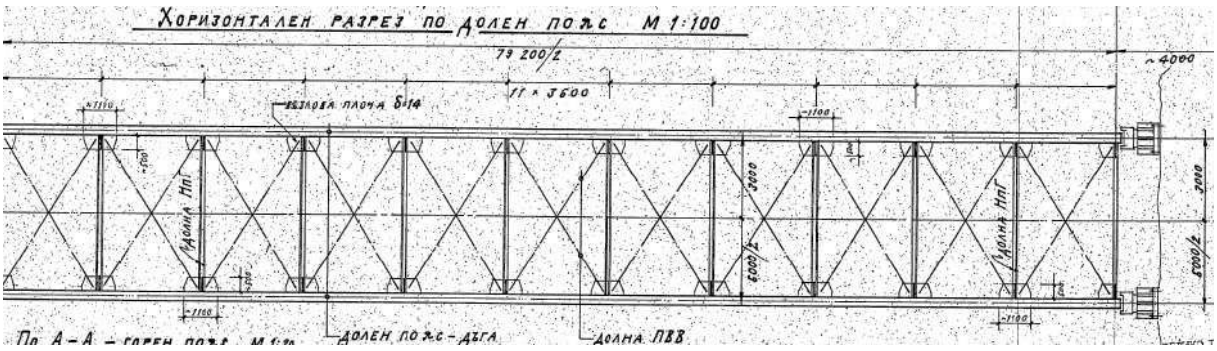
Longitudinal full-wall steel beams with "I-26" shaped cross-section are designed between the transversal beams. They are positioned on the upper belt of the transverse beams, on distances of 1500 mm, equal to the axes openings of 3.60 m between the main truss transverse beams. Thin prefabricated reinforced concrete panels are laid on the longitudinal bridge beams, which are part of the road slab. Asphalt and concrete pavement of 5 cm thickness is laid on top of the prefabricated panels. The height of the arched trusses at the supports is 7.0 m. The upper girdle of the bridge step on the stone abutments and is fixed by stationary wrought iron cylindrical joint bolt. The stepping of the lower girdle of the bridge on the stone abutments is realized by joint bolts. The spatial stability of the bridge is ensured by horizontal X-shaped wind-resistant connections located between the upper and lower truss belts as well as by the vertical Xshaped wind-resistant connections, installed between the verticals of the two adjacent loadbearing bar structures of the bridge.



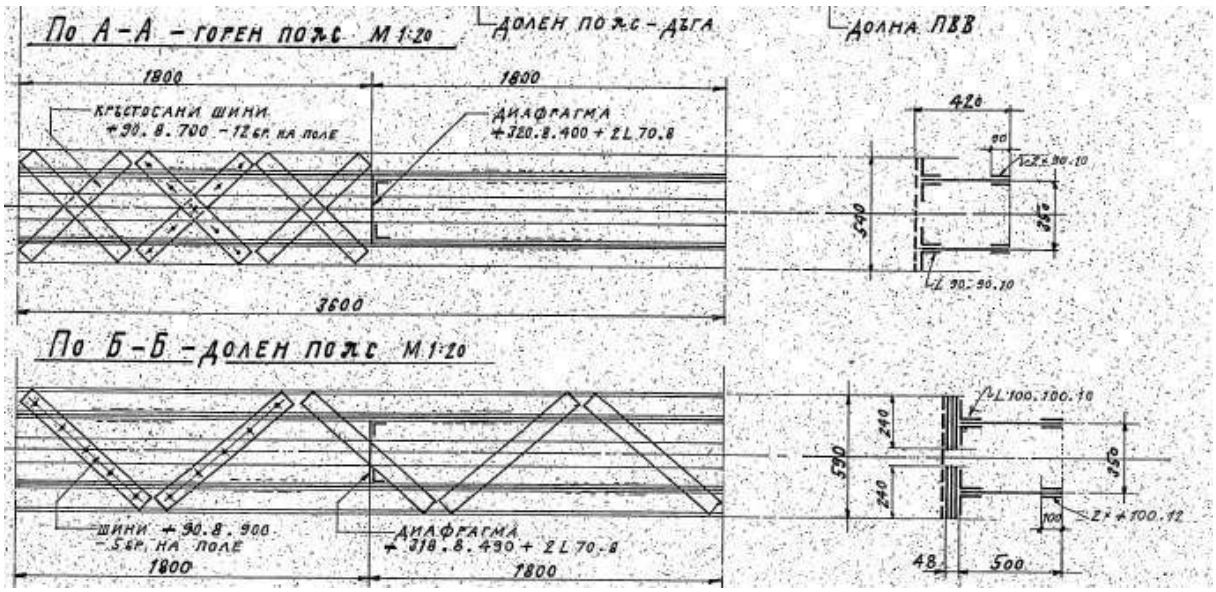
8a)



8b)



8c)



8d)

8e) 

8f) 

Fig. 8. a) Plan of the road lane of the bridge [4], b) Section of the road lane of the bridge [4], c) Cross section of the lower belt [4] d) Details of construction of the lower and upper belts [4], e) View on the lower belt, f) View of the lower and upper belt, (Photos: D. Partov, 2009).

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At 1996 a project for the rehabilitation of the Stambolov's bridge is prepared [9]. Two basic variants for replacing the deteriorated parts of the deck are investigated – replacement with orthotropic steel deck and with reinforced concrete deck (Fig. 9) and (Fig. 10). However up to now rehabilitation works are not performed and the deterioration of the bridge structure due to lack of maintenance is in progress.

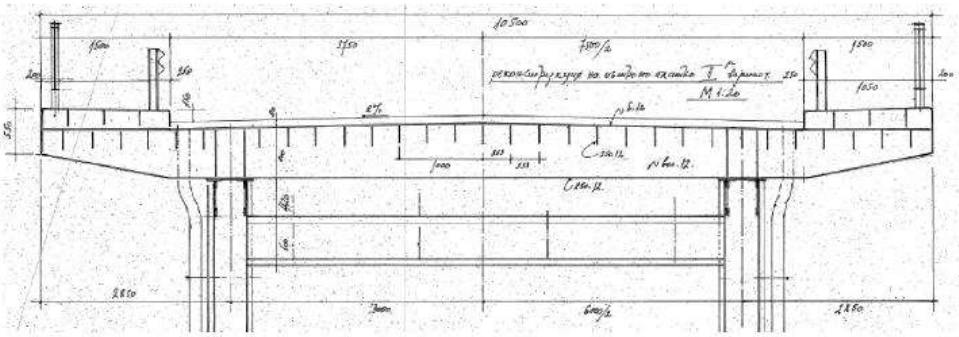


Fig.9. Variant for the deck replacement with orthotropic steel deck [9]

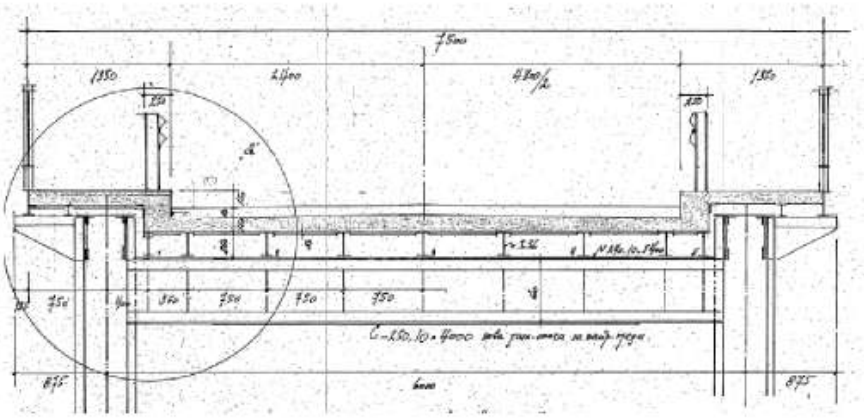


Fig. 10. Variant for the deck replacement with reinforced concrete deck [9]

3. A brief comment on the bridge installation technology (text by Nikolay Tuleshkov)

According prof. N. Tuleshkov,at the end of the 19th century, the first metal lattice bridges began to be built in Bulgaria, at first for the needs of railway construction, and later for road construction. The road bridge over the Yantra River in the medieval capital of Bulgaria, the city of Tarnovo, is considered. Its construction is necessary due to the specific hilly land relief cut by the deep gorge of the river, which here flows in the form of a large meander surrounding the city on three sides. Therefore, when the first main railway line for the region has to pass from here, the station has to be built in a more remote place, outside the city limits. And when in 1892, in the direction of "Marno Pole" and the hill "Sveta Gora", where the station is, the construction of a road connection between the city center and it began, it was necessary to cross the river on a bridge rising about 37m above the water level(Fig.11). And because of the steeply falling rocky banks, it could not rely on intermediate river beds to strengthen its bridging structure. This is how we arrive at the design choice of an arch truss stepping with its heels on both banks.



Fig. 11. General view of Stambolov's bridge, 1930

The project was made by the Italian architect Giovanni Mosutti, who at that time was the city architect of Tarnovo. Together with engineers from "Bulgarian State Railways", the author is reworking an existing project for a railway bridge, changing the construction of the roadway that steps onto the truss construction. The metal parts of the bridge were made

by the famous Viennese company "Rudolf Philipp Wagner", to which the Bulgarians most often place their orders for metal construction structures. Even in Tarnovo there is a preserved catalog of this company from the end of the 19th century, according to which constructions for supporting columns, beams, consoles, balconies, stairs and stair railings and other elements were ordered. For the construction of the bridge, the city administration concluded a contract with the famous construction entrepreneur Stoyan Tsonev Gerganov (1865 - 1906)(Fig.12), who was trained by the local guilds and came from a village of old hereditary builders - Kereka, Dryanovsko.



Fig. 12. Stoyan Tsonev Gerganov (1865 - 1906)

At that time, the talented Gerganov was already the most famous builder in the city, having built many public buildings and over 70 residential buildings here in 16 years. He independently mastered drawing design and therefore for each object he created according to his own ideas, he made the necessary plans. At the beginning of the work on the bridge, he appointed another famous craftsman from Dryanovo - Grozyu Enchev Tsonchev from Djurovtsi huts in the village of Parsha - as the responsible master of the construction.

Initially, the stone abutments were built on both banks, and then master Stoyan Gerganov designed a solid over 30m high wooden scaffolding from which the installation of the metal structure was carried out. However, the technology for raising its components creates a problem, which is solved based on ideas for lifting devices, which were proposed and implemented by master Grozyu Tsonchev, who supervised the entire construction.
Installation work is carried out by a large team of Belgian, Italian and Bulgarian craftsmen. On the wooden platforms of the scaffolding, hearths were created in which the rivets were heated before they were installed, and here the units forming parts of the structure were riveted.

The poet Kiril Hristov, born in 1875 in Tarnovo, recorded his memory of a large-scale tide on the Yantra River that carried away parts of the bridge that were carried downstream. "The waves of victory"- he writes, "drag the iron planks several meters long all the way to the bridge in the Turkish hamlet". This 1890s incident delayed construction. But unfortunately, and shortly before the last fragment of one supporting arch was installed, it was dislodged and fell into the river, from where they were unable to pull it up. It had to be ordered for a new construction in Vienna, and therefore the bridge was completed only in 1899-1900, when on October 8 the railway line and the Tarnovo station

"Trapezitsa" were solemnly opened. According to some reports, some additional works were carried out on the bridge, for example painting the metal parts, and therefore 1904 is indicated as the final year. In 1990, the company that produced the metal structure sent a notification to the city government of Tarnovo that the hundred-year warranty period of the bridge expired, but that they were available for restorations and repairs. Therefore, by order of 1992, it was forbidden for cars to pass through it, and it began to function as a pedestrian one(Fig.13).

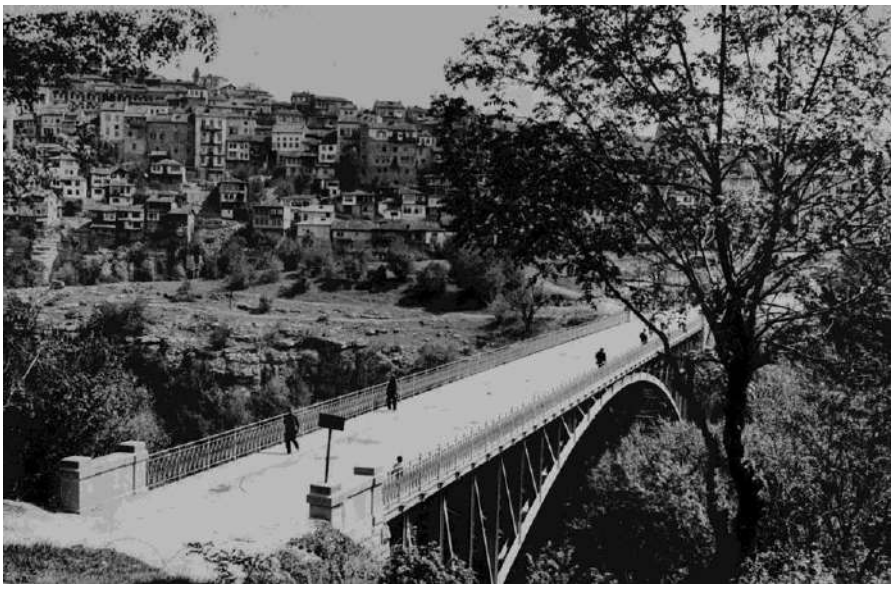


Fig. 13. General view of Stambolov's bridge, 1950

4. Conclusion

The Stambolovs' bridge over Yantra River is a part of Bulgarian structural heritage and also it is a symbol of Veliko Tarnovo town. It was designed and executed in 1892 and served as a road bridge. Between 1970 and 1998 the bridge served for the road traffic to the University of Veliko Tarnovo but from 1998 only pedestrian traffic is allowed. The last rehabilitation of the bridge including corrosion painting renewal was in 1935. It must be mentioned that there is no information for the detailed original structural documentation of the bridge in Veliko Tarnovo municipality and activities for the bridge rehabilitation, strengthening and preservation can be estimated as unsatisfactory. The project for reconstruction finished in 1996 is not realised up to now. As the last serious rehabilitation was in 1935 and considering the importance of the structure as pedestrian connection and also as a part of Bulgarian structural heritage the rehabilitation and strengthening of the bridge is urgent in order to stop the registered degradation processes and ensure the safety and comfort of the pedestrian traffic.

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