БЪЛГАРСКА АКАДЕМИЯ НА НАУКИТЕ И ИЗКУСТВАТА
Научен институт по Архитектура, Строителство и Градоустройство

BULGARIAN ACADEMY OF SCIENCE AND ARTS
Scientific Institute for Architecture, Construction and Urban Planning



София / Sofia
2017

Българска
Първо издание

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Компютърен дизайн:
Милена БОЖИДАРОВА

Предпечатна подготовка:
АИ „Арх & Арт“
1505 София, бул. „Ситняково“ 47А

Печат:
Direct Services Ltd

ISBN: 978-954-8931-52-6

БЪЛГАРСКА АКАДЕМИЯ НА НАУКИТЕ И ИЗКУСТВАТА
BULGARIAN ACADEMY OF SCIENCE AND ARTS

доклади

МЕЖДУНАРОДНА НАУЧНА КОНФЕРЕНЦИЯ ПО ОПАЗВАНЕ НА НЕДВИЖИМОТО КУЛТУРНО НАСЛЕДСТВО

БАНИ’2017
23-25 ноември 2017

proceedings

INTERNATIONAL SCIENTIFIC CONFERENCE “PRESERVATION OF CULTURAL HERITAGE”

BASA’ 2017
23-25 november 2017

София / sofia
2017

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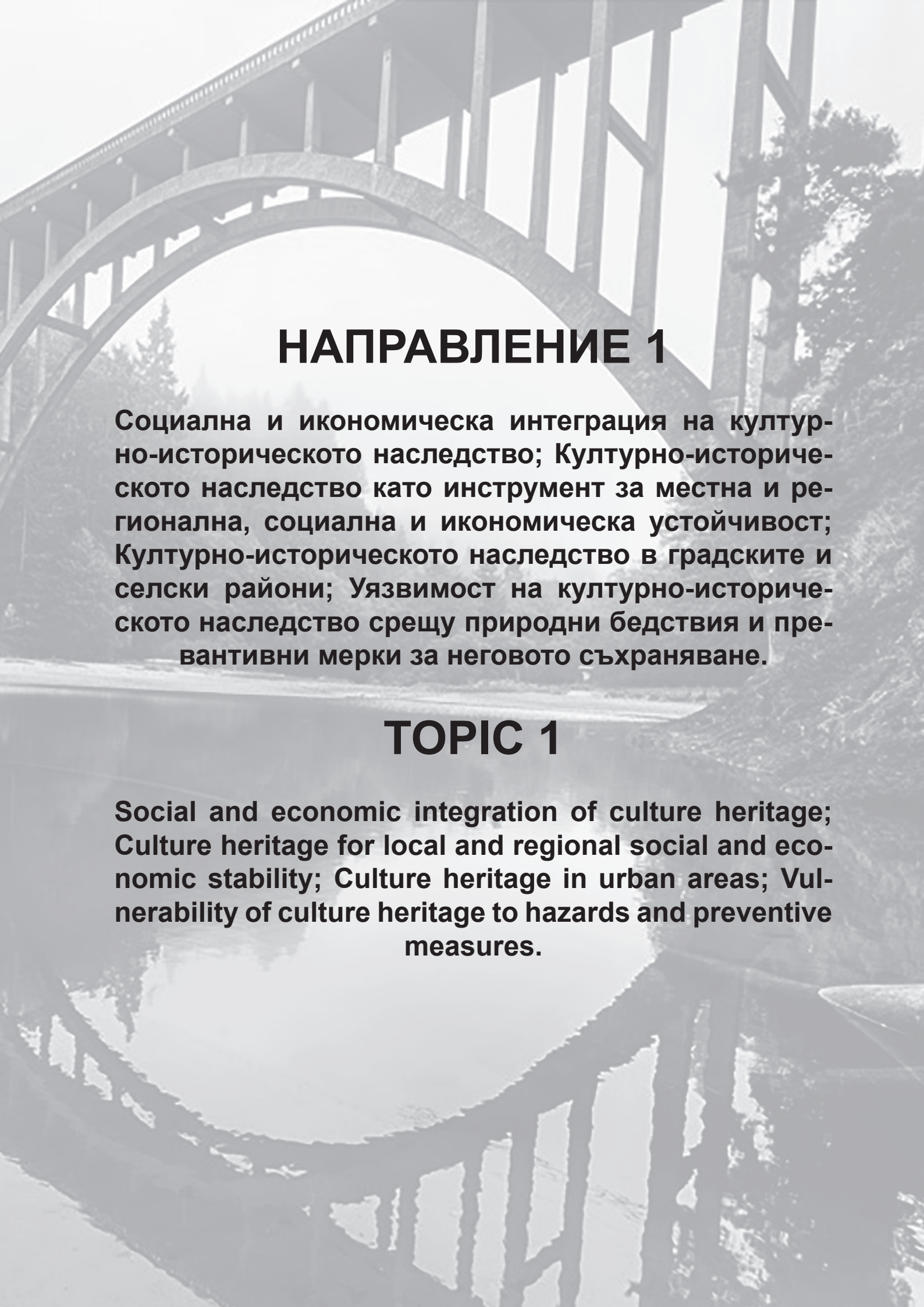
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НАПРАВЛЕНИЕ 1

Социална и икономическа интеграция на културно-историческото наследство; Културно-историческото наследство като инструмент за местна и регионална, социална и икономическа устойчивост; Културно-историческото наследство в градските и селски райони; Уязвимост на културно-историческото наследство срещу природни бедствия и превантивни мерки за неговото съхраняване.

TOPIC 1

Social and economic integration of culture heritage; Culture heritage for local and regional social and economic stability; Culture heritage in urban areas; Vulnerability of culture heritage to hazards and preventive measures.

DISAPPEARING PROBLEM OF GAZIANTEP LIVAS AND KASTELS AND DAMAGE ASSESSMENT

H. Merve Altan, E. Görün Arun

Hasan Kalyoncu University, Faculty of Fine Arts and Architecture

***Abstract**: The water needs for the residents of Gaziantep were met through carving rock tunnels to supply water. These tunnels called Livas were one of the historical underground water system that takes water from origin to required areas through carved water tunnels. This system provided water to pools and wells in houses, mosques, baths, fountains and Kastels. The 'Kastels' unique structures to Gaziantep, were social and cultural places for people built as a rock cut and stone masonry system in history. Nowadays, these historic places are unheeded and most of Livas system are broken. Out of the thirteen Kastels, only six of them reached today and the two of the six is about to disappear due to lack of care. The aim of this study is to introduce these water structures and to assess the damages caused by nature and human in extant Kastels. Making damage assessment of this system will be useful to contribute to their preservation.*

***Key words**: Gaziantep, Kastel, Livas, Rock-Cut Structure, Underground Water Structure, Historical Water Systems*

1. Introduction

Gaziantep, a city on Southeast Anatolia of Turkey, has no natural water source as river [1]. The settlement for the first time began to develop in Gaziantep Castle. After the earthquake that took place in 499 A.D. [2], settlements spread around the castle and the city centre was established in the neighborhood. Accordingly, the water requirement of the city increased with growing population. Due to geological and climatic structure of the region and lack of water in the city, it was necessary to build water management system. The water needs of the city was provided by transferring water from water source to city by carving water tunnels called "Livas" in history. Limestone, clayey limestone and chalk formation of Gaziantep [3] affected the development and character of the city and this formation made it possible to carve the underground tunnels and water way system. While transferring the water from the sources outside the city, in the places where livas system neared the surface, kastels, social places for the residents, were built.

Kastels and livas were usually made of rock carving techniques and have been used for centuries to meet the water needs of the region and the people. These rock-cut water structures were social and cultural places for people in history but today it is not well recognized by people and its value is not understood. Unfortunately, livas and kastels have been destroyed throughout history and suffered various damage due to lack of care. In order for these structures to be transferred to next generation in a healthy way and taking intervention decisions correctly first step should be to make damage assessment.

2. Gaziantep's Water Systems; Livas and Kastel

The fact that Gaziantep is located at the intersection of the Mediterranean and the Southeast Anatolia region and the first settlement is located on the roads from the south to the East and North to the Mediterranean. The city, which hosted many civilizations since prehistoric times, is on the historic Silk Road. The civilizations of the Chalcolithic, Palaeolithic, Neolithic and Bronze Age and Hittite, Assyrian, Med, Persian, Selefkos, Roman, Byzantine, Arab and Turkish have contributed to the formation of the city by leaving their own works. [4, 5, 6, 7, 8]. Doliche, the oldest settlement at 10 km distance from Ayıntab city, the former name of Gaziantep meaning beautiful spring in Persian (Figure 1), has been destroyed by the earthquake in 499 A.D., and thus the settlement started to develop at the Antep Castle [2, 9]. The city was under the rule of Hittites, Romans, Sassanids until the 10th century. Over time, the settlement spread out of the castle, around the castle and at the edge of the Alleben River, and the population increased rapidly.

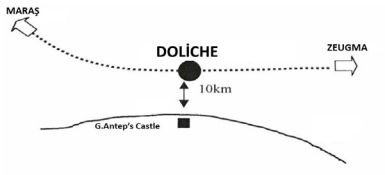


Fig. 1. Doliche (Uğur, 2004)

2.1. Livas

The water distribution system, called livas, is a slightly inclined underground water channel formed by carving a rock on the city so that it can be taken from the natural source of water to the required areas. The water was carried to baths, mosques and kastels by livas channels that are carved from rock. So the houses of the settlement developed near the livas channels to supply water needs [7]. In the system of livas, water to the houses and required places was provided by wells. The wells, carved deeper than the livas level were organised at a distance to the channel. Livas channel and well connection was provided by horizontal channels [9] (Figure 2a). In order to ventilate the water distribution system, chimneys were opened at certain intervals [10] (Figure 2b-2c-2d).

Although it is not exactly known how the city water was transported before, it is thought that the livas system was used first in Doliche. The water was transported by the livas system from Dutluk region at the south and the Pancarlı region at the west of the city. The Alleben Stream, transported water from Pancarlı region, from the upper branches of the Euphrates River, to the Sacur River [1, 11, 12]. At the same time livas system supplied water to the city of Aleppo by the Kuveyk line [10].



a) b) c) d)

Fig. **2**. a)Livas and well connection b) Livas c) The closed chimneys d)The apperance of chimney

In Livas, wastewater and clean water is separated from each other by channels. The wastewater channels are called kehriz. According to Şer'i Court Records, the wastewater collected in pools at certain points out of the city were cleansed and used for the irrigation of agricultural areas [13].

This water distribution network is similar to the system of qanat that was implemented by Persians in Iran for the first time 2000-3000 years ago [14]. The Qanat system is a slightly inclined underground rock carved channel that carries water from areas of higher altitude to agricultural land. To construct qanats, the masters called Muqannis drilled a main well that reached the underground water and carved horizontal tunnels to transport water and chimneys for ventilation [15].

This traditional water distribution system has spread over time to Arabia, Oman, Central and South Asia, North Africa and Spain (Figure 3). The Livas system of Gaziantep is called Quanat, Can ant, Connought, Kanat, Khanate, Khad, Kanayet, Ghannat, in Southwest Asia and Foggara, Faghare, Mayon, Iffeli and Negoula in North Africa. The kehriz system is called Karez, Kariz, Kahriz, Kahrez, Karaz, Kakoriz and Falaj in Southwest Asia and Khettara, Khottara, Rhettara in North Africa [15, 16].

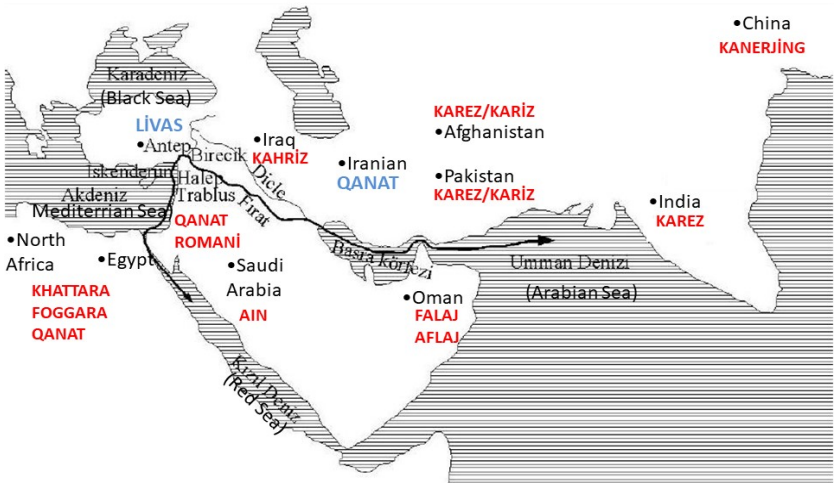


Fig. 3. Qanat's spread over the world (Uğur, 2004 was developed)

2.2. Kastel

In the places where livas system get close to the earth, the kastels were made by rock carving or stone masonry system. The word kastel is thought to have passed from the Arabic language to Turkish language [14]. The people of Gaziantep used kastels for hundreds of years especially in the hot summer times to gather, socialize and be included in the cultural life. These kastels, unique to Gaziantep included sections for people to take water, wash their dishes and laundry, bathrooms, toilets, ablution and resting places. The water continuously flowing in the kastels from the livas system was divided into two branches, one to the pools and the other to the toilets and the baths and then discharged to kehriz.

Out of 13 known kastels in Gaziantep 7 of them, namely Kastels of Şahveli Mosque, Ayşe Bacı Mosque, Kabainek, Kanar, Mehak, Sultan Gavri and Eşraf have disappeared and 6 of them namely Pişirici, Şeyh Fetullah, Kozluca, Ahmet Çelebi, Imam Gazali. and Ihsanbey have reached today [2,10,14,17]. Among the 6 that reached today, Ahmet Çelebi and Imam Gazali are about to disappear due to lack of care.

3. Kastels in Gaziantep

3.1. Pişirici Kastel

The construction year of Pişirici Kastel is not known exactly for there are no inscriptions. Masjid in the kastel is thought to be made during the Memluks period in 1283 for the inscription over the Mihrab is in Memluks style [14]. Kastel was carved from rock, dressed stone was used in its arched openings and ceiling. Besides the Masjid, there are three pools, eight toilets and two bathrooms (Figure 5a-5b).



a) b)

Fig. 5. a) Pisirici Kastel and Masjid plan [17] b) Pisirici Kastel and Masjid

3.2. Ihsanbey Kastel

Ihsanbey Kastel also contains a Masjid. It is known that this kastel was in use during 15th century. It is completely carved 5.15 m. down under the ground. To level the rock at the floor, some parts of walls and ceiling, dressed stone is used. There is a pool with a seating area around (Figure 6a-6b). Previously, in the north of kastel, there were 4 toilets and a bath but these were removed during the restoration works in 1970s and ventilation hole over the sitting room was closed during the road construction in 1980s [2, 10].

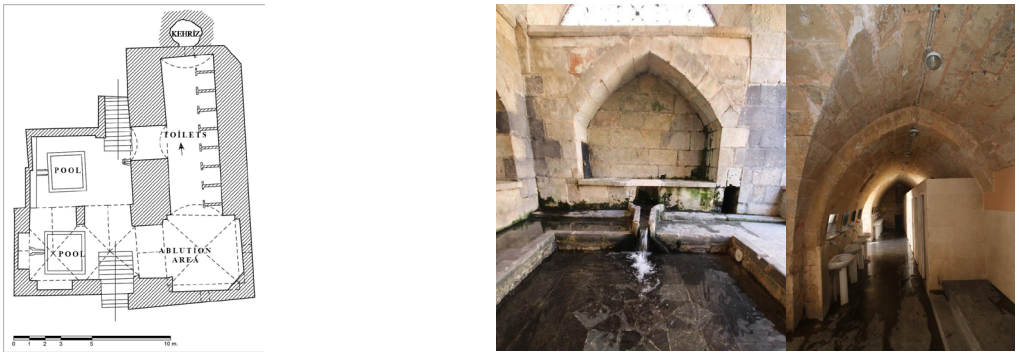


a) b)

Fig. 6. a) Ihsanbey Kastel and Masjid plan [17] b) Ihsanbey Kastel and Masjid

3.3. Şeyh Fethullah Kastel

Şeyh Fethullah Kastel was in use during 16th century. Because it is close to the surface, some parts are under the ground. The walls and ceiling are of dressed limestone and the floor is of dressed basalt stone. The Kastel is reached by two stairs descending from north and south. The two pools on the west side of the structure get water from the west channels (Figure 7a-7b). Nowadays the toilets at the east side have lost their originality and some have been demolished and replaced with ablution place [2, 10, 17].



a) b)

Fig. **7**. a) Şeyh Fethullah Kastel plan [17] b) Şeyh Fethullah Kastel

3.4. Kozluca Kastel

There is no inscription about the construction year of the Kozluca Kastel and the Masjid inside. According to the Şer-i Court Registers and Ayintab Foundation Book this place was used during 1500s. The location of the masjid section, referred in the sources, is not known for it is not present today. The Kastel carved in rock is covered of dressed stone during restoration. It consists of two sections covered by cross vaults to the north and south. There is a pool on the south side of kastel and water is provided by the channels coming from the south. However, these channels do not function today (Figure 8a-8b).

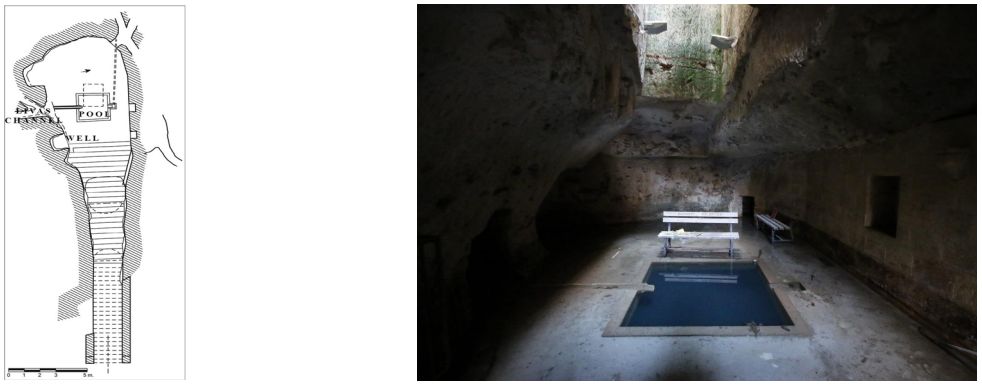


a) b)

Fig. **8**. a) Kozluca Kastel plan [10] b) Kozluca Kastel

3.5. Ahmet Çelebi Kastel

According to some sourses, Ahmet Çelebi Kastel was used during 16th century. But its construction year is not known exactly. Kastel is carved from the rock and is completely under the ground. It is reached by stairs from south. There is one pool inside. The upper part of the stairs is made of dressed stone and plastered. In the part where the pool is located, only the north wall is dressed stone, the ceiling and the remaining parts are gemstone. In this Kastel, five livas intersect. There is one well to control underground water rising and a chimney for ventilation (Figure 9a-9b).



a) b)

Fig. **9**. a) Ahmet Çelebi Kastel plan [17] b) Ahmet Çelebi Kastel

3.6 İmam-ı Gazali Kastel

Imam-i Gazali Kastel's construction date is also not known. It was in use during 16th century. Kastel was made with rock carving technique under the ground. The livas channels and a pool is on the east. To get light, there is a chimney over the pool. The pool place is reached by 46 steps. At the 20th step, there is a masjid on the north (Figure 10a-10b). Nowadays, the livas channels no longer function. There is no detailed information about this kastel.



a) b)

Fig. **10**. a) Imam-ı Gazali Kastel plan [17] b) Imam-i Gazali Kastel

4. Damage Assessments of Kastels which Reached Today

It is important to know the factors that caused damage and deterioration on the rocks of the Kastels It is seen that the interventions made on the Kastels, damages were not properly examined.

The investigations have shown that the livas carrying water to Kastels are damaged and destroyed due to the use of new hydraulic and city water distribution systems and construction works. Because of this, Kozluca, Ihsan Bey, Imam-ı Gazali and Ahmet Çelebi Kastels cannot be ventilated and water cannot be discharged from these Kastels. The water in Şeyh Fethullah and Pişirici Kastel cannot be used because the water coming from livas, is mixed with sewage water due to cracks and damage in the channels and contain bacteria and microorganisms. Today the restoration of the Pişirici Kastel is finished and can be visited. The restoration work in Kozluca and Ihsanbey Kastel is continuing. Imam-ı Gazali and Ahmet Çelebi Kastels are not used and neglected. Some spaces of the Kastels have lost their authenticity due to wrong interventions and improper additions. Seyh Fethullah Kastel's original restrooms and bathrooms were demolished and replaced by squat toilets,

handwashing and ablution places that do not match the texture of the building (Figure 11a). The rock carved original steps of the Imam-ı Gazali Kastel were changed by making concrete additions to the stairs (Figure 11b). Because the livas channels are damaged in these kastels, the water rises in rainy weather and damages are caused by the atmospheric effects.



a) b)

Fig. **11**. a) Seyh Fethullah Kastel incompatible additions b) Imam-ı Gazali Kastel

Pisirici Kastel and Ihsan Bey Kastel have wrong repair damages. The surface of the rock covered with plaster hold the water flowing from the cracks and avoid breathing of the rock. This caused damages due to moisture effect on the surface of the plaster (Figure 12a-12b).



a) b)

Fig. **12**. a) Pisirici Kastel repair damages b) Ihsanbey Kastel repair damages

In general, cracks and fragmentations are observed in all kastels. Imam-ı Gazali, Ahmet Çelebi and Pisirici Kastels have more damages (Figure 13a-13b-13c).



a) b) c)

Fig. **13**. a) Imam-ı Gazali Kastel cracks and fragmentations b) Ahmet Çelebi Kastel cracks and fragmentations c) Pisirici Kastel cracks and fragmentations

Imam-ı Gazali, Ahmet Çelebi, Şeyh Fethullah and the Pişirici Kastels have biological damages. Imam-ı Gazali Kastel has lichen, black moss and plant species in the entrance door and in the chimney section over the pool (Figure 14). Ahmet Çelebi Kastel's have lichen, black moss and plant species at the stair edges and various fungal damages are seen on the chimney (Figure 15).



Fig. 14. Imam-ı Gazali Kastel lichens, black moss and plant species biological damages



Fig. 15. Imam-ı Gazali Kastel lichens, black moss and plant species biological damages

On the surface of rock carved mihrab of Pişirici Kastel's masjid section there are spalling and crusting. The pier on this section has voids and vandalism damages (Figure 16a-16b-16c). Foliation and pitting is seen on the ceilings of bathrooms and toilets (Figure 17a-17b). Besides the similar damages, there are crust formation, abrasion, surface loss and discolouration on the ceiling and the walls of Ahmet Celebi and Imam-ı Gazali Kastels (Figure 18).



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

Fig. **16**. a) Pisirici Kastel pitting b) Pisirici Kastel vandalism c) Ahmet Celebi Kastel



a) b)

Fig. **17**. a) Pisirici Kastel spalling and crusting b) Pisirici Kastel foliation



Fig. 18. Imam-ı Gazali Kastel spalling and crusting, surface loss

5. Conclusion

Since the prehistoric eras, Kastels, rock carved underground architectural spaces, has been used for many years at Gaziantep. There are 13 kastels mentioned in the sources but seven of them have disappeared and only six of them, namely Pişirici Kastel, Ihsanbey Kastel, Kozluca Kastel, Imam-ı Gazali Kastel and Masjid, Şeyh Fethullah Kastel and Ahmet Çelebi Kastel have reached today. In this study, these kastels were introduced and types of damages and deterioration were examined. Despite the lack of awareness, maintenance measures taken over time were insufficient and the restorations that have not considered the rock material behaviour were improper works. So, to meet fragmentation, crack, spalling, exfoliation, crust, discolouration, surface loss, abrasion, vandalism, inappropriate additions, biological and repair damages are common in these kastels.

According to the results obtained, rock properties should be known and damages on the rock should be determined in order to make correct interventions to preserve the kastels. When giving new functions to Kastel or keeping the old function protected, the safety of the users should be ensured. In order to hand down these Kastels and Livas system to future generations, damage assessment has to be done correctly. The opinions of experts should be taken in to consideration to reduce the loss of integrity and authenticity of heritage structures.

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ARCHAEOLOGICAL STUDIES AS A MEANS OF PRESERVING THE ARCHITECTURAL HERITAGE IN BULGARIA

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***Abstract**: What is the content of the term cultural heritage? The sum of preserved evidence for the existence of intelligent human beings left a mark in the ground for their beliefs, an understanding of the surrounding environment and material values. These values and concepts are the basis of archaeology, their search and documentation is the basic method of archaeology for the preservation of the cultural heritage. So far, architecture, lost in the past in dozens of cubic meters of soil and ruins of old constructions, and the trapped between thousands of objects and coins of their inhabitants, enables us to learn and preserve the culture that they created.*

However, what is missing: examples of a disproportionate increase in attendance after the completed stages of conservation and restoration; the content, as a goal in the research of archaeology and the main argument for the demand for sustainability in the expected added value of the tourist product; municipal and institutional practices, advertising and management.

***Key words**: Cultural heritage, ancient and medieval architecture, preservation Lyutitsa fortress, "Saint Petka" fortress, Urvich fortress*

1. Introduction

This report will present the work on some archaeological sites on the territory of Bulgaria in the last decade. Among them are Lyutitsa next to Ivaylovgrad, Kokalyane Urvich next to Sofia, and Peristera in the town of Peshtera. In the light of archaeological discoveries, on the surface emerges an ancient and medieval architecture, which, like the iron objects, needs conservation and restoration. The maintenance and systematic work of the archaeological teams has a key impact on the development of the sites, not only because of the strong tourist interest in the current discovery. Through systematic cleaning of strong forest vegetation, by wrapping or restoration, treatment with herbicides in the building or renovating joints, these are just some activities of systematic care of old age. This creates a welcoming vision of the monuments and increases the tourist demand.

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2. Presentation of the sites – location and history of research

2.1. Lyutitsa fortress

Lyutitsa fortress is located 6 km east of Plevun village, and at the same distance west of the Ivaylovgrad neighbourhood "Ladza", above the village of Sviraci, on a 590-meter-high altitude plateau, over the surrounding terrain. It is splendidly preserved, but by the year 2002 among the inhabitants of the surrounding settlements the area is known as Gradishteto or Fortress near the village of Rogozovo. Its designation as the famous town of Lyutitsa is an idea of Prof. Bozhidar Dimitrov and the archaeological researches conducted under the supervision of Assoc. Prof. Bonnie Petrunova not only confirmed the veracity of this hypothesis but added new curious facts to the Medieval History of Bulgaria (Petrunova 2004[1]), [1].



Fig.1. Octagonal tower at the South-eastern end of Lyutitsa (photo Petrunov)

After the initial clearing it was discovered that Lyutitsa is one of the largest fortresses in the Eastern Rhodopes. The length of the cave is over 600 m there are twelve towers, one of which is eight-sided, two rounded and eight rectangular. The walls are in very good condition and in some places their height is preserved up to 10 m. They outline an irregular ellipse according to the configuration of the terrain [2]. The entrance is only one, flanked by two rectangular towers with its long side perpendicular to the wall. The tower and the towers are built with gems processed on the face, welded with mortar. Inside, the building is reinforced with "concrete" of

[1] Петрунова., Б., Средновековната крепост при с.Рогозино, Ивайловградско. Етност и менталност, София, 2004

stones and mortar or the so-called *opus implectum.* Ninety percent of the plates that form the faces of the walls are marble, which is why, for the media this magnificent monument became known as the "Marble City".

During the excavations, until now, were discovered two churches, and later from the XVI century, a rectangular naos, a shallow apse, and traces of frescoes found at the North-eastern corner of the building. The roof was sloped, tiled, crowned with a wrought iron cross that was found among the rushes in the centre of the rectangular room. Church No. 2 is a three-nave, built in the IX -X century. It was richly decorated with frescoes and plastically shaped marble panels. In the South-eastern part of the fortress, the rock has a cut-out (well) with a diameter of 4.60 m. Until now it has been cleared at 5 m depth, the finds found there give reason to assume that the facility was incised in the rocks, more during the late Bronze Age, and in the Middle Ages it was used consecutively as a courtyard, dwelling and a dump pit [3].

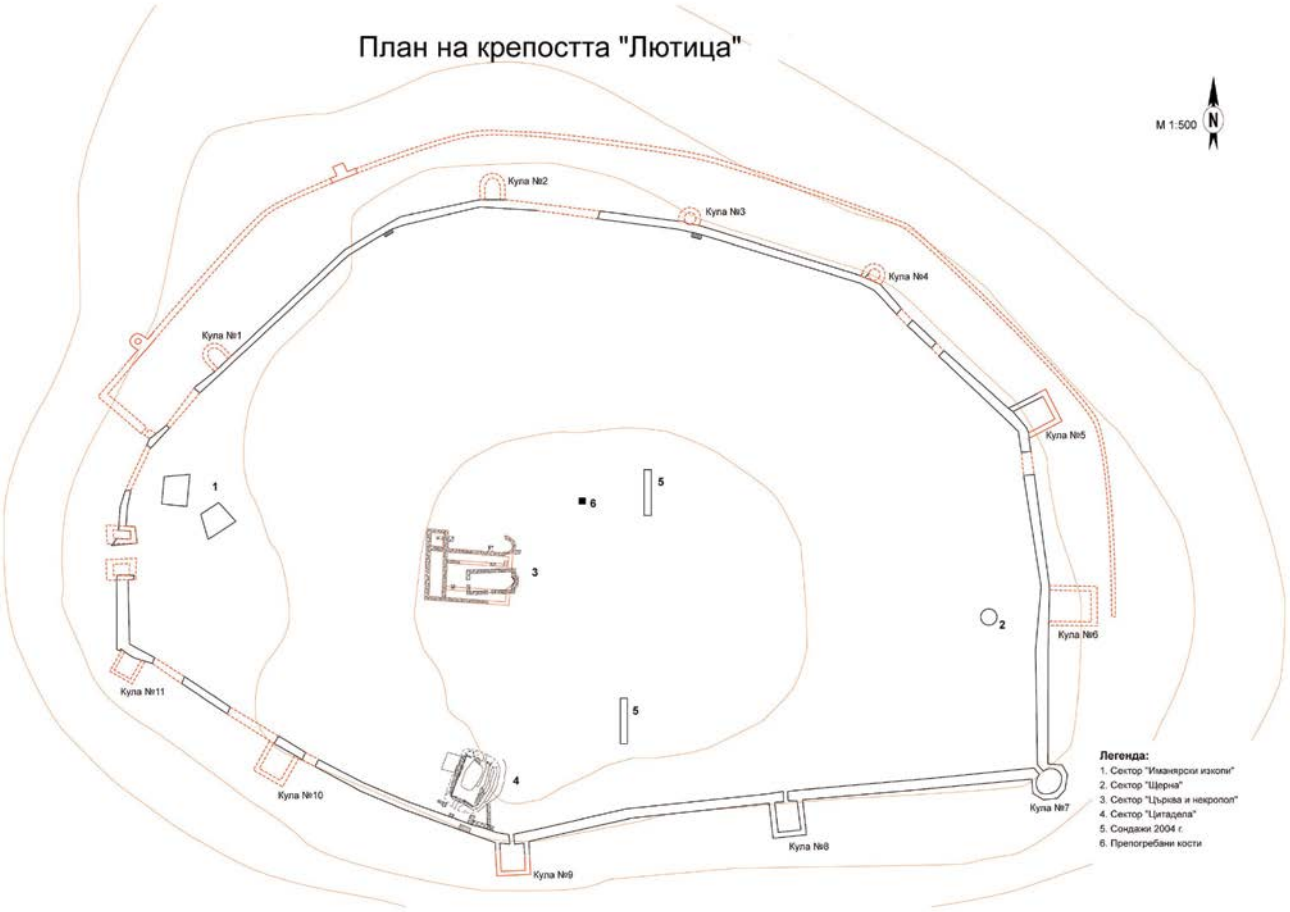


Fig.2. Plan of Lyutitsa fortress (made by



Fig.3. Water reservoir (photo Petrunov)

Archaeological studies have revealed a huge amount of ceramics, some of which are elitist, with embroidered decorations as a golden engobe, and medieval sgraffito. The ceramics, as well as the finds: coins, jewels, metal and bone objects of the household, weapons, objects of the cult, architectural details, etc., show that the hill was inhabited since the 1st millennium BC until 17th century AD.

In 2011 the Kopam BG Association for Archaeological Research and Environmental Protection of Monuments implemented a project for the socialization of the Lyutitsa fortress, funded by United Bulgarian Bank1.

As a result, the following activities have been carried out, making the site a pleasant tourist destination:

- Planned cleaning of the studied sectors from vegetation.
- Forming of paths to the studied sectors.
- Renewal and updating of signposts on the road to and within the fortress.
- Creating recreation areas - benches, sheds toilets.
- Issuing a tourist brochure.
- Partial rehabilitation of sections of the road to the fortress.

All this was accomplished with little means, much professionalism and desire.



Fig.4. Photo of the renovated site, view of the remnants of the medieval churches (photo Petrunov)

2.2. "Saint Petka" fortress

The other fortress called "St. Petka" on the hill where it is located (or Peristera - according to the folk tradition) is within the boundaries of the town of Peshtera. The presence of remains from it is reported by St. Zahariev and K. Irechek respectively in 1845-1850 and 1883. The first archaeological excavations were carried out in 1954, but there is no field documentation for them. Under the leadership of Tsvetana Gizdova excavations were made in 1958 and 1972. In 2007, the Municipality of Peshtera launched an archaeological research in the South-eastern part of the fort, on and around the so-called St. Petka Church. During the years 2007 - 2013, the head of the archaeological research was Assoc. Prof. Dr. Boni Petrunova (NAIM-BAS). Active participation in the campaign was took Elena Vassileva (NAIM-BAS), Dimitar Pavlov (IM Peshtera), Technical Director Filip Petrutov (NIM), Violina Kiriakova, Gabriela Raykova and others. Architect of the study is July Farkov (Petrunova 2007[2]). As a result of the archaeological research it was found that the fortress walls have a length of 253 m and surround an area of 2,895 sq. m. The fortified area we call "citadel" has the shape of an ellipse, with the long axis orientation - SI-SW, length of 94 m and the width of the fortress - 37 m. The displacement of SW from SW is 5 m and the highest point of the fort is 494.6 m, [5].

[2] Петрунова., Б., Крепостта Св.Петка в гр. Пещера., Археологически открития и разкопки през 2007



Fig. 5 Photo made with drone, in the study in 2010 (photo Petrunov)

The fortress's structure of the Citadel was gradually elucidated, alongside the tower, two tower churches, one triangular and three rectangular security facilities were registered. It was found the presence of a second fortification belt, situated on the slope of the hill, and probably spans a space that could be called "Inner City". At the base of the hill is also a third fortification belt, whose role is to secure the suburb and its neighbourhoods. Throughout the years of archaeological research, over 400 objects have been found, with only 266 finds in 2013, a huge mass of material (only iron objects over 50 kilograms) and more than 2,000 fragments of ceramics. Collective coin finds are few, two of which consist of golden solids. As a result of the surveys in 2007, 2008 and 2010 years, in 2012 was won a project1 "Conservation, Restoration and Exhibition of Ancient and Medieval Fortress" Peristera "located on the hill" Saint Petka", Peshtera town with beneficiary Peshtera Municipality.

During the implementation of the project[3], especially during the period from April 4 to November 17, 2013, additional archaeological researches and clearings were carried out, as a result of which new data on the interior of the fortress (the citadel) and the slopes of the fortress the hill where the two additional security spaces are located. 28 "spaces" were discovered and cleared, chained along the inner faces of the eastern and western fortress walls. The newly opened premises are formed of walls made of stone in mud and adhered to a joint, perpendicular to the wall. They are of different sizes and probably of a different purpose: living quarters for soldiers, business

[3] The project is implemented under The Operational Program "Regional Development" 2007-2013, Priority Axis 3: "Sustainable Development of Tourism", Operation 3.1: "Improvement of Tourist Attractions and Related Infrastructure", Grant Scheme : BG161RO001 / 3.1-03 / 2010 "Support for the Development of Natural, Cultural and Historical Attractions", Budget line: BG161RO001 / 3.1-03 / 2010. – St

premises, workshops, warehouses, free spaces between the premises and others. All the exposed walls are of mud stone, above the stone (i.e., above 1.50 m above the base) were built with sharp, which is partially stored in some places. They were covered with beam work and tiles above. Although the buildings were destroyed as a result of a ubiquitous fire, some were found floor levels that were recorded at an average depth of 490.00 m. During the realization of the project, on the recommendation of the archaeological team, these rooms were covered with clean earth at approximately the original under. In the towers, the premises and beyond, there are registered, cleared and documented 87 dolomites (naturists), including the ones discovered in previous archaeological seasons. Their content has been studied by harvesting samples for testing. All open spaces were preserved, some were restored, and 47 of them were exposed in the study rooms, [6].



Fig. 6 Conservation Status of pithos, (photo Petrunov)

Field Restoration and Conservation of pithos was carried out by and under the direction of Maria Taseva, a licensed restorer from the Conservation and Restoration Laboratory at NAIM-BAN. According to its data, the conservation and restoration activities include the following: Cleaning of the pots from surface contamination (biological agents and salt deposits), which was done by treatment with aqueous sodium hydride (SCHEME III). The adhesion of the small potato fragments is carried out with PARALOID B 72 dissolved in acetone, the binding sites of the individual fragments being reinforced with a master mix of MD. The repair of missing elements and grooving of the joints is done with a master mix of MD with brick filler. The gluing of the fragments of the big pithos was done with epoxy resin with brick filler. The stitches were additionally reinforced with a blend of MD, and brick filler, and the kits were made of the same material.

A protective coating of PARALOID B 72, dissolved in xylene, has been applied to all pithos. Some of the pithos have been restored and preserved with the chemicals described above without being moved from the original places where they were found. A site conservation of brick surplice was also carried out in one of the rooms which was treated with PARALOID B 72 and covered with a hydrophobic mixture, the wall being exposed to in situ. Separate metal parts, which are part of the cast and the fortress facilities, are also processed. Conservation and partial restoration of smaller vessels were carried out, which were later used in expositions dedicated to Peristera, [7].



(**Fig.7.** Photo of Exhibit, photo Petrunov)

In the researches, samples of plant remains were collected from the pithos, and in only one pithos, in room 24, the packages were 8. The samples were examined by Assoc. Prof. Tsvetana Popova, who established 8 plant species, of which 4 cultivated plants - beans, rye, millet and soft / compacted wheat, and charcoal - oak, beech, white pine as well as walnut shells. I will quote one of the findings of Prof. Popova: ".. (a) The presence of charred wood in pithos can be assumed to have been invaded by the structures of the premises or to have been covered with wooden shutters."

We certainly know that the dolomites were covered with stone shutters, so we accept the above view as another confirmation that in some rooms there was a second floor separate from the first with a wooden structure.

On May 22, 2013, the restored antique fortress of Peristera was opened, the purpose of the archaeological team being to maintain the authentic appearance of the cedar as much as possible and not to rebuild the uncovered buildings such as the northeast tower church, [8].



Fig.8. The restored Peristera (photo Petrunov)

2.3. Urvich fortress

The last fortress I represent here is located 2 km from Sofia, on the way to Samokov. The Urvich fortress is protected from steep cliffs on all sides, and the river bed of the Iskar River is bordered by it. It is known that even during the Roman epoch, there was a road called Trayanov, named after Emperor Trajan (98-117), and in the 6th century the road connects Vinoboda- Singidunum- Nijos- Serdika- Trimontium (Philippopolis) Adrianopolis- Byzantium (Vienna- Belgrade- Nis- Sofia- Plovdiv- Edirne- Istanbul). Some researchers suggest that before the construction of the medieval fortress, the Roman and Early Byzantine fortress Eskumanne was built on the hill, which means the fortress of Eskus (Iskar) (Petrunova 2009[4]).

The medieval fortress Urvich was first mentioned in the history of Paìsiy Hilendàrski. There he writes: "Tarnovo's masters and King Shishman could not oppose the Turks. And they escaped from Tarnovo, from Zagore and Stara Planina, who came to the town of Sredets or Sofia, and here they stopped in some narrow places on the Iskar River and the Vitosha Mountain. So they received help from the Serbs, from King Vukashin and from the Ohrid Bulgarians. For seven years King Shishman stood in Sredets and past Iskar. He had a monastery called Urvich with a strong fortress,

[4] Петрунова Б. - Урвич-последната крепост на Шишмановци.- 1200 години Сердика-Средец-София в България 809 г./1300 години София-столица на България/140 години Българска академия на науките, София 2009, 102-117, ISBN 978-954-516-895-6.

surrounded by water ... When the Turks took from the king Vulkushina the Ohrid land and enslaved it and overcame it, they finally defeated and destroyed King Shishman and the army, the Tarnovo and the Bulgarian nobles.

In 1969, under the leadership of Prof. Dimitar Ovcharov, part of the fortress wall and a battle tower were studied. After a major interruption, the excavations were renewed in 1979. The fortress church is located in the centre of the area surrounded by the fortress walls. Prof. Ovcharov first finds that they were erected on the very edge of the steep slopes of the ridge and that they surrounded a space with "oblong, trapezoid form" (Ovcharov 1980[5], Ovcharov 1981[6], Ovcharov 1981a[7]). In 2008, a joint project of NAIM-BAN and the Culture Program "Urvich - the last stronghold of the Asenevtsi" was developed.

The main purpose of the program was to link the study and ennoblement of the Urvich fortress to the 1200 anniversary of the accession of Serdika to Bulgaria by Krum in 709. According to the funding, the following main activities were carried out: Cleaning the fortress of grasses and shrubs, shaping paths and places for rest, providing access to the walls, the towers and the more attractive remains [9]. At the beginning of the project (November 2008), we found the monument without visible traces of care over the past 30 years. The first activity was to restore the paths that provide access to the fortress walls, to the towers, to the church "St. Ilia ". The work was distributed in the mentioned sectors, and they were completely cleaned of vegetation. Signs were placed, [10].



Fig. 9. Photo of medieval tower (photo Petrunov)

[5] Овчаров Д. - Урвич-легенди и действителност., София-древна и млада, 1980
[6] Овчаров Д. - Крепостта Урвич край с. Кокаляне.-МПК, 1981
[7] Овчаров Д. - Църквата в крепостта Урвич при с. Кокаляне, Софийски окръг.-Археология, 4, 1981а

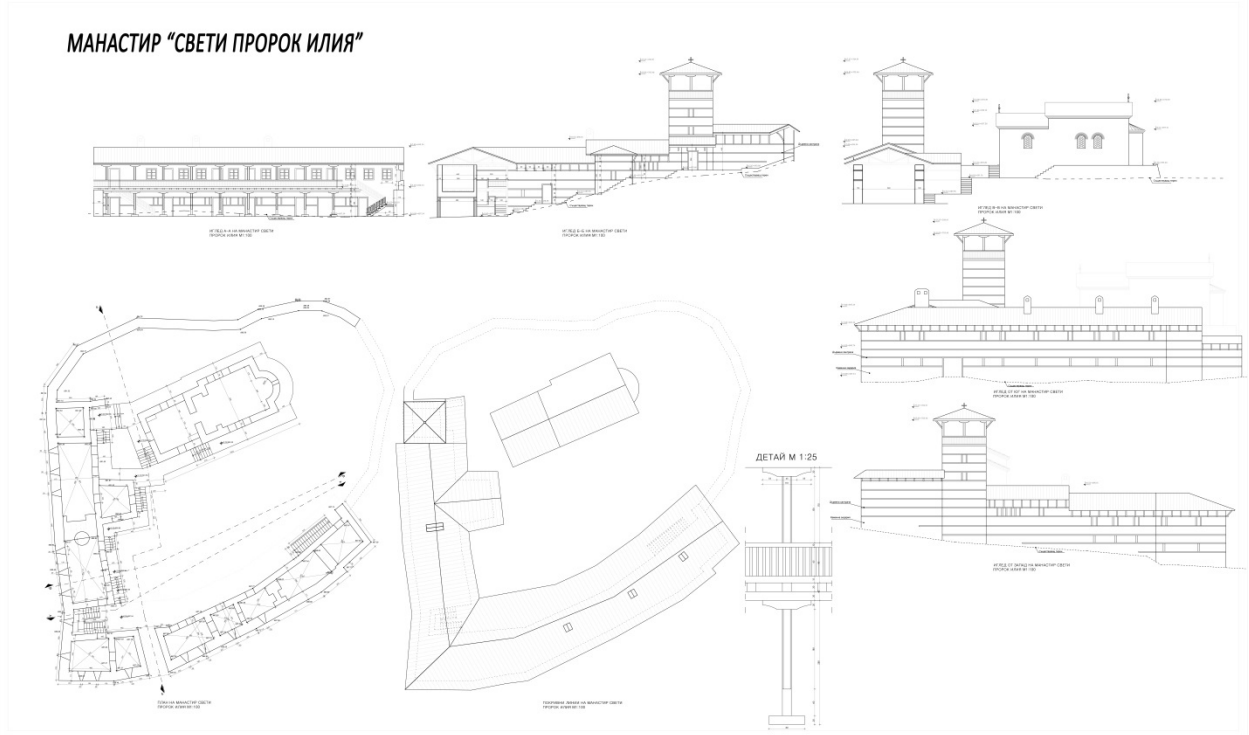


Fig 10 Plan of the monastery ‘’St. Proroklia’’, (July Farkov`s 2008 Project)

3. Conclusion

In addition, apart from the archaeological research, processing of archaeological data and materials from an archaeological site, the conservation and preservation methods occupy a central role in preserving them. Archaeological sites and artifacts are not just objects and ruins, shown to visitors without care for their protection and conservation. The maintenance (such as cleaning of strong vegetation, maintenance of facilities, etc.), and the systematic work of the archaeological teams have a basic impact on the development of the sites as part of the cultural heritage.This creates a welcoming vision of monuments and increases the demand from tourists.

Acknowledgement

Sincere thanks to the team of archaeologists and collaborators, took part in the study and ennoblement of the Lutitsa fortress: Filip Petrutov, Elena Vassileva, Boryana Radeva, Rosen Peevski, Radoslav Georgievski, Atanas Georgiev, and many others.

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PRORA, A CASE STUDY OF AN HERITAGE BUILDING OF CAST REINFORCED CONCRETE

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Abstract: *Prora is a beach resort on the island of Rügen, in the German part of the Baltic Sea. It was planned as a huge holiday resort for growing middle class, and partly built between 1936 and 1939. It is situated 160 metres from the beach and follows the cost line for 4.5 km. The design won the Grand Prix award at the Paris World Exposition in 1937.*
An on-site cast reinforced concrete skeleton construction is applied in every of eight housing blocks. Each of them is 500 m long and six storey high, and it was designed to accommodate 20,000 guests at once. The complex was listed as heritage in 1992, as rare example of architecture from the period of the Second World War which was created for entertainment and recreation, and not either for industrial or military purpose. The construction was considered innovative for the time when it emerged.
Five of the housing blocks still exist. The last attempt for revitalisation, among many, came in 2012.
In this paper, we researched this early application of on-site cast reinforced concrete for daring structure of colossal dimensions, and we analysed it in the context of historical development of concrete application techniques.

Key words: *heritage, on-site cast reinforced concrete, skeleton construction, beach resort, the Second World War*

1. Introduction

Prora is a beach resort on the island of Rügen, in the German part of the Baltic Sea. After smashing the German Unions in 1933, it was planned as a huge holiday resort for growing middle class of industrial workers, and partly built between 1936 and 1939. It is situated 160 metres from the beach and follows the cost line. The radius of the curved building is 13 km.

Prora was used for many purposes over the years. At the end of the Second World War the complex was used for refugees from Hamburg and later for the Army of East Germany. Recently, the complex has been transformed into a 5-star hotel and luxury apartments.

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Fig. 1. View Prora complex [1] according to [3]

The structure itself extends impressive 4.5 kilometres (Fig.1), which explains why the complex is also known as Colossus of Prora. At the time of the construction, the construction technique was considered innovative. An on-site cast reinforced concrete skeleton construction was applied, in every of eight housing blocks. Each of them was 500m long and six story high, and it was designed to accommodate 20,000 guests at once. Each room was directed to the East, overlooking the Baltic Sea. Therefore, the depth of the housing blocks was 10.5 meters on the ground floor and at the first floor, and only 8 m above that. The ground floor (ceiling height 2.85 m) was planned to be used for children's premises, shops and apartments of employees. The other floors had a ceiling height of ca. 2.50 m. In addition, there have been built so called "Community Housing". That is a 3.5 story high and 110 m long tower, with main axe perpendicular to the frontline of the housing blocks (positioned ahead of them, towards the beach). In its ground level, there were utility rooms and dining rooms, while on the second floor, there were cafeterias and restaurants with terraces; the mezzanine was reserved for writing, reading and children playrooms, and the upper floor, for bowling and billiard. The building had elevators, every 50 m, on the side of the back yard front.

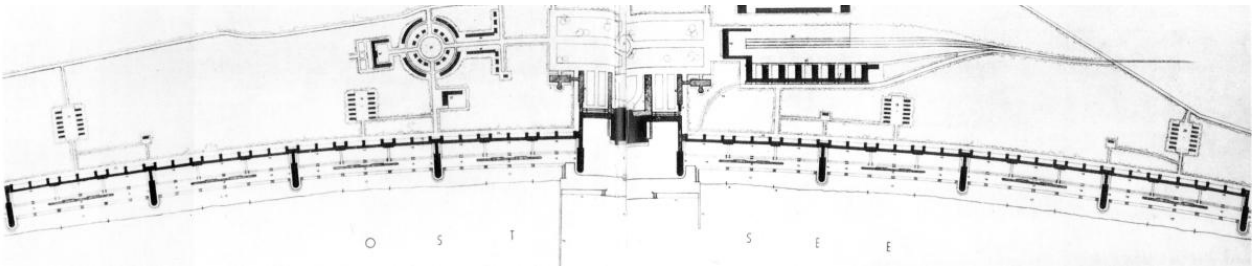


Fig. 2. Original ground plan [2] according to [3]



Fig. 3. Planned design on the left [4] and typical built single room on the right (photo: Th. Schramm [6]), according to [3]

Each guest room was of 4.75 x 2.25 metres in size, had two beds, a wardrobe, a seating corner and a sink, all designed according to the modular principle. Figure 4 shows the outlook of typical room. The guest blocks included communal toilets, showers and bathrooms on each floor.

The housing blocks (Fig. 2) were designed by Cologne architect Clemens Klotz (1886–1969), whole the main hall for festivities (Fig. 3, in the upper right corner) was designed by Erich zu Putlitz (1892-1945), an architect from Hamburg. The design attracted lot of attention due to its size and daring concept; therefore, it won the Grand Prix award at the World Exposition in Paris, in 1937. Nowadays it is mostly preserved for its colossal dimensions and atypical purpose it was originally designed. Using cast concrete for such huge structure in 1930s is recognized, in this paper, as pioneering research of construction techniques limits, and therefore discussed in that context.

2. Original building design and construction

The housing blocks were designed according to 5 m x 5 m grid. The load-bearing exterior walls have been made of masonry, while the window lintels have been made of reinforced concrete. The cast concrete technology was already widely known and accepted at that time. Reinforced strip and individual foundations (C20/25) have been carried out. The floor of the housing blocks consist of reinforced concrete slab (C20/25) with steel bars (d= 6 mm and a= 5 cm), which carries the load in two directions.

Starting from 1935, high-strength concrete steels were developed for the purpose of material savings by twisting round steels ("Torstahl"). This innovation has been applied in Prora. The bricks in the "Reichsformat" have the dimensions 25 x 12 x 6.5 cm.

All 9 major construction groups and 48 construction companies of the Third Reich with nearly 9.000 workers were involved in this project (Fig. 3, on the right), starting from November 1936. In October 1938 the structural work of the first 500m long building was ready. Some of the companies which were engaged, e.g. Dyckerhoff & Widmann, Hochtief, Polensky & Zöllner, exist to this day. Then, the building site was temporarily closed due to the World War II. The swimming pools, a cinema and a theatre, although included in original design, were not built at the time (Fig. 2).

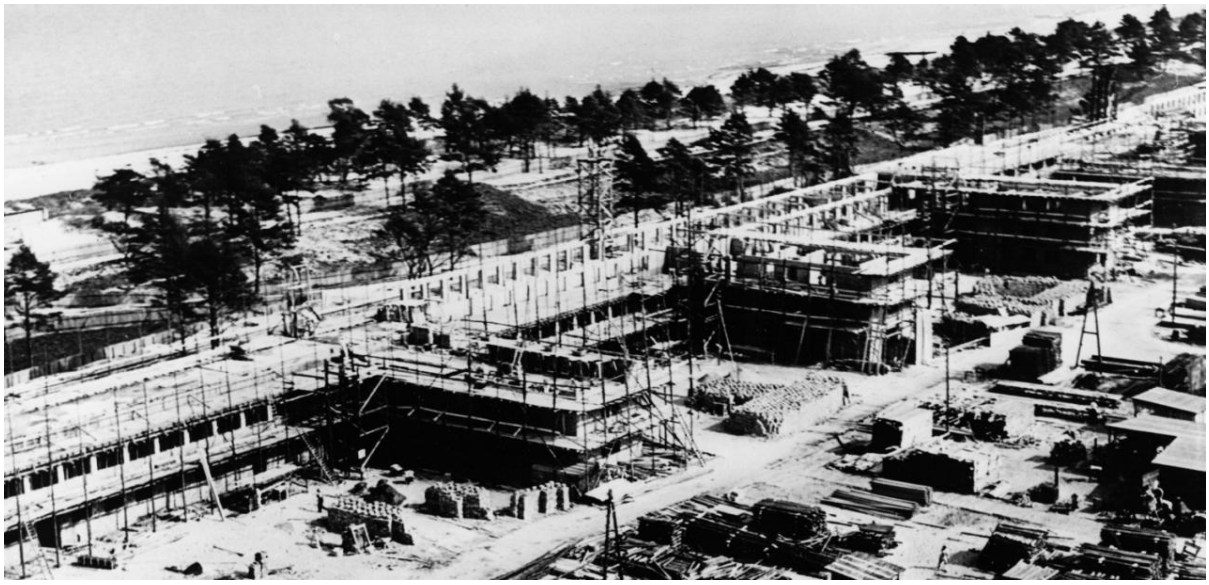


Fig. 4. Building under construction [5]



Fig. 5. Building under construction, photo by Bundesarchiv [4]

3. Colossus as Heritage

"The Colossus of Prora is interesting from two sides – as an extraordinary huge structure, and as an example of pre-WW II architecture which reflects values of the epoch. Basically they are values of a former totalitarian regime, but superposed to values of architecturally advanced society, which also gave Bauhaus to the World, among other contributions, in years immediately before the Colossus was erected. The complex was listed as heritage in 1992, because it was considered to be rare example of the Third Reich Architecture, built for entertainment and recreation, and not either for industrial or military purpose. Decisive for the Monument Protection Authority were: "the impressive and characteristic cubic structure of the building body", the "reinforced concrete skeleton

construction, cast on site" (Fig. 6), and, in addition, necessity of saving it from becoming another "abandoned building".

Embracing principles of sustainability, in nowadays Europe, it is common to preserve out-dated, abandoned industrial buildings without particular artistic values, aesthetically insignificant, for the sake of their size (being hard to demolish), its symbolic value or/and as element of cultural landscape. It is unusual to apply such approach on a holiday resort, but in general, that is what happened in this case. Colossus is preserved for its size, which defines a skyline integrated with beach of the Baltic Sea. Regardless of its beauty, according to nowadays criteria, the Colossus generated cultural landscape, and became landmark whose lose would make the area unrecognisable. Prora complex was different from other war buildings, because it is not preserved for sake of some 'threatening memory', normally based on aesthetic of ugliness (as in case of concentration camps), and therefore its rarity, as much as the size, make it valuable and worthy of preservation.

The logic of its revitalisation is essentially the one typically applied to industrial buildings – with loose limits to intervention, allowing changes of façades. Economic sustainability is expected to be provided on site (which is not typical). It has little economic impact to the development of the area. – which is at the moment considered to have already too many tourists. From the point of view of concept of economical sustainability, Colossus revitalisation can be compared with very successful City G in Vienna (Gas factory revitalisation in Siemering), and probably learn from it. Similar to City G, most of the Colossus will turn into comfortable apartments for residents, and the rest of it in variety of carefully chosen purposes.

The environmental sustainability of Colossus revitalisation is somewhat ambiguous. Such building cannot be good for the environment, generally speaking. However, it exists very long and living environment is well adjusted to it by now, therefore radical change would be shocking. Demolishing such huge building, as alternative to preservation, would produce huge amount of debris which would be devastating for the environment. So, from point of view of sustainability, preservation is fully justified.

Finally, Colossus remains mostly challenging from the point of view of heritage management. It is well known that it is possible to adapt even an extremely huge, over 80 year old, building complex for needs of contemporary life. However, it is not yet proven that current strategy would be entirely successful". [3]

4. Extent of Revitalisation Interventions and impact on structure

Five of the 8 blocks still exist. The last among many attempts for revitalisation of Colossus came in 2012, when the complex was sold to private investors, whose plan for rehabilitation included luxury holiday apartments and a 5 star hotel, for 3,000 people, with an indoor swimming pool and a wellness area (which were partially realized). By removing interior walls it became possible to receive larger apartments. Additional measures were: reinforcing the foundations, cutting out the masonry walls for doors and the erection of lightweight partitions" (Fig. 6) [3].

"One of the requirements of Monument Protection Authority was to keep the board formwork of the ceilings visible. On the other hand they agreed to equip every apartment by a terrace or light cantilevered balcony. Freestanding parking houses have been built. Colossus reflects genuine sensibility of its architect, Klotz, which has not changed much even in decades of architecture practice following the end of War. Klotz`s understanding of architecture, served the particular political regime, but it reflects values which are praised, in general, by every totalitarian regime. Therefore, a part of complex is justly preserved as museum. The rest of it significantly changed. It was widely accepted that the Collossus must be adopted for new purpose…with interventions including façade remodeling" (Fig. 7) [3].



Fig. 6. Interior under reconstruction (photo on the left by M. Hartwig [3] and new lightweight partitions on the right)



Fig. 7. Floor plan of a 3-room apartment and living room [7] and façade

Range of interventions regarding structure can be briefly summed up as the interventions which include change of load (increasing in some cases and decreasing in others), and introducing new forces (as in a case of added terraces), and therefore creating the new input for calculations of structure.

5. Discussion: Technique of Reinforced Concrete of Colossus in Historical Context

Reinforced concrete was known since Roman times, but it was "introduced" to Europe again in the middle of 19th century. Among those who pioneered use of reinforced concrete were Francois Coignet (in 1853, for four-storey house in Paris), William B. Wilkinson (for two-storey building in 1854), Joseph Monier (in 1877, who was granted patent for reinforced flowerpots and therefore considered principle inventor) etc. In Germany, a civil engineer, G.A. Wayss bought the patent rights in 1884 for Germany and further developed the technology. However, in books on history of architecture, mostly praised contributors to popularity of reinforced concrete are August Perret and Le Corbusier. August Perret designed in 1930s the Mobilier Nationale in Paris (1937) and the French Economic, Social and Environmental Council building in Paris (1937–39) (Fig. 8). At the same period, Le Corbusier, who was already beyond the Plan Voisin (1925), and the Villa Savoye in Poissy (1928-1931), focused on experimenting with different urban forms e.g. Radient city (Cité Radieus), city Algiers, Rio Janeiro Plan etc, which were essentially based on reinforced concrete. He used skeleton structure made of reinforced concrete for exploring open floor plan, and free ground level for communication (Fig. 8).

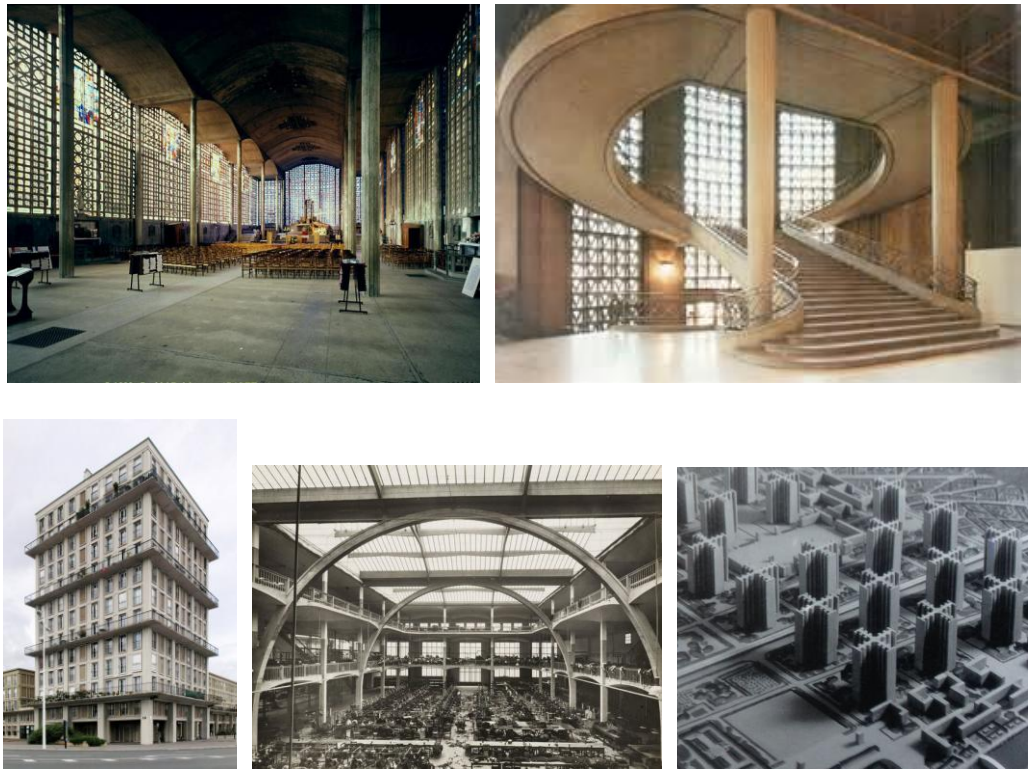


Fig. 8. Several August Perret's projects of reinforced concrete between the two WWs [8] and Le Corbusier's Plan Voisin down, in the left corner [9]

Conceptually, Colossus was fundamentally different. Without free ground floor, it reflects different spirit then e.g. le Corbusier's projects - in architectural sense reflects strength and not grace, as well as fearlessness and equality, and not Le Corbusier's fairness wrapped in social mercy. More importantly, from engineering point of view, Colossus significantly differs, from Perret's own purse. Unlike Perret's projects, it demonstrates *massive* use of reinforced concrete. Klotz structure takes advantage of characteristics reinforced concrete foe which it is used nowadays: low tensile strength and ductility, easy transportation, simplicity of application, relative strength, high toleration of tensile strain, good bonding, irrespective of pH, moisture, and similar factors, thermal compatibility, not causing unacceptable stresses in response to changing temperatures, durability, irrespective of corrosion or sustained stress etc. Therefore, use of reinforced concrete in Prora resembles to its nowadays massive use e.g. for dams (Figure 9). It is worth mentioning that in the first decades of 20th century, the idea of building as "machine" for working, living etc, was popular.

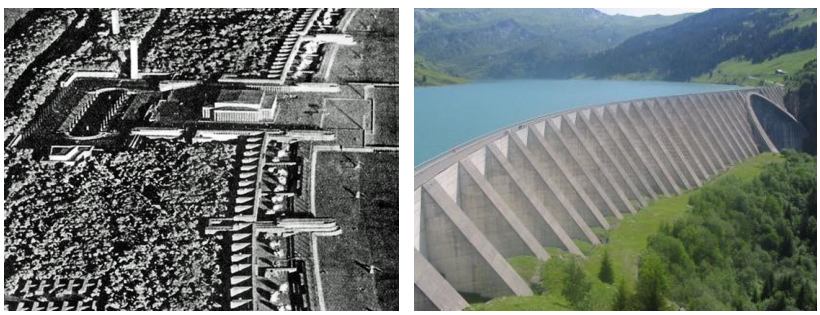


Fig. 9. Prora (on the left) [4] and Canning dam of reinforced concrete in Western Australia (on the right) [10]



Fig. 10. Former Berlin Airport (Flughafen Tempelhof) by 1936-1941 [11]

In the same period when Prora was build, Berlin airport was under reconstruction (built in 1923). That 1.2 km long building was the biggest airport in the world. Obvious resemblance and open "competition" in size points out why size of Prora mattered and implies Prora was created in semi-official architectural style of the Third Reich.

6. Conclusion

The approach of heritage preservation of Prora, similar to those typically applied to industrial buildings, is not random. The analyses of structure reveal that reinforced concrete dam was probable inspiration for Colossus of the architect Clemens Klotz and that heritage interventions naturally followed initial actions. Colossus was building which reflected well spirit of time when it was built which justifies its preservation as heritage. Essential proof for those statements comes not from architecture of Colossus but from analysing its structure and materials it was built from.

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THE CONSERVATION OF THE WALL PAINTINGS OF THE ROYAL CHAMBER AT HERODIUM

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Abstract: *The magnificent palace of Herodium was built by Herod the Great (73-4 B.C.E.) between 25 to 10 B.C.E. in the Judean desert not far away from Jerusalem. The complex includes Mountain fortress-Palace, Lower Palace with bath house, pool and gardens and a small theatre with decorated observation room – The Royal Chamber. Some years before his dead Herod modified artificially the shape of the mountain destroying and covering with debris the Chamber and the theatre. In this way, the site was transformed to a burial monument including monumental staircase to the mausoleum of the Herod's sepulchre.*
This article follows the extremely complicated conservation process of the wall paintings decoration of the Royal Chamber from the excavations until today.

Key words: *research, documentation, environmental monitoring, condition assessment, management sources of water penetration, conservation wall paintings*

Excavation

The site was excavated in 2008 by prof. Ehud Netzer (Fig.1) and immediately after the excavation a chain of mistaken decisions influenced the conservation treatment and leaded to deterioration and loss of valuable paint layers. The work of the team of the Conservation Department of the Israel Antiquities Authority begun in 2012 affording really difficult conditions and problems.

Description of the monument

The theatre built at Herodium was a small intimate structure which could seat 300-400 spectators [1]. The group of rooms built at the back of the theatre is particularly interesting especially the central room – the "Royal chamber" (Fig.2). The magnificent decoration of the room included unique stuccoworks (Fig.3) and a series of "hanging" pictures ("pinakes") against a white background – imitation of "windows to another dimension "(Fig.4).The decoration of the room represents the most beautiful decoration from the Hellenistic-Roman period discovered in our land (Fig.5).

Conditions of the wall paintings

The conditions of the wall paintings in 2012 were related to 4 main factors that can be described also like historical events:

1. The original building and painting techniques.
2. The Herod's decision to reshape the mountain and by consequence the destruction and burial of the building.
3. The discovery and the way of digging.
4. The conservation and protection of the site until the end of 2011.

The original building and painting techniques

The complex was built on the slope of the mountain by carving in the bedrock, leveraging terraces (the floors) and building walls with big well-dressed stones and lime mortar. For part of the building walls (in the South mainly) the bedrock was simply carved and plastered. The decoration has two faces: the first one was executed in "fresco" technique and the painted part is only in the lower register. The second decoration is executed on a thin layer of plaster covering the first painting. The antique masons did small perforation holes in the old plaster in order to make better connection between the new and the old plasters but obviously this was not enough. The carbonated paint layer of the first layer doesn't absorb water and is really smooth. In brief the second plaster was executed hastily without good adherence to the substrate. The stuccoes were executed with lime mortars and the elements attached to the smooth surface of the second layer of plaster after making some scratches on the surface for better adherence. The painting was executed using the "secco" technique – with addition of binder (glue) to the pigments. There are indications that the surface was very dry and not isolated before the application of the pigments. How all this affects the conditions:

- Detachment of plaster: from the beginning the upper layer of plaster was with relatively bad adherence to the substrate and this is the main cause for the observed detachments between layers after the excavation (sometimes arriving to 80% of the surface).
- Detachment of stucco elements: prevalent part of the elements was destroyed during the dismantlement of the building, but the task was very easy because the lacks of connection between plaster and stucco.
- Loss of large painted areas: this happened in the antiquities and possibly is related to the fast absorption of the binder by the dry and not isolated plaster. The paint layer simply disintegrated without the binder. The possibility that the activity of microorganisms was the cause of this particular deterioration is probable in the same degree.
- Disintegration of paint layers: the "secco" technique is very sensitive to the fluctuation of the environmental conditions in first place humidity. Some of the areas were affected under the debris of the destroyed building leading the total loss of the paint layer. This happened mainly on the South wall because the way of water movement on the slope.
- Oxidation of pigments: The red colour of the upper layer is sensitive to light and oxidation and during the excavation changed to black in some area. The phenomenon is related as well to the "secco" painting technique.

The Herod's decision to reshape the mountain and by consequence the destruction and burial of the building

The work was executed by workers living inside the rooms. They leaved traces of their everyday life – walls, the kiln of their kitchen, traces of fire on the walls etc., together with graffiti and paintings on the walls (Fig.5). The physical destruction of the building was completed with partial reburial with debris.

How all this affects the conditions:

Partial loss of walls and decoration: the complex was destroyed following the new shape of the mountain. By consequence the remains are following this line. Part of the decoration was deliberately destroyed, part was destroyed together with the falling walls and part was affected because the particular conditions under the reburial.

The discovery and the way of digging

The excavations were executed step by step and every step affected badly the paintings:

- The discovery of the upper part of the decorated wall and the lack of conservation measures preventing salt crystallization leaded to loss of paint layer and oxidation of pigments.
- The digging of the Central room half way (Fig.6) and building a shelter without to dig from the South and West created dry spot in a very humid environment. All the water from the surrounding with the diluted salts evaporated thorough the upper level of the walls affecting paintings and plaster.
- The following digging of the Central room until the floor without to dig from the South and West aggravated the conditions affecting this time the lower level of the paintings and plaster following the same mechanism.

The conservation and protection of the site until the end of 2011

The conservation was performed without proper understanding of the environmental and micro clime conditions under the shelter. This is the main reason for the non-effective harmful intervention.

- The conservation measures didn't take in consideration the antique painting technique and the used materials in general were not compatible with the environmental conditions.
- The work was performed without scientific and research support.
- The cleaning of the paintings was performed by workers without proper training and during the operation irreversible damage was done. Traces of hard brushes, scratches with scalpels and other sharp tools together with the obvious lack of sensibility to the painting layer are the testimonies of an extremely unsuccessful operation (Fig.8).
- The consequences of the intervention together with the precedent point were: loss of around 30% of the discovered paint layer, unresolved problems of drainage and water penetration (of liquid water!), unresolved salt crystallization (Fig.9,10) and plaster and paint disintegration (Fig.11) and lack of conservation methodology.
- In addition the shelter doesn't provide enough protection from the rain and permit water penetration from the top of the wall and mainly from south thorough the not protected bedrock, there is no drainage system and all the water from the surrounding is concentrated under the shelter. The relatively dry environment under the roofing is the cause for permanent cycles of salt crystallization and on-going deterioration of the plaster. In addition harmful micro organism's growth was observed because the favourable micro environment (Fig.7).

Conclusions

We have a sample of an intervention done without to take in consideration the environment and the interaction between antique materials and changing environment, without respect to the newly created conditions by the excavation.

In brief we afforded a complicated type of project were is necessary to deal with different kinds of and ways of water penetration to stabilize the environment and minimize damages. In our case we have:

- Water coming from the non-excavated and not protected by the shelter areas by gravitation, infiltration and rising damp
- Water coming from the leaks of the shelter
- Water coming by infiltration from the not–treated and not protected by the shelter walls of the complex (lack of copping and joint filling)
- Rising damp because the lack of drainages and walls insulation
- Condensation water because the insufficient roof insulation



Fig.1 The Upper fortress, the theatre and the Royal Chamber excavation

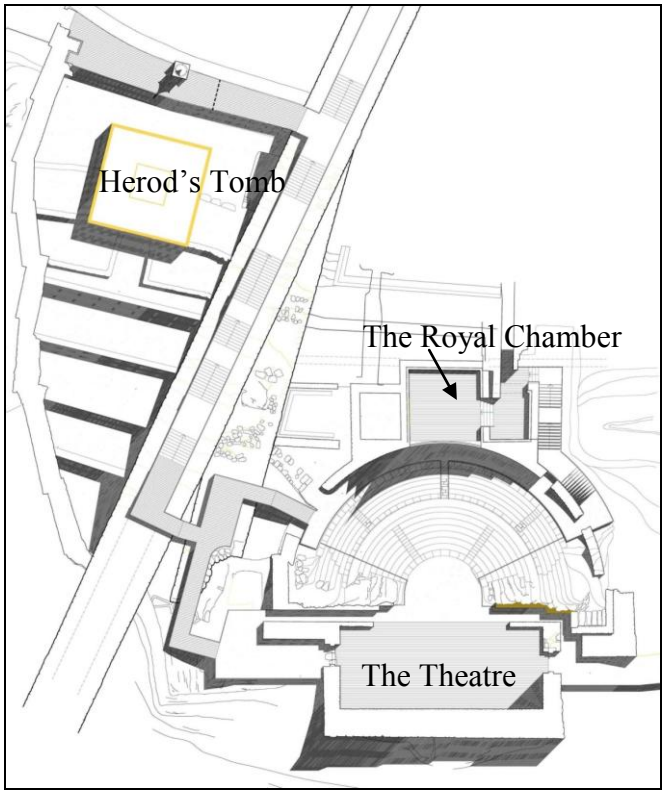


Fig.2 The plan of the site



Fig.3 Magnificient stuccoworks



Fig.4 Windows to another dimension



Fig. 5 General view of the Royal Chamber



Fig.6 The Chamber excavated halfway



Fig.7 Microbiological attack



Fig.8 Traces of cleaning with hard brush



Fig.9 Salt efflorescence



Fig.10 Dammage because sub-efflorescence



Fig.11 Detachment of paint layer

Conservation decisions

In these circumstances was obvious that a new conservation approach based on scientific research and analysis of the environment related to the deterioration processes was needed. The new approach was based on a scientific understanding of the problems and deterioration factors and several operations aiming to limit the water penetration to the site. This understanding was crucial for the decision making process and the order of

intervention. Only by elimination of the main sources of water was possible to establish the right conservation strategy and to identify compatible materials.

Conservation methodology

The methodology was based on a holistic approach to the site and included:

- Identification and mapping of problems
- Identification of deterioration mechanisms
- Research on antique painting materials
- Salt testing
- Microbiology growth testing
- Identification of sources of water and ways of water penetration
- Planning of preventive intervention including:
  1. Excavating the areas west and south of the Royal Chamber
  2. Building additional shelter from East
  3. Expanding and repairing the existing shelter
  4. Treating the top of the walls of the adjacent buildings
  5. Insulation of the south and west walls
  6. Infill existing archaeological pits
  7. Systematization of the drainages around the Chamber
- Planning long term monitoring
  1. Installation of Drytronic system
  2. Monitoring conditions
  3. Monitoring environment (T, RH) for 18 months
  4. Monitoring Moisture content on a regular basis
- Planning a conservation intervention including
  1. Emergency conservation treatments
  2. Testing compatible materials
  3. Micro grouting
  4. Salts extraction
  5. Consolidation plaster
  6. Consolidation pigments
  7. Consolidation stuccoes
  8. Cleaning
  9. Integration of lacunas
  10. Creating test areas with proposals for the final presentation of the monument
- Planning a new shelter and drainage system
- Construction the new shelter and drainage system
- Monitoring conditions under the new shelter

Conservation intervention:

The work was carried on from the beginning of 2012 until today March 2015 and will continue until the completion of the new shelter and exhibition of the Royal Chamber to the public

Emergency Conservation treatments

Archaeological digs: the area south of the chamber was excavated to the bedrock. In this way the zone was cleaned from water absorbing materials and was possible to perform the insulation and drainages of the chamber.

Conservation walls – the treatment includes mainly the structures south of the Chamber and consists in rebuilding, consolidation, plastering, pointing and copping the walls of the entire complex

Drainages: the drainage system was planned in order to afford future problems as well because the problem of water evacuation in this very complicated site is crucial for the conservation of the remains. Possible scenarios were studied, but the final decision should be a function of the future way of site presentation and involve the whole site

Shelter: the existing shelter (Fig.15) was enlarged in order to cover maximum possible space (Fig.16). In addition a shelter was built over the Eastern room and the floor of the western room insulated with plaster.

Insulation: The exterior parts of the walls were plastered to prevent water penetration. Excavation pits were backfilled and insulated, the bedrock surface was insulated with rubbles and plaster and drainage channels added when necessary

Environmental Monitoring: In order to understand the micro clime conditions under the shelter 3 data loggers were installed and work at the site. The data was collected during 18 months and processed every 3 months.

Internal moisture content was collected every 6 months with Moisture Psychrometer monitoring device.

Physical condition monitoring: The problems were mapped on base map and the map was updated in regular basis every 3 months (Fig.12).

Research

Microbiology: The main conclusions were that the cause is the presence of water and favorable environment created because the use of some specific organic conservation material.

Salt testing: Presence of sulfates, chlorides and soluble carbonates were observed.

Painting techniques: the samples were studied in the laboratories of the Courtauld University of Art, London. Additional samples were sent to laboratories in Italy in order to try to identify the binder of the secco painting.

Testing of conservation techniques and materials

Basic restrictions - the choice of materials was limited by the properties requested:

- Compatibility with the original materials
- Use of minimal quantities of water for application
- Vapour permeability

Various materials and techniques were tested for grouting and micro grouting, lacunas fill and edges of plaster consolidation, structural consolidation of plaster, consolidation of pigments. The results of the test were reassessed and the chosen materials implemented.

Tests for salt extraction were performed successfully after the consolidation tests using the Drytronic device in extraction mode.

Conclusions from the testing and emergency measures undertaken

Archaeological digs: the performed additional excavations improved visibly the condition of the wall painting together with the whole complex of measures undertaken.

Monitoring: The monitoring of the environmental conditions and moisture content was performed together with the monitoring of the wall paintings conditions and revealed that the problem of humidity penetration persists (Fig.17). This indicated that the undertaken measures were not effective enough to deal with the problem. The testing of the Drytronic system was decided after consulting with experts from Germany.

Tests of materials: The successful results of the tests were used for the treatment of the decoration. Nano lime (CaloSil) and acrylic Nano-emulsion (AcrilME) was used for consolidation of plaster and pigments respectively together of lime mortars.

Conservation works 2012-2015

The intervention was based on the performed research, testing of materials and techniques and analyse of the previous intervention.

Installation of the Drytronic system: the device was installed and tested for 3 months with excellent results. The Moisture content in the western wall decreased with the time arriving to some level of stability (Fig.18). Fluctuations due to torrential rains and conservation treatment with excessive use of water were observed without to change the whole picture. The Drytronic device operates using system of graphite electrodes inserted in drilled holes in the wall, fixed with mortar containing graphite powder (Fig.14). The device create electrostatic field "pushing" the water to the core of the wall and to the soil. There is no complete drying of the wall, because the device needs some humidity level in order to perform.

Monitoring: The Moisture content was measured in fixed point every month with Moisture Psychrometer device. The data was presented in a graphic way.

Intervention: Involves consolidations of edges of plaster, lacunas, pigments and stuccoes, grouting and micro grouting of voids.

- Consolidation of edges and lacunas: all the previous interventions with not compatible mortars were cleaned and replaced with new mortar. All the lacunas were treated according to the conditions of the surrounding surfaces to integrate completely.
- Grouting: the injections of hydraulic mortar were performed in order to arrive to 80% level of stability and the treatment repeated after reassessment.
- Consolidation of plaster: performed with CaloSil were needed
- Consolidation of secco paint: performed with Acril ME
- Consolidation of stuccoes: the remains of the decoration were consolidated with lime mortar; detached parts were re adhered to the wall using small diameter fiberglass pins and lime mortar.
- Salts extraction: performed after consolidation of the paint layer with poultices of de-ionized water, Japanese paper and paper powder. On the west wall the Drytronic device was activated in poultice mode and the salt extraction was performed on the lower layer of paint.
- Cleaning: this operation was extremely complicated because during the previous intervention very high concentrations of consolidation materials were used together with unknown (not mentioned in documentation) materials with highly negative impact on the conditions. In addition was needed to clean areas not properly cleaned during the precedent intervention in order to arrive to some acceptable level of cleaning without to make any new damages (Fig.13). Different solvents and techniques were tested before to arrive to the right cleaning methodology. Soft solvents and careful mechanical cleaning were combined in order to clean properly the paintings.

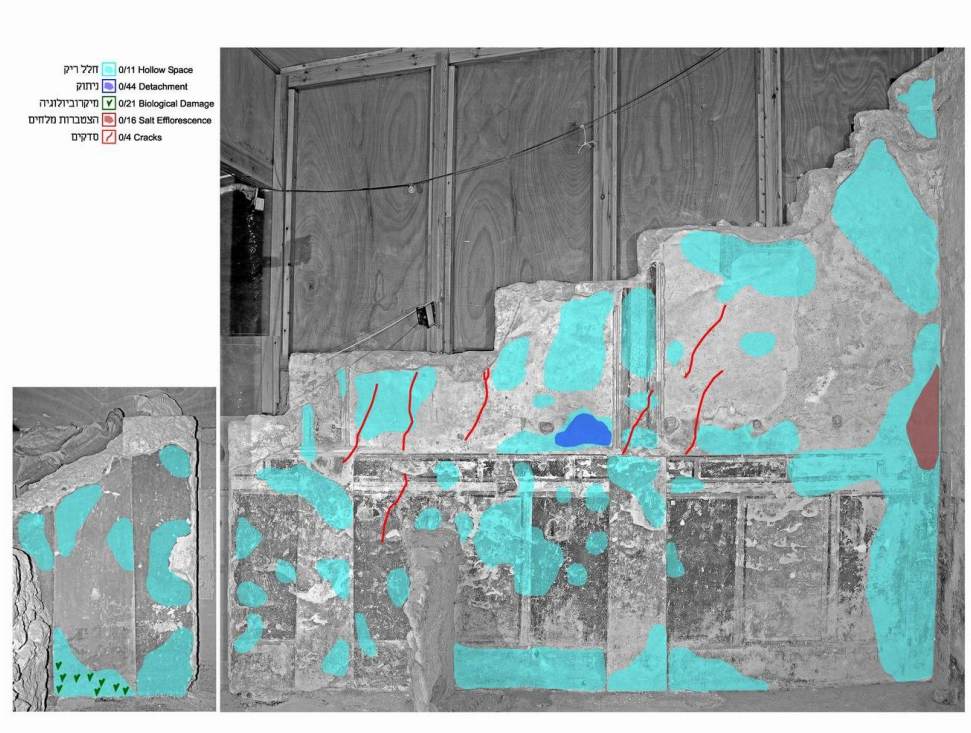


Fig.12 Condition base-map (East wall)



Fig.13 Cleaning samples

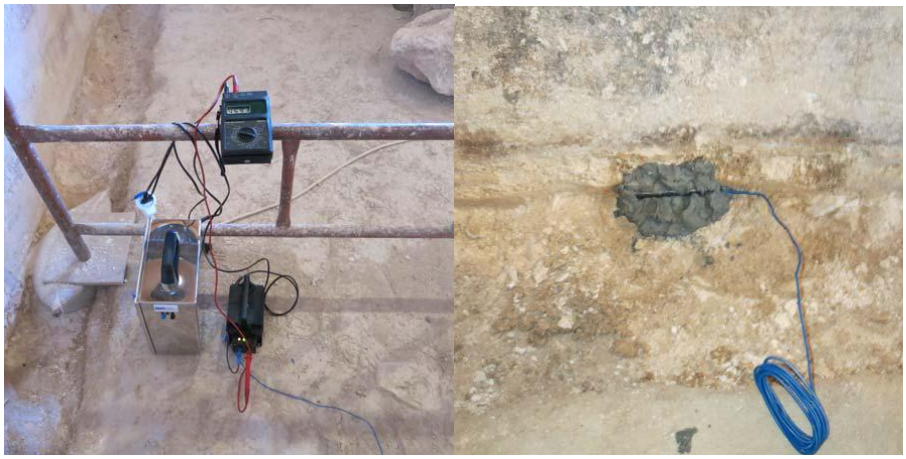


Fig.14 Installation of the Drytronic system and electrodes



Fig.15 The shelter 2011



Fig.16 The enlarged shelter 2012

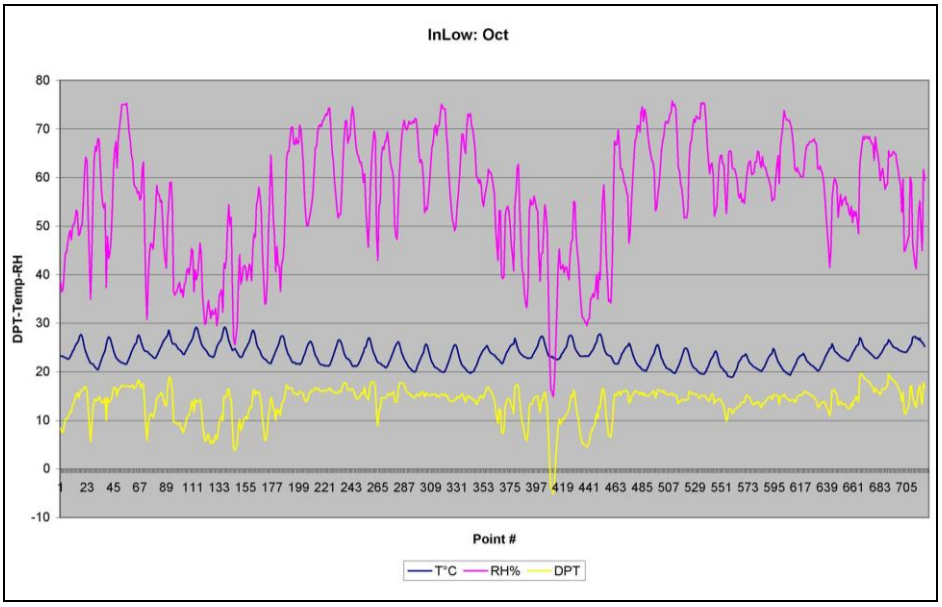


Fig.17 Monitoring environmental conditions (T and RH)

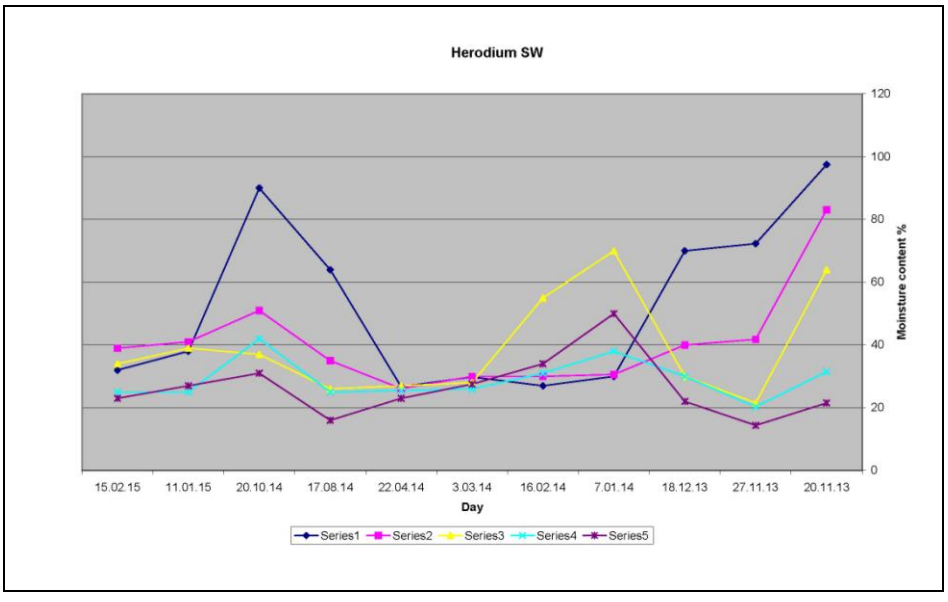


Fig. 18 Monitoring Moisture content after the installation of the Drytronic

Presentation and integration of the wall paintings

The presentation of the wall paintings to the public is related to the transformation of the site to museum exhibition and the "mise en valeur" (emphasizing of the values) of the monument. The treatment is limited by:

- The historical evidence: the Chamber was deliberately destroyed during the last period of the Herod's life. The remains were buried partially with debris and deterioration processes were active for very long period.
- The condition of the painting during the archaeological digs: a lot of original material was lost and the appearance of the paintings affected. In addition fragments of wall paintings and decoration (stucco) were transferred to storages and museum exhibitions making the theoretical restoration of the monument very problematic.
- The present conditions of the paintings: after the conservation the monument doesn't represents the values of the original. In order to restore the esthetical and historical values of the monument an intervention should be performed.
- The scope of this intervention, based on the existing information and respect to the original materials and historical evidences, will be to present the values of the monuments in a minimalist way with the intention to make them understandable for the public.
- The methodology used to achieve all this will be to integrate the knowledge about the monument with the existing evidences and materials to present the continuity of the decoration. In order to establish the right approach to the integration test areas (accomplished in 2015) were treated in a different level within the following restrictions(Fig.19):
- The intervention must be completely reversible
- Original colours are not recovered with retouch in any case
- The treatment is a function of the conditions and the percentage of existing original
- The intervention will take in consideration the archaeological documentation during excavation
- The intervention will take in consideration the existing materials in storages and exhibitions and will try to use some of them in the integration process.
- The intervention will be done with full respect to the history of the site: graffiti and paintings performed by the workers during the destruction of the Chamber, evidences of the destruction on the walls etc. will be conserved and presented in the frame of the whole project.



Fig.19 The samples for integration



Fig.20 The permanent shelter 2016

Monitoring conditions and maintenance

During 2016 the new shelter (Fig.20) with the complete drainage system was built and the project entered in the final phase. The new environment created new problems and we wait to have more problems when the air conditioning system will be installed. In the meantime the team is monitoring the environment, the influence of the new conditions and performing maintenance of the paintings. Obviously a reassessment will be needed and new challenges to be afforded will arise – the conservation is an endless process…

Lessons learned

Again we learned that we never learn, not from our mistakes and not from the mistakes of the others. The conservation is no more a profession for self educated artisans and the archaeology is not a movie for Indiana Jones. Only holistic scientific approach from the first day of the excavation and the full collaboration between archaeologists, conservators and scientists can give positive results and achievements(some times, because we are limited by our knowledge and abilities) in the field of the Heritage preservation and conservation.

Acknowledgement

Sharon Cather (Courtauld University of Art – London) and Maurizio Tagliapietra (Scuola Regionale di Verona) visited the site and send back to us reports and recommendations. The consulting works in a permanent basis with testing results, questions and reports exchange between the persons involved. Christoph Schmidt from Germany consulted us about the possibilities of the Drytronic system for moisture control in the walls.

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SOCIAL AND ECONOMIC INTEGRATION OF CULTURAL HERITAGE - IN CASE OF TRADITIONAL ARCHITECTURE ON RURAL AREA IN SERBIA

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Abstract: *National architecture falls among the most important achievements of one nation. As a rule, the objects of folk construction are unfairly neglected in relation to other forms of cultural heritage, due to the nature and the small durability of the materials from which they were built, as well as the very area on which they were built - mostly underdeveloped poor peaks. The paper deals with the social and economic integration of this form of cultural heritage in Serbia. Examples of good practice from Serbia and the region are presented, with the aim to raise awareness of this type of heritage, and to show that the existence of traditional construction in one rural area can increase the chance for its development.*

Key words*: Cultural heritage, national construction, rudimentary space, tradition*

1. Introduction

The current global economic recession, as well as the multifaceted social and economic transformation in the Balkans, to which Serbia belongs, is a socio-political stage where the drama of the present moment takes place in a wide spectrum of manifestations. This contemporary moment implies certain actions and reactions of a society on all aspects and factors of development, especially in three domains of existence: ecological, economic and social.

Regardless of the strong message of today's modern and developed world that all processes are interdependent and conditioned, and that development and protection in one

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sector can not be viewed in isolation, but as part of a wider picture and mechanism of social development, this inertia is immanent to this space. It is clear that the same fate divides culture and cultural heritage as one of the very important material testimonies of our duration and in the temporal and spatial continuum.

The term SUSTAINABLE DEVELOPMENT promoted back in 1987, which basically defines: "a development concept that meets the actual needs of the inhabitants of a settlement in a certain area, but does not compromise the ability of the next generations to meet their needs."[4]

Many works, which are part of the world's cultural heritage, are waiting for their order to be adequately valued, then adequately protected, preserved and presented to the public. As a part of the memory of one place, the testimony of past centuries and some lives, which are witnesses of our duration, development, the growth of one culture are inextricably incorporated into our existence.

When we talk about the remains of earlier civilizations and past centuries, which in the territory of Serbia can be recognized and monitored at several locations of the world, the perception of their importance and scale of values is somehow clearer. The UNESCO World Heritage List contains and recognizes the objects of our architecture and material cultural heritage, and it looks like this list is expanding to some very important sites. In this way, these objects and sites are protected, recognized and affirmed in some way.

The problem of cultural heritage that is embodied through the traditional architectural heritage, however, has a different treatment. Since this aspect of cultural heritage is predominantly located in the rural area, and this is synonymous in Serbia for an underdeveloped area, with almost no exception, this part of the material culture has much less treatment. Therefore it can't be distinguished as an independent and separate element, but it is only right to see it through the development possibilities and the current moment in the development of rural space.

If the chances and opportunities for the development of a rural area are higher, this means that the objects of traditional architecture will be more easily preserved in this case. It can be discussed: does it still apply? Namely: does the existence of a cultural heritage in a rural projection in the form of traditional construction increase the chances for the development of this?

Through examples of good practice and some bright experiences, this work will examine whether it is possible to strengthen and increase the chance for development, and how the generator of such a development impulse can or may already be a cultural heritage in the form of traditional construction.

2. Traditional architecture

The diversity and peculiarities of geographical conditions in some parts of Serbia have also caused differences and special forms of traditional architecture in this area. These differences in the way of organizing the space, as well as the specifics in the way of

[4] S.D.-Sustainable Development, Brundtland Commission (Commission Brundtland). Formerly known as the World Commission for Environment and Development.

construction, arise from the rich experience of Serbian anonym masons, self-taught builder, who adjusted their solutions to specific living conditions.

Work and life, with special conditions that make up the city and cultural differences, customs also had an impact on the solutions of traditional architecture. As a basic characteristic one can distinguish the utilitarianism and the applied aesthetics of the material itself, which it did not use. Disposition of the object, mutual relationship, and even its construction and proportions are never wrong. This ArchitectureSO

National construction in Serbia is the result of the influence of numerous Old-Slavic cultures with the Indigenous (Thracian and Illyrian), Roman, Byzantine, and later Oriental. The last great influence came from Central Europe (Austro-Hungarian), technologically (and today) developed world, bringing the modern way of construction and a new culture of housing. (Deroko 1968, Kojić 1973).

2.1. Basic characteristics of traditional architecture and determining elements

Traditional architecture has great potential, and as a teacher of the next generations of builders, and as a witness of time and product of the socio-economic conditions in which it was created. The way of life is one of the basic elements that influenced the design of this architecture. The house is not only a shelter, a place of residence, but also a commercial center. It should realize the conditions and the space to perform all the activities that follow the obligations of the household, and so there are different types of space and related contents, depending on the activity.

The climate has its own peculiarities and sets different requirements of the order of the object: the way of covering, opening of the opening holes, their number and size, as well as the appearance or absence of the ground, are directly related to the average temperature and amount of snowfall in one climatic zone. The material that appears on the site of construction, and in the immediate vicinity, is implied. It is often a pletar or a kerchief, or in the Pannonian area of the charge, and is covered with straw, reed, shingles or ceramide.

In addition to the characteristics that apply to the use of natural materials, from the immediate environment of the place of origin of the house, it is also a feature of measurement. The measurements were anthropomorphic, so the rate, the palm, and the arches appeared. Arshin is an old measure of length, which masons used most. What Le Corbusier mentions in Le Modulor is the total sum of the elbow and the rate of 45 + 30 = 75 by ~ 76 cm, and based on the calculation of the mean values that Neufert has done."[5]

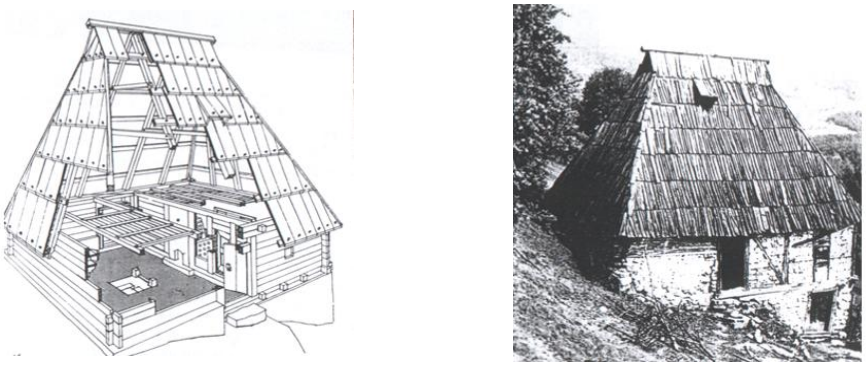
Under different conditions, there are different response to these conditions, so the houses of traditional architecture are distinguished by their appearance, construction, situation and mutual disposition. Based on these elements, several regional types have developed in Serbia.

[5] Đorđe Petrović: Neimarska pravila i aršin. Doktorske Univerzitetske Disertacije, časopis Univerzitet danas, Beograd, 1973, 46str

2.2. Typology and basic characteristics

The aforementioned various tasks, which were posed before these objects: relief, material, conditions and customs in certain parts of Serbia, differ, as are the villages that are also different. Villages can be by the place of origin: flat, mountainous, mountainous; and according to the type of built settlement is compact, broken or combined type.

There are a number of objects encountered within a village farm, such as: buildings of economic character, hay and grain hamburgers, maize bins, dairies, cakes, dryers, kayaks, or by the vineyard pubs; then houses for housing - the same houses, sculptures, huts or cisterns along the place for livestock grazing, and so on. The basic house for housing, about it will be words in the further work.



a) b)

Fig. 1. Dinar logs: a) circuit and organization; b) outward appearance (Ranko Findik, "Dinarska brvnara", 1998, p.85)

The shape of the house can be: one-sided - in which the host lives in the same area with the livestock it owns; double-sided - those that separated into a separate building for the host, consist of a "house" (with a fireplace) and a "room" where it is sleeping. Later, a more complex view of the house appears, multiphase - which is at a higher level of development.

It is mostly a house with only one room and a fireplace in it, and the main difference in shape and appearance will determine the material from which it is built, as well as the conditions of the relief and climate where it is built. Central and western Serbia, which are rich in forests, with a sharp climate with plenty of snow, have a characteristic house, built of wood-timber, it is a house so called *cabin* **(Fig. 1.).**

There are several types of houses throughout the Serbia[6], and each of the types has distinctive characteristics depending on the specific differences in the conditions in which it arises, or from the non-builders who build it.

3. Access to cultural heritage and traditional architecture in Serbia

A modern understanding of sustainable development requires a new multidisciplinary approach in which the rural landscape is not seen only as an agrarian landscape, but as an

[6] Types of houses are: moravska, vojvođanska, šumadijska, kosovska hose.

ecologically preserved environment rich in the resources necessary for the survival of the entire population (clean air, healthy water and food), but also civilization's most important goods (first-class cultural monuments). (Sustainable Development of Serbia 2010)

The largest number of our most important cultural assets, such as archaeological sites, medieval churches and monasteries, various examples of national non-emigration and other forms of cultural heritage, are located in rural areas and villages. (Nenadovic, Pavlovic, 1955, Monumental Heritage 2007)

However, the corpus of architectural heritage in the rural area does not merely constitute a monumental heritage, but a number of artifacts differ in content, function and form, size, age, materialization, structure and other properties. There are various forms of architectural heritage.

As far as the village, the rural area and the traditional architecture and folk architecture are concerned, it must be noted that national construction provides an image of the way of life in that area. It is possible to find out about association, mobi, the old custom of making big works, about the village craftsmen who made the tool necessary for building a house, about the old crafts that participated in the construction of various buildings, which are sporadically mentioned today. National construction gives insight into customs and beliefs when choosing a place for building, laying the foundation and decoration of a roof, a culture of housing and the rest. (Vukosavljević, 1953)

Each of the elements that can be studied and learned from the analysis of this heritage is just one of its parts. Of course there are also possible contents that are adequate as the possible purpose of these objects in the future. This practice has already been applied in some of the best practice examples in Serbia. This will be discussed further.

3.2. Examples of good practice in Serbia

3.1.1. "Old Village", Sirogojno

One of the first open-air museums in Serbia is the "Old Village" Museum in Sirogojno. The first discussions about the Museum program and the first search for the old architecture in the Zlatibor villages began in 1979, and during the summer months of 1980, the first buildings of the future Museum were transferred and built. Until July 30, 1992, when the Museum was officially established, a large number of facilities were placed on this site, six of which were reconstructed according to the old samples, because they could not be transferred.

The open-air museum "Old Village" is a cultural and cultural monument of great importance for the Republic of Serbia, which shows the housing and housing culture preserved for centuries in this part of the Dinaric region. Within this Museum, both material and non-material culture from this area is presented. In the permanent museum setting there are buildings characteristic of the Dinaric cultural region, and in whose interior there are authentic furniture and three fully equipped workshops (*Kačar*-/master for making barrels/, *Blacksmith* and *Potter*). The **main house**, workouts (facilities intended for the accommodation of families of married members of the cooperative household), **dairy** (an auxiliary building used for keeping and processing of milk),

bakery (an object in which large quantities of bread are prepared for the cooperative family), **salad** (holding and drying building corn in the piston) and **amber** (storage for the holding and drying of cereals). According to the characteristic schedule, the objects are placed within a single garden so that their relationship, distances and orientation within the household it represents can be perceived. In the economic part there are economic objects: **kacara** (a facility for keeping the pomegranate and baking brandy), a mouse (a fruit drying building) and a **stand** (a two-storey building for keeping livestock).[7]

Since the protection and preservation of material culture in the overall museum setting implies the preservation of spiritual heritage, in the open-air museum "Old Village", reconstruction of customs and revitalization of old crafts are performed within regular program activities. Intangible cultural heritage is presented simultaneously with material work and reflects the dominant religious, social and economic circumstances. One of the important endeavors, which may have been the most economically powerful for Sirogojno and its surroundings, is activity with the famous Sirogojno sweaters, made of wool, with embroidered motifs of the houses on these sweaters. Many housewives were trained and became co-operatives of the cooperative, which initiated this initiative. To date, these sweaters are a trademark of Sirogojno and Zlatibor mountain in general.



Fig. 2. Open Air Museum, Old Village Sirogojno (photo A.M.-P.)

This also aims at expanding the tourist offer itself (Christmas, Easter, and other customs related to traditional and religious rituals can be seen in the Old Village). And when it comes to visits to school excursions, which are frequent in the Old Village, this aspect of the revival of old customs and trades has a great educational significance and is a great guarantee for the preservation and transfer of tradition to young generations. Also, the Old Village is host to numerous events, music festivals for seven years now, and now (from 8th to 10th September 2017) an international scientific conference of retrospectives and perspectives of ethnology and anthropology was held on the occasion of the 70th anniversary of the Ethnographic Institute SANU.8

[7] Journal From the History of Agriculture UDC 63: 93/99 (05) YU ISSN 0353-412X Year: XV - XVI (2007/08) Notebooks related: 1-2 and 3-4; work by Bojana Bogdanović, Open Museum "Old Village" in Sirogojno - presentation of material and immaterial culture - pp. 32-34.

8 SANU- Serbian Academy of Sciences and Arts

3.1.2. Drvengrad and Andrićgrad

Drvengrad, also known as Mecavnik, is an ethno village in Serbia in Mokra Gora, Zlatiborski district, located on the borders of Zlatibor and Tara. This ethno village was built on Mecavnik hill for the film "Life is a Miracle", according to the director Emir Kusturica. After the completion of the recording, the director expanded the built fund of this ethno-village, and with the power of his popularity, but also with the concept that he offered, he turned it into a tourist attraction, visited by tourists from all over the world.

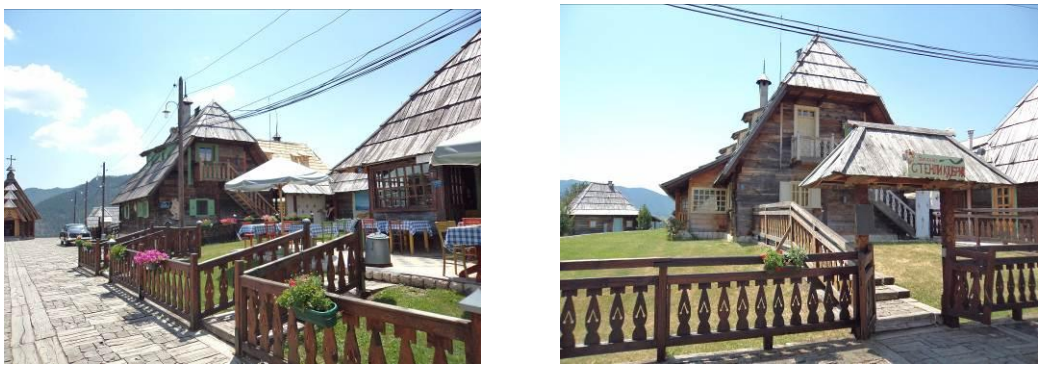


Fig. 3. Mećavnik (photo A.M.-P.)

The concept: "everything that is needed for an active and contentive stay - Drvengrad has," starting from a beautiful nature, then natural-domestic food, which was organized and with the help of locals began to produce and produce the director himself (in the immediate vicinity of ethno village has its own cow farm, which supplies this risort with fresh milk and dairy products). Then the traditional house - as a basic inspiration and space for accommodation of tourists, provides an authentic experience of staying at this place. Integrally, it is a unique and unrepeatable "entry into the retro film", with the remnants of modern comforts that contemporary tourists can demand and expect in this place.

In order for this whole story to have the necessary cultural dimension, which is of great size, such as Kusturica, there are many festivals and cultural events held in Drvengrad, which are celebrated throughout the year by celebrities from the world of film, music and other artistic svera. Kustendorf, is a review of young director's films, which along with numerous workshops and the competitive character of this, every February is the reason for the big gathering at this place. In the summer of July, 5 years ago, the festival Bolshaya, the Russian Music Festival, is being organized. In addition to everything mentioned in Drvengrad, there are a number of sports fields, outdoor and indoor, swimming pool, gym, sauna, as well as equipped ski fields in the immediate nearby. So there is a rich sport offer both in summer and in winter.

An example of extensive planning and a widespread consideration of the development and growth of this ethno-village is an effort to be associated with local authority, to renovate and again put into operation a narrow-gauge railway that goes through the Sargan Eight and a unique opportunity for today's tourists to travel " ". Jatare and other stations on the road of this tourist train, all the way to Andrićgrad in Visegrad, R.of Bosnia, which is a continuation of the story from Drvengrad.



Fig. 4. Andrićgrad, (photo A.M.-P.)

A city which, according to motifs, paintings and scenes from Andrić's literature, in cooperation with the Republic of Srpska, was initiated and built by Emir Kusturica as well. Andrićgrad is a cultural and educational center, with the Andrić Literature Research Institute and expert meetings on various topics. This is a special homage to the great writer, at a place from which a magnificent view of the even more magnificent on the Drina River is provided.

4. Examples of good practice examples from the region

On all four sides of the world in the environment, both immediate and further, this problem of revitalization and modern use of traditional architecture, its fresh and new interpretation has long been recognized as a potential. The European Union has a positive experience in assisting and supporting this kind of self-help, when the rural areas are at stake. This is known and recognized, the inhabitants are the main headquarters on which the activity is necessary, which is necessary to start the revitalization and re-use of these facilities.

The basis for new activities in the old buildings of rural architecture lies in the activities of a modern tourist or visitors to these sites. New needs of a modern man, who appears as the basic "consumer" of such content. There are some good examples in the region, Macedonia, Croatia, Hungary, but we will just stay here on a case from neighboring Bulgaria

4.1. Koprivštica, Republic of Bulgaria

Koprivštica, so called "*architectural reserve*", is one of the finest examples of preserved and shows well-presented folk architecture, how it can be presented in an adequate way. The city located in the western part of Bulgaria, in the Sofia region and the administrative center of the municipality, has 2500 inhabitants. The fact that it is a good part of the buildings of cultural and historical significance, this city is among the most interesting cities-museums. This way, it is possible to present this place in the shortest time, which is made by tourists - both domestic and foreign. The city has always been inhabited by rich inhabitants, merchants, who also had large and beautiful houses, but it also attracted many who destroyed this city and burned it because of kidnapping. Thus Koprivštica burned

three times in 1793, 1804, and 1809., each time it was restored and even better with more beautiful houses.

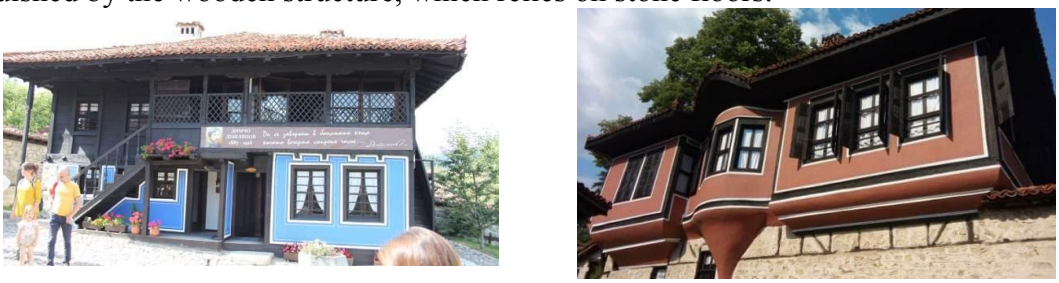


a) b)

Fig. 5. House of the rich merchant Petko Ljutova, from 1854. With the details of the interior. It is rarely encountered in this period of such decoration in the house, (Photo M.V.)

The richer houses also had models in European cities, which can be recognized especially in interior decoration and painting of walls and ceilings. House of the rich merchant Petko Ljutova, (Fig.5.)from 1854. A unique monument of this sumptuous architecture from this period. With the details of the interior, which is painted and has the motive of Venice, with which it was traded by the owner, and from where it was probably brought to this interior decorating model. Next is the house of the famous poet Dimce Debeljanov dates back to 1830. In 1887, he was born in this house. And house of the hero Kableškov(Fig.6.)

The festival of folk creations, then the annual sessions of the Sofia Architects Association, and a whole series of events are part of the offer of this city-museum. The objects are representatives of the so-called "renaissance" period, although due to the time of origination under the influence of Islamic architecture, they are essentially distinguished by the wooden structure, which relies on stone floors.



a) b)

Fig. 6. a) The house dates back to 1830. In 1887, the famous poet Dimce Debeljanov was born; b) House hero Hero of Todor Kableshkov, who was the initiator of the April uprising in 1876 against the Turkish authorities. (Photo M.V.)

5. Types of social and economic integration of traditional architecture in the contemporary rural area of Serbia

There are many reasons to take special care of this richness and a diverse range of cultural and architectural heritage. When we talk about the active social and economic

integration of this heritage, a broad social engagement is needed, first of all in informing and training the population, in order to sensitize to this resource.

Within the framework of various projects and activities initiated by different European Commissions and funds, all with the aim to help in the development of rural areas, as a resource that is unused - special emphasis is placed on traditional architecture.

This wealth, neglected and forgotten, especially in the area of underdeveloped rural areas, can be an incentive to realize a richer tourist offer, then various camps and research centers, or as a place for accommodation of smaller economic units (old crafts, collection and processing of medicinal herbs, mushrooms and others natural products that can be found in our rural areas**).**

When it comes to research centers, there are primarily new types of activity of academic citizens, students, who in the summer period mainly relate to the maintenance of workshops, summer schools (depending on the direction and type of activity at faculties) can take place under these conditions on raul space, using revitalized facilities.

Fore example, in the village of Gornji Krivodol the students and professors of the Veterinary Faculty from Belgrade, revitalized the school in this almost abandoned place located on the Old mountain[9] (the most eastern located place in Serbia) and adapted it to the needs of its students for summer practice. This location was suitable because there are a number of farms in the immediate vicinity, where these students perform their practice and apply the acquired knowledge. There are many good and positive examples, and we hope that there is room for optimism that there will be more and more of them.

5.1. Activism of the inhabitants

The countries of the Western Balkans are implementing reforms of their governments' systems in order to bring regulatory bodies and legal acts into line with the standards and policies of the European Axis. In order to speed up this process, the EU provides support through the Instrument for Pre-Accession Assistance (IPA). One part of this instrument focuses on rural development, including the possibility of using LEADER (the link between rural economy and development actions) access, which works well across Europe. "We need to enable local people to take part in deciding on the things to be done because they are best aware of the problems and ways to deal with them ..."[10]

Positive examples are listed in this paper. In this way, when people in whose territory there are facilities that offer a chance to develop one of the business lines, and can help, provide material support in the form of grants for the reconstruction and conversion of objects of traditional architecture, things will start in a good direction. Awareness of the chances and opportunities, material and professional assistance, as well as permanent education - are a sure way to fully utilize the resources we are writing in this paper.

5.2. Legal framework

Caring for the heritage of a nation is as old as the legacy itself, as it begins in the process of creating a certain object, it continues through its life ... Caring for cultural heritage should be a part of everyday life of all citizens.

[9]Old mountain - Balkan, prim.author.
[10]Dirk Ahner, moderator of the Final Conference Strengthening of the Rural Stakeholders of the Western Balkans.

The first institution entrusted with the care of cultural property in Serbia was founded in 1947. It was the Institute for the Protection and Scientific Study of Cultural Monuments of the People's Republic of Serbia. In 1960, this institution changed its name to the Republic Institute for the Protection of Cultural Monuments. Four years after the founding of the central institution, the organization of the network of institutions of protection on the territory of the Republic of Serbia, of a total of 15 institutes including the territory of Kosovo, has begun. In 1995, the Ministry of Culture determined by its decision the territorial competence of the institution.

The first law regulating the protection of cultural monuments was adopted in 1977. It represents a significant reform of the monumental legislation. According to him, depending on physical, artistic, cultural and historical characteristics, cultural goods are divided into movable and immobile. According to the same criteria, immovable cultural goods are divided into cultural monuments, spatial cultural and historical units, archaeological sites and famous sites. The 1977 law was partially changed. In 1990, the Law on Cultural Property was enacted, and in 1994 the current Law on Cultural Property was in force. (Group of authors: Serbian Heritage Monument, (Editor Svetlana Pejić) Republic Institute for the Protection of Cultural Monuments Belgrade, 2007, 455).

This law also envisages the categorization of cultural goods, depending on the importance of: cultural goods, cultural goods of great importance and cultural goods of exceptional significance. (Official Gazette RS No. 71/94).

6. Conclusion

Based on all of the above, it can be concluded that there are many possible forms of social and economic integration of traditional architecture in the modern rural area of Serbia. Some examples that are already active and represent a kind of *in vivo* experiment, support the fact that with the systematic and permanent education of local residents and the initiation of regular procedures at the local level for the initial financial investment in such projects, a greater number of activities can be expected.

This plan may be the main positive impetus for empowering this type of business while preserving traditional material heritage, the fact that young people who would stay in the country will find this kind of activity close and appealing. Due to the modern way of communication, which is increasingly entering the rural area, making it very good for all residents. Modern way of communication opens the door to the furthest and underdeveloped rural areas, for people from all over the world. If this work has also made a small contribution to highlight this possibility - one small road has been made, a small step has been made, but the longest journeys begin with the first , even with a small step it's important to go ahead.

Acknowledgement

The paper was written within the project Optimization of architectural and urban planning and design in function of sustainable development in Serbia, (TR36042) funded by the Ministry of Education and Science, Republic of Serbia.

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CONTRASTING INTERPOLATIONS OF NEW ARCHITECTONIC FORMS

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Abstract: *In this paper, the questions of interpolation of new architectural forms in inherited urban matrices of cities have been brought about by drastically disturbed situations and serious aesthetic-artistic and historical changes in space. Examples in this work show contrasting interventions with a chaosized organic metabolic relationship between physical structures. A long-standing lack of design sensibility, vision and strategy is evident not only for urban heritage, but also for the natural environment, according to mistakenly shaped and reversed urban sites.*

It is a philosophy of a peculiar architectural and urbanistic expression related to the real, existing architecture and conceptualization of streets, settlements, squares and piazzas, the morphology of unisonized blocks of buildings. Here we read the drama of the city, the uniform ambient continent, the loss of identity in which globalization, imitating manners of construction bring users unattractive culture of ecourbarchitectural living space. With the same intensity in the interior and exterior environment, intrusive interpolated forms, unacceptable geometry of volume and facade levels, have been achieved, thus inheriting architectural physical structures significantly degraded and spiritually-memory devastated.

Key words: *interpolation, architectural form, contrasting structures, vision, strategy*

1. Introduction

There is a large number of newly constructed buildings in the cities of the world, which brought about new ecourbarchitectonic situations in micro-ambient environments and essentially changed their identity. They are mostly interpolated between the existing, adjacent buildings, and they created a new designing iconography without any respect towards the historical values of inherited structures, an improper conceptual and cultural-historical imbalance, damage in which new materials and contemporary technologies brutally disrupt the spirit of a place with their non-synthetic "fitting in". The violent interventions comprising building of new structures in space disrupted the harmony of appearance of the streets, squares and plazas, and of their planned design and created problematic "avant-garde" fusion without any designing and town planning logic and sensibility, by which this hybrid option inaugurated the philosophy of disrespect and devaluation of esthetic-historical-cultural places, old, inherited spatial physical structures. It is a global phenomenon. We can read strategic deviations on the newly constructed building, on extensions from the images of cities which do not have style or architectonic but only utilitarian values. They do not contain an idea of mutual coordination of adjacent buildings, nor plans for conservation and preservation in the original form. There are many examples of such lack of care of the patterns where there is no interaction of building

culture nor artistic components. With them, architecture is going towards a dramatic demise. This is not a good sign.

2. Research

This research shows 10 indicative examples from the world of ecourbarchitercture which confirm irregular, controversial, conflicting, polemical city-building and historical, culturally-(un)esthetical and non-artistic interventions in space.



Fig. 1. The new Musée d'arts in Nantes[1]
https://divisare.com/projects/346766-stanton-williams-hufton-crow-stefano-graziani-musee-d-arts-de-nantes
https://www.e-architect.co.uk/images/jpgs/france/grand_musee_d_art_nantes_s171209_1.jpg
https://blueverticalstudio.com/wp-content/uploads/2017/06/stanton-williams-musee-darts-de-nantes-art-m

At the competition for conceptual town planning and architectonic design of extension of the Musée des beaux-arts, in Nantes, France, in 2009, Stanton Williams won the first place. It is a very bold intervention in space, where the physical structure is extended with new square meters and new physiognomy which drastically differ from the buildings in the neighborhood, provoked various opinions of the user. The majority thinks that the form and frontage of the proposed and realized designing experiment is very specific, compromising, non-synthetic and conflicting in comparison to the inherited structure in micro-ambiance. An unexpected architecture occurred, erasing the inherited patina of the houses in the neighborhood. There are those users who think that this action is a beginning of creation of new cultural identity of the place, of high quality of the iconic character of the street, of different joining of the past and the present in remodeling of urban districts. The designer says that the dominant white stone façade generally opened new potential in the epoch of arrangement of public city areas.



[1] Year: 2017. Place: Nantes, France. Client: Ville de Nantes and Nantes Métropole. Value: 48,8 million euro.

Fig. 2. Conflict Exterior, Conflict Creation. In Ghent, Belgium. Clever Architecture Approach in Ghent, Belgium: House 12k[2]
https://freshome.com/2012/03/26/clever-architecture-approach-in-ghent-belgium-house-12k/
https://i.pinimg.com/736x/e7/82/4e/e7824e4ffd06276923cf18648196ae5f.jpg
http://images.adsttc.com/media/images/5005/ef5e/28ba/0d07/7900/23ba/large_jpg/stringio.jpg?1414436213
http://3.bp.blogspot.com/-NNruEtOXzsI/UGb-DCeNj1I/AAAAAAAAHII/NpXmXeamt0o/s1600/House+12k_6.jpg

The three-storey new house "Terraced House 12K" in the central area of the city of Ghent, was designed by the team from the designing Bureau Dierendonck Blancke Architecten[3], at the location between two large buildings with a small street façade front. A new physical ecourbarchitectonic structure was interpolated, which has a different esthetic-designing discontinued conceptualization of space in the contact zone, both in the exterior and interior, and particularly the street façade appearance which drastically interrupts the cultural narration of façade material in the street front. Flat concrete, pointed blocks in the street façade resulted in an unattractive and improper façade structure, created with a low investments and unacceptable esthetic criteria. In the process, the designer focused the attention on the functions of the newly designed housing building with central staircase and skylight which introduces daylight into the interior. What is lacking is organic, necessary and required continuous architectonic relation with the inherited buildings. This is a contrasting interpolated composition of small artistic value.



Fig. 3. The Black Ocean Firehouse, New York City
http://www.knstrct.com/interior-design-blog/2014/4/29/black-ocean-firehouse-new-york
https://www.pinterest.com/pin/777222848161888040/
http://nymag.com/homedesign/articles/2014/03/spaceoftheweek0306/
https://i.pinimg.com/600x315/a2/97/b6/a297b65bd7b9798

In New York, on the W 30th , there is the latest building with a "lowered roof" between two buildings, designed by the architect Rafael de Cardenas: the renovated firehouse „Black Ocean". The building of the Firehouse was converted back in 2014 with new functions. Now it is the headquarters of the digital media company, "Black Ocean". De Cardenas and his firm, „Architecture at Large", conceived "Black Ocean Firehouse" as a communal workspace, with a number of various scenarios. It contains private office space as well as large areas of useful space. Apart from the administrative-office premises, also planned for are the collective areas in the building, including the inner yard, roof terrace and several locations for rest and sitting.

Although the Neogothic street façade is depressing in terms of the coloration, with a prominent dark gray color and rich relief plastics. The structure dates back to 1890, and it is very imaginative, attractive. It is illuminated with strong fluorescent lighting. The walls

[2] Architects: Dierendonck Blancke Architecten. Location: Ghent, Belgium. Design Team: Alexander Dierendonck, Isabelle Blancke, Marie Decoene. Engineering: Arthur De Roover, Structure. Budget: 145,000 euro. Area: 108.0 sqm. Project Year: 2010
[3] Dierendonckblancke Architecten, Nederkouter 111, 9000 Ghent, Belgium

are decorated in dark color, in tones of earth and wood and light points. "Black Ocean Firehouse" is an attractive place, especially for people of action and those starting a business. Good arrangement of the rooms is an ideal place where those eccentric, creative minds can find additional inspiration. The former Firehouse, "Black Ocean" shows that skillful designing concept, conversion of functions, can rejuvenate old architectonic structure, which nowadays on Manhattan is an extremely agreeable place for the beginning of big dreams.



Fig. 4. Casa de ladrillos[4],Ventura Virzi Callegari Arquitectos, Buenos Aires
https://www.pinterest.com/pin/446771225507485007/
https://ensaiosfragmentados.com/2014/01/24/brick-house-ventura-virzi-arquitectos/
http://www.plataformaarquitectura.cl/cl/02-162388/casa-de-ladrillos-ventura-virzi-arquitectos

In case of the building Casa de ladrillos, the designer Ventura Virzi Callegari, in Buenos Aires, a new façade was made from orthogonal fields, which masks and conceals the existing house in the street. The accent is on the designing-visual play of the openings off various sizes which let the light into the interior. The interpolated façade also disrupts the continuity of the appearance of the street, and visually and architectonic pushes the buildings in the contact zone into the background. What is missing is ecourbarchitectonic artistic scenography, investigating metabolic-artistic implementing attributes with which potential would have a far higher expressive quality and emotional attractiveness for the users. Instead of the expected sensitivity for the spirit of the place and its materialization of the houses, the result is a grate-like, sensationally senseless screen, an utterly simplified illogical street backdrop which is not perceived as a work of art. What needed to be done, as Peter Zumthor[5] said "the forms and materials, sound of the space, temperature of the space, tension between the interior and exterior, degree of intimacy and light over structures should have been coordinated..." We think that a new, more quality micro-ambienc atmosphere of the street should have been implemented, a better human feel of space, based on the examinatino of the existing, inherited condition on the site.



[4] Architects: Ventura Virzi architects. Location: Buenos Aires, Argentina. Project team: Daniel Ventura, Andrés Virzi, Juan Pablo Callegari. Area: 90.0 m2. Year Project: 2011
[5] Peter Zumthor (born 26 April 1943) is a Swiss architect.

Fig. 5. Hardel + Bihan - Building a Household[6], Paris
https://www.pinterest.com/pin/573716440023257748/visual-search/?x=16&y=16&w=530&h=671
https://bienchezmoi.wgcdn.net/bienchezmoifly/surelevation-immeuble.jpg
https://www.amc-archi.com/photos/hardel-le-bihan-surelevation-d-une-habitation,537/terrasse-surelevation-d-un.2

Ecourbarchitectonic discord of buildings can be seen on the example of a Paris street The buildings designed to have different number of floors and different appearances of street facades show a designing-town planning and historical-artistic insensibility, where the roofs as a "fifth façade plane" represent a special curiosity. Interpolation of a cubical housing volume and the existing building dating back to 1950 and its materialization, disrupt the urban identity of the street area, while the adjacent buildings with their openings and façade decoration contribute to town planning-architectonic disorder and creation of an unrecognizable character of the place. Disintegration of urban design shows a brutal destruction of the existing multi-layered cultural heritage, erasing the memory of the city with conflicting city building interventions behind which are most often the private interests. The continuity of ecourbarchitectonic space does not rely on the past and accumulated historical creative images of urban matrixes, and it is a devastating fact.



Fig. 6. Built by James & Mau in Madrid[7], Spain with date 2012
https://www.pinterest.com/pin/10414642863070726/
http://www.archdaily.com/288571/san-vicente-ferrer-james-mau/5036580428ba0d4a38000069-san-vicente-ferrer-james-mau-photo
http://www.archdaily.com/288571/san-vicente-ferrer-james-mau/5036590f28ba0d482400002d-san-vicente-ferrer-james-mau-elevacion

Even though in the case of inherited urban entities it is regulate how new interpolated buildings and areas on the free land lots should be formed, this rule under the pressure of globalist "contemporary" tendencies in ecourbarchitecture is violated in many environments. The example from Madrid confirms abandoning of traditional defining of the volume of the house and street façade through new materials and new philosophy of space conceptualization. The street physiognomy with, in places, drastically changed appearances of the houses does not support the general interest about common created values, nor cares about preservation of the inherited condition. We are dealing with the collapse of observation of the tradition and with establishment of new forms of urban life. It is obvious that the concept of "contemporary" city building diversity brings about a new situation of defining of hybrid structures in the inherited public space where boundaries which were being created throughout centuries are easily cancelled.

[6] Technical specifications: Location: Paris 16e. Client: private Project manager: Hardel Le Bihan Architects Program: dwelling in raised area. Surface: 80 m². SHON terraces 32 m². Cost: 340 000 € Ex.
[7] Architects: James & Mau. Location: c/ San Vicente Ferrer 78, Madrid, España. **Nº** de viviendas: 4 de 50m² y 2 de 75m². Constructor: Arbohor Inversores S.L. Area: 504.0 sqm. Project Year: 2012



Fig. 7. 111-121 Westminster Bridge Road House, London
http://www.geograph.org.uk/photo/2638334
https://d24r6cdbdf33pd.cloudfront.net/wp-content/uploads/2016/02/08-9.jpg
http://www.atlasobscura.com/places/london-necropolis-railway-station

An example of the administrative-office building from London, built in 1970's indicates a non-integral approach to urban design and ecourbarchitectonic excess in which the historically important structures are pushed to the background. New materialization degraded the adjacent house, and the others, with the monumental façade - Westminster Bridge, built for the company "London Necropolis", in 1900 of granite, bricks and terra cotta. The building was intended for the railway authorities, near the Brookwood cemetery, near Woking, until 1941.

One of the interviewed citizens said: "it is so frustrating for the denizens to see that stupid architects and companies place ugly, mediocre buildings between the beautiful and old ones. We very much hate to see an interpolated, unattractive façade, and hear that they observed the setting. No, you didn't! Any city has got new city blocks. You can place your ugly concrete boxes into those blocks, and not in the dear old neighborhoods!" It is obvious that the new building deviated from observing at least three levels of building culture in this situation.



Fig. 8. Mixing old and new architecture. Buildings in Madrid that were obviously built at very different periods. Note also the more modern buildings in the distant background. The next several pictures were taken in Barcelona and illustrate similar situations. Insert from New York.
https://provocationutah.files.wordpress.com/2012/09/dscn7141.jpg
https://provocationutah.wordpress.com/2012/09/04/mixing-old-and-new-architecture/
http://archleague.org/main/wp-content/uploads/2015/04/DBPA_48BondS_CT704_e_dusk_resized.jpg

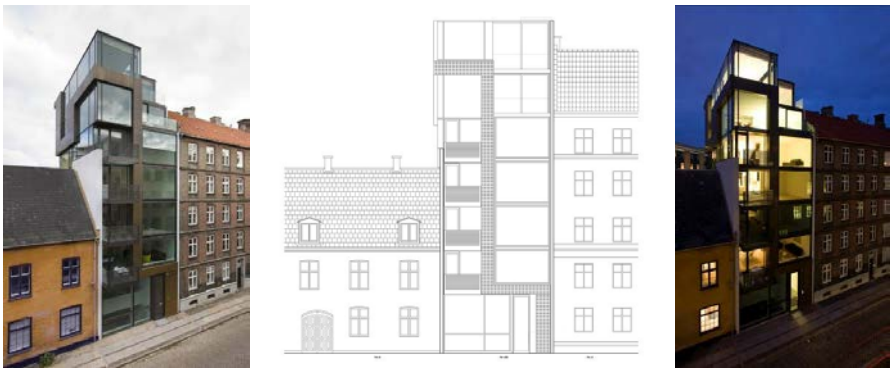
Interpolation of new mixed-use buildings in central areas of large cities also caused displeasure of their citizens. The examples from Madrid, Barcelona, New York and other metropolis confirm that failing to observe historicity of a locality and its buildings, especially in the contact zone, always generate an unacceptable esthetic-designing and ecourbarchitectonic mixture, where certain environments almost do not differ at all. The globalist pattern for newly designed physical structures is almost a carbon-copy. Genius loci disappears, between the traditional cultures and new forms. The building models are uniform, with decreasing differences. They effectively eliminate idiosyncrasy of the location, introduce quick changes and negate importance of the place, neglect the existing

local cultural identity, create new conceptual strategic urban design. New criteria are established, a new ideology for professional action in space. Historical, modern and postmodern ecourbarchitectonic structures come together in an obviously inappropriate "compromise".



Fig. 9. Moritzburg Museum Extension[8]. Images of an extension to a museum inside a ruined castle in Halle, Germany, by Spanish studio Nieto Sobejano Arquitectos.
http://c1038.r38.cf3.rackcdn.com/group4/building39377/media/qfbz_1.jpg http://2.bp.blogspot.com/-kFGxbtLG3u4/UFAKt7kGd9I/AAAAAAAABAs/QiCqbDnNirM/s1600/ExternalWallsC3+article.jpg
http://c1038.r38.cf3.rackcdn.com/group4/building39377/media/wfsj_151005.jpg

The old castle Moritzburg in the city of Halle, in Germany is a very valuable example of gothic military architecture typical for the end of 15th century. Even though it is very old, this physical structure retained all the main architectonic characteristics: surrounding wall, three of four round towers at the corners and an inner yard. The design by Nieto Sobejano Arquitectos for extension of the building, to obtain the function of an art museum is based on an original architectonic concept. It includes a new roof, conceived as a large folding platform, which can be raised and lowered to let through natural light for the new exhibition space. The result of this operation is a complete freeing of the bottom of the ancient ruin, which resulted in a single space providing for a different exhibition potential. This design is complemented with construction of two new vertical communication cores. The first is located in the northern wing which connects the levels which must have a mutual communication. The second is a new, modern tower, 25 meters high, on a location of a former bastion, which provides access to new exhibition spaces with their distant view of the city. The angular geometry of the new appearance of the roof and the metal tower is in contrast with the existing irregular form and tall roof of the castle, which creates an impression of inconvenient forms of expression.



[8] Architects: Nieto Sobejano Arquitectos. Location: Halle, Germany. Architect: Nieto Sobejano Arquitectos, S.L.P. Project Team: Fuensanta Nieto, Enrique Sobejano, Sebastian Sasse (Project Architect), Udo Brunner, Nina Nolting, Dirk Landt, Susann Euen, Siverin Arndt. Competition Collaborators: Vanesa Manrique, Nina Nolting, Olaf Syrbe, Miguel Ubarrechena. Site Supervision: Nieto Sobejano Arquitectos S.L.P., Fuensanta Nieto, Enrique Sobejano, Sebastian Sasse, Johannes Stumpf, Karl Heinz Bosse. Structural Engineer: GSE, Jorg Enseleit. Mep Engineer: Rentschler y Riedesser, Jürgen Trautwein. Models: Juan de Dios Hernández-Jesús Rey. Roof Construction Company: Dornhöfer GmbH. Project Year: 2008.

Fig. 10. Stokhusgade, Copenhagen[9], DK
https://www.pinterest.com/pin/308144799494565016/
http://images.adsttc.com/media/images/5170/97a4/b3fc/4b20/1400/0006/large_jpg/Stokhusgade_07.jpg?1417005660
http://images.adsttc.com/media/images/5170/99c0/b3fc/4b9b/ac00/0009/large_jpg/facade.jpg?1417005691

In the center of Copenhagen, in Stokhusgade street, nearby Nyboder, Holscher Arkitekter designed and constructed the housing building on a very narrow location, with a façade front of seven meters. The interpolated structure is unusual in its form, in the individual details and relationship towards the neighboring buildings in the street. From ground level to the fourth floor, the building is positioned to face north and south. From 4th to 7th floor, the building is oriented towards west and Østre Anlæg park. The façade band is polygonal. It starts from the street side and ends on top towards the courtyard. The attempt was, as claimed by the designers: "to achieve historical urban context". In contrast to that, the building has an archisculptural character and it is interpolated so that it disrupts with its size the historical harmony and designing-architectonic narration of adjacent buildings as well as vertical composition in the street contour. It is particularly disrespectful towards the structures in the contact zone from years 1862 and 1878. Even though some critics say that the building architecture is "too neutral, generic and smooth without identity or agenda… excessively self-centered and flashy", it definitively neglects the coherence of old buildings and cannot be adapted to the conditions and cannot create quality in its environment.

3. Conclusion

Disrespect of ecourbarchitectonic heritage is evident in many city environments in the world, where interpolation of new structures reduced value of the space, but also where dramatic changes were brought about and where new spirit of the place and inadequate identity were established. The existing adjacent physical structures lost their recognizable, historical, cultural authentic expression through contrasting and complex designing "integrations". In the presented examples a new geometry of physical structures was accentuated, with opening on the façade planes which largely reinforce the non-artistic dimension and diversity of architectonic forms. There are many failed designing, experimental interventions ins pace which was not previously well analyzed, so transformations in micro-ambiance environments often have a degrading character disrupting spatial urban-planning development, with new vistas and materials, banal markings and advertisements. One has an impression that such uninventive, odd and hybrid logic is globalist, and little is done to stop it. As if the rules of observing organic ties of buildings in space, and unavoidable, sensitive metabolism of integration of new and old, inherited structures which need protection are forgotten. New houses in city cores are not sufficiently controlled, and have now quality, uniform sensible architectonic esthetics for the considerable and often impressive historical heritage.

If we would like to single out only ten successful examples of new ecourbarchitectonic structures interpolation in the existing, inherited fabric of the cities in the near past, we would have difficulty to find such continuous, metabolic dialogues, synthesis of the old new, quality designing-cultural narration. Instead of it, we find uncoordinated architectonic interventions without style, typology and quality which emphasize tensions in dramatic flows of cities.

[9] Architects: Holscher Arkitekter. Location: Copenhagen, Denmark. Co-Workers: Nils Holscher partner, Claus Sivager partner, Mikkel Nordberg part-ner, Philip Pedersen sagsarkitekt, Maria-Rose Guldbrandsen stud, Thomas Bossel stud Engineer, Erik K. Jørgensen. Landscaper: Holscher Arkitekter as. Area: 580.0 sqm. Project Year: 2008

We endorse a more selective, careful approach and a new strategy of preservation of inherited ecourbarchitectonic structures, a better cooperation of architects and conservators whose action can stop the haphazard and conflicting building interventions in urban space.

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THE CONTEMPORARY VALUE OF THE ANCIENT LUNULAS. ARE THEY JEWELS OR AMULETS, AND ARE THEY OBJECT OF INTEREST FOR THE TREASURE HUNTERS

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Abstract: *The article discusses several movable cultural assets from the National Historical Museum Fund, acquired in 2012, which are part of a find that has been seized by the law enforcement authorities. Besides the topic of illegal treasure hunting and the way in which it affects the tourism and economy in Bulgaria, the article presents some controversies in the scientific circles, which objects are amulets and which are non-volatile jewellery. In this article I look at twelve lunar-shaped objects. They represent pendants and earrings of silver and bronze. The cult and religious character of this kind of objects is undoubtedly proven by the information that we receive from ancient authors, historical and archaeological research. But whether it still matters after having been barbaric taken out of their original context without the necessary archaeological documentation. Do we have the opportunity to preserve the national and world cultural heritage on the territory of the Republic of Bulgaria and what does it cost us its destruction.*

Key words: *amulets, jewellery, lunulas, cultural heritage, treasure hunters*

1. Introduction

Though the theme of treasure has tended to slow down in the last ten years, this problem has not been eliminated and has not been adequately explained to the broader audience. The illegal discovery of ancient objects is not only a source of funds for the whole nation. Through unregulated digging, cultural monuments are destroyed - both movable and immovable. An even greater loss is that the information is being destroyed, resulting in a series of problems and losses, again to the state, to its economy and to all of its citizens.

1. Description and identification of the artefacts

The description and identification of unregulated artefacts is hampered by the lack of information on how, when and where the object was discovered. This mandatory in archaeology information increases several times the possibility of determining the type and purpose of archaeological values. In many cases, when there is no history of discovery, the subject is measured by the material from which it was made, or compared

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with similar objects found under better circumstances. Very often, however, disputes arise over identification. As an illustration of what has been said so far, I will present a group of articles that are unpublished and which are in the National Historical Museum fund. They are found in the region of Stara Zagora, but unfortunately, the context and their exact location are not clear. They are twelve in number and they are of type crescent-shaped *lunula* intended for wearing as pendant or earrings. Their treasure hunting origin does not allow for direct interpretation, and in order to resolve the dispute over whether these items are amulets or ordinary ornaments, they have to be compared with our known sources.

The first amulet, with a height of 2.3 cm and a width of 2 cm, is a bronze-shaped plate with a polygonal cross-section. The ends end with small buds. It is designed to be worn as an embroidered ornament. The trapezoidal sockets are formed on the surface with yellowish and dark blue enamel. The *lunula* is in a relatively good condition, there are traces of corrosion only on the ring. The date of this item is II-III c, [1].



Fig.1. Bronze mural with enamel - pendant

The second amulet is a bronze crescent-shaped plate with a width of 3.4 cm. Its edges are open. It is decorated with groups of transverse relief lines alternating with embossed hemispheres. A tubular suspension sleeve is formed at the top. The subject is relatively well preserved with mild corrosion, in some places the plate is visible. The date is III c, [2].



Fig.2. Bronze *Lunula*, Pendant

The next item from the NIM Fund is also in the shape of a thin *lunula*. Its cross section is D-shaped. The diameter is 3 cm and the height is 3.6 cm. The edges end with a conical thickening. In the upper part there is a wide plate, which is bended tubularly, forming a cylindrical suspension sleeve. The material from which the *lunula* is made is silver. The item is in very good condition. The date is II-III c, [3].



Fig.3. Silver *Lunula*, pendant

The next subject is an amulet, a silver-shaped, mullion-shaped plate with a polygonal section. Dimensions are in diameter - 2.2 cm and height - 2.9 cm. The edges end with a bulging thickening. At the upper end is a tubular ring decorated with embossed ribs. The date is II-III c, [4].



Fig.4. Silver *Lunula*, Pendant

Another crescent-shape *lunula* is bronze, thin and almost closed. Its cross section is rectangular. Its dimensions are 1.7 cm in diameter and 2.5 cm high. The edges are thin. The upper wide part is a cast pyramid pin for attachment. The *lunula* is well preserved, but in places there are heterogeneous batts. This type of *lunula* occurs in the 2nd-3rd century AD, but they also occur in the 4th century. An example of this is a group of crescent-shapes found in Beroe's necropolis. One *lunula* was discovered in a

grave along with a perforated coin of Constantine II (314-340), dating back to the IV century. Its texture is coarser[1]. The other two copies logically have the same date. They are more finely crafted, with more pointed edges, [5].



Fig.5. Bronze *Lunula*, Pendant

Again, there is a silver crescent-shaped amulet, with oval-shaped and thin, open edges. The diameter is 1.3 cm and the height 1.5 cm. In the upper part is mounted a tubular suspension ring, decorated with two transverse incision lines, which form embossed rings at the edges. The frontal surface of the *lunula* is decorated with granules. It is dating II-III c, [6]. Similar in shape and decoration, but golden *lunulas*, were found in the village of Lisets, Lovech; Necropolis of Chatalka; Tlachene village, Byala Slatina; Gorni Dubnik, Pleven district; Mezdra, Vratsa district. [2]

Apart from pendants, crescent-shaped amulets are also used as earrings. In the presented collection there are six such objects. One of them is a *lunula* with a diameter of 2.5 cm and a polygonal section. Edges are almost closed, thin, ending with a spherical ball with a bud on top. The earring is silver. Its condition is good, in some places the surface is disturbed and there is no bud on the ball. The date is II-III c, [7]



Fig.6. Silver *Lunula*, pendant



Fig.7. Silver *lunula* - Earring

[1] Thanks for the information and materials provided by Dr. Boyan Dumanov. Dumanov, B. - "Late Antique Jewry in the Dioceses of Thrace and Dacia (4-7 c.)". Dissertation for awarding educational and scientific degree "PhD". Sofia, 2003

[2] Ruseva-Slokoska L. - Roman Jewellery. Academic Publishing House "Prof. Marin Drinov" Sofia 1991 - 147 - 148

Two silver earrings that are almost the same with a diameter of 1.9 cm. One has a polygonal section. The edges are thin and open, [8]. The other one is oval. Its edges are again open and thin. Their condition is relatively good, the edges are deformed and one of them has a missing fragment. In some places there is a surface injury. The date of both is II-III c. (Rev. 59203),[9].



Fig.8. Silver Lunula – Earring



Fig.9. Silver *lunula* - Earring

Another silver earring from the NIM Fund is typical of the group of amulets earrings. It represents an oval section *lunula*. Its diameter is 1.7 cm. Its edges are open and thin. The face part is decorated with granules. The condition in which it is found is good, some of the granules are broken, and the edges slightly deformed. The date is II-III c, [10]. Other silver earring with *lunula* shape is of oval cross section. Its diameter is 1.5 cm. The edges are thin and open. At the top end with nodular thickening. A granule is preserved on the front side in the widest part. It is dating II-III c, [11]. Last earring of the presented collection is silver *lunula* with oval cross section. The diameter is 1.4 cm. The edges are thinned, almost closed and end with a conical buds. The subject is in very good condition. The date is II-III c, [12].



Fig.10. Silver *Lunula* – Earring



Fig.11. Silver *Lunula* - Earring

2. Examples and description of *Lunula* objects from other sites

Crescent-shaped objects are one of the oldest and most common amulets in ancient times. They have a long history in the Middle East. The earliest type found in West Asia is dated back to 1600 BC. In the second millennium in Egypt, this type of amulets is rare, although there is a specimen found in el-Amarna.[3]



Fig.12. Silver *Lunula* – Earring

The *Lunula* may be found throughout the whole Mediterranean world.[4] The most common late Hellenistic crescent is made of sheet metal covered with garnet. This species is well known from Greece, Asia Minor and Syria, and survived the Roman era.[5] This type of amulets is believed to be protective, protecting newborn babies as well as their mothers. This understanding comes from the moon as a symbol of the female cycle and pregnancy.[6]

During the Antiquity, masks and coffins were created with images of crescent pendants. They are carried by women - never by men. Numerous are the Fayum portraits on which are depicted their owners, carrying on their necks *lunula*. They are usually hanging on gold chains, but on some early Egyptian masks and wooden sarcophagi the pendants are attached to long black and white strings. Crescent pendants are also one of the commonest ornaments mentioned in Romano-Egyptian dowry lists and might have belonged to the bride since early childhood. In Plautus' Epidicus, a slave says to the girl Philippa: "*don't you remember my bringing you a little gold crescent on your birthday"*.[7] In the fourth century AD Gregory Nazianzenus describes "moon-shaped plates of gold, silver or cheaper materials which foolish old women fasten upon infants". In Plautus' play Rudens, Palaestra refers to "a gold amulet that my father gave me the day I was born" but the type of amulet is not specified.[8] Isidore of Seville gives a clear definition: "The *lunulae* are ornaments for woman; they are small pendant gold

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[7] Plautus. Epidicus. 5.1.632.
[8] Ogden J. M. .. Opus. cit., p.. 213- 215

balls shaped like the moon" (luna).[9] On the eve of marriage, the *lunula* is removed and burnt with the other kid's toys. There is much speculation about what it means, but many say it is the beginning of a new phase of life.[10]

Lunula-like finds have been found on the Mediterranean since the Hellenistic era. They were attached to different types of necklaces or as a single ornament. They have been testified on many palm monuments of the II-III century.[11] From the Balkan provinces of the Roman Empire are known specimens of typical lunar form from the village of Tlachene near Byala Slatina, Gorni Dabnik, Mezdra, Chirpansko, Treasure of Nikolaevo, Lisets, Lovech, Kutina, Yagodin Mala (Nis), Nisus, Margu, Guberevets, Lüleburgaz and others.[12]

Lunulas are a strong amulet with a long history and wide territorial reach. The lower limit of their wearing can be determined IV century BC[13] on the territory of Mesopotamia. The use of this type of amulets was greatest during the Roman imperial period, as evidenced by the vast amount of *lunulas* found in all the provinces of the Empire. At the end of the 4th century and with the strengthening of Christianity, a ban was imposed on the wearing of objects of apotropic character. As the upper limit for the wearing of *lunulas*, we can assume the end of the 4th century, though they also occur as a type of ornamentation without semantic value.

3. Conclusion

Finally, commenting on the issue of treasure hunting, I did not accidentally pick these seemingly small and popular ornaments. If they were discovered in regular, scientifically conducted archaeological studies, we would be able to get closer to the age they represent. This would increase both the information they gained and their value. Now the loss is for all of us. It is compensated by carrying out a scientific analysis, somewhat. But evil is done and some of the information is forever lost.

Acknowledgement

Sincere thanks to Lyubava Georgieva and Elka Penkova for the materials provided in order to make this publication. Huge thanks from Assoc. Dr. Nikolay Markov for the help and the provided literature on the subject. Thanks for the information and materials provided by Dr. Boyan Dumanov

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ВЪЗМОЖНОСТИ ЗА ОБНОВЯВАНЕ НА ЦЕНТРАЛНАТА ГРАДСКА ЧАСТ НА ГР. КЮСТЕНДИЛ С КУЛТУРНО- ИСТОРИЧЕСКА НАСОКА. ЕКСПОНИРАНЕ НА АРХЕОЛОГИЯТА ОТ ДРЕВНА ПАУТАЛИЯ-АРХЕОЛОГИЧЕСКИ МАРШРУТ.

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POSSIBILITIES FOR RENOVATIONS FOR THE CENTRAL URBAN PART OF THE CITY OF KYUSTENDIL WITH CULTURAL AND HISTROCAL SENSE. EXHIBITION OF THE ARCHELOGY FORM ANCINET PAUTALIA-ARCHEOLOGICAL ROUTE.

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***Abstract**: This report is about the possible design solutions for the exhibition of ancient archaeological monuments from the central part of Kyustendil by analyzing possible development routes with a view to creating new and enriching existing public spaces. Suggested solutions for realization are proposed with a view to renovating the central city parts in terms of the cultural and historical heritage.*

***Key words**: Exhibition, archaeological monuments, central city part, Kyustendil, development routes, public spaces, sample solutions.*

1. Въведение

Кюстендил има осемхилядолетна история и повече от 1900-годишна градска традиция. Известен е с имената Пауталия, Улпия Пауталия, Пауталия Аврелий, Велбъжд, Константинова баня, Ълъджа, Баня, Коласия, Кюстендил.[1] През V- IV век пр. н. е. траките основават селище. Под римска власт (45 г. от н.е.), римляните превръщат населеното място във важен търговски център и известен балнеологичен курорт, който наричат Пауталия (на лат. Pautalia). С административните реформи на император Траян (98 – 117г.) е потвърдена градската уредба на Пауталия, с което започва цялостна реконструкция по римски образец(Фиг.1а,б).

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а)



б)

Фиг.1а,б. Ортогонална система на древна Пауталия; а) национален културен и археологически резерват Пауталия - Велбъжд-Кюстендил, б) (карта на археологията от историческия музей на град Кюстендил)

2. Римски период от културно археологически забележителности - Пауталийски асклепион

Кюстендил е бил е втори по големина на Балканския полуостров след този в Епидавър - Гърция[3]. В римските терми имало аподитериум (съблекалня), тепидариум (умерено топла зала), калдариум (гореща зала), фригидариум (студена зала), лаконикум (пералня), палестра (гимнастическа зала). Отоплението на баните било по типично римски модел - с хипокауст, който затопля пода, после стените и цялото помещение(Фиг.2а,б,в).



а)



б)



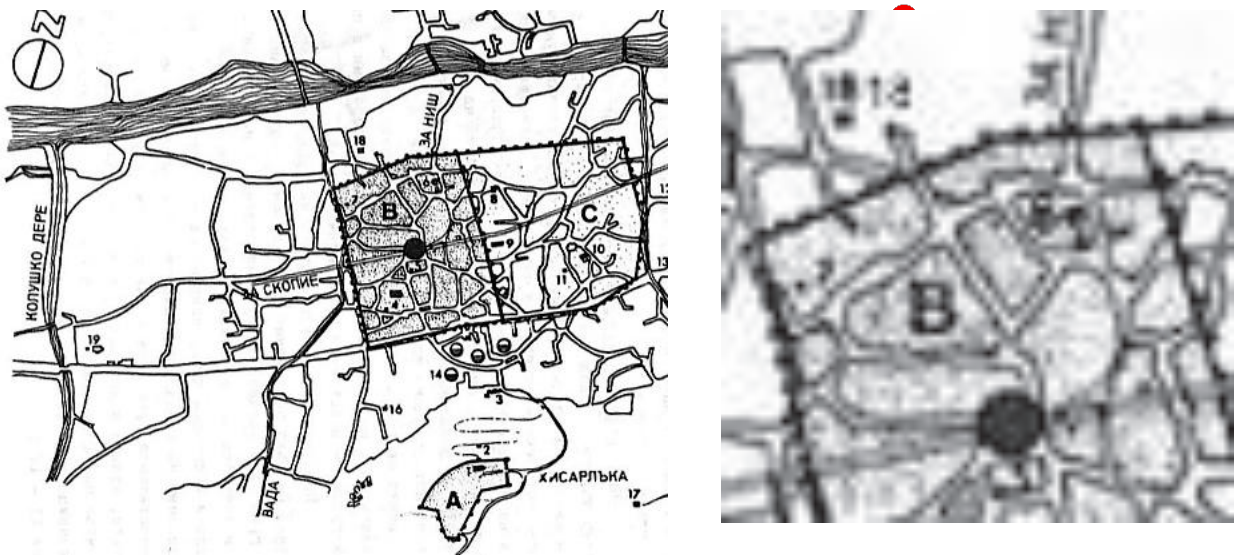
в)

Фиг.2а,б,в. Пауталийски асклепион: а) монета, , от която се съди за архитектурата на асклепиона. б), в) разкрити римски термы в Пауталийския асклепион**;** [3]

3. Архитектурни забележителности- базилики на раннохристиянска Пауталия.

През втората половина на IV в., Пауталия е и средище на епископия. Започват да се строят нови раннохристиянски църкви, някои от които върху по-ранни езически храмове. Регистрирани са в района на днешния град седем трикорабни базилики: две при северната порта, построени една върху друга; други две базилики извън крепостната стена (изградени също една върху друга); една базилика северно от форума; една сред античния некропол и една базилика в крепостта "Хисарлъка"(по дадената карта на фиг. 3)[4]. Част от мозайките са реставрирани и преместени в Историческия музей на Кюстендил.

Двете ранновизантийски базилики извън крепостта на Пауталия се намират близо до северната стена (по дадената карта на фиг. 3а,б. позиция номер 18).Те също са разположени една върху друга.[4]



а) б)

Фиг.3. Схема на средновековния и възрожденски Кюстендил с местоположението на християнските паметници (по арх. Ю. Фърков)**;** [4]

Базиликата северно от форума на античния град е разкрита е през 80-те години на XX в. В базиликата в югозападния некропол на Пауталия са разкрити източните части на трикорабна църква с голяма полукръгла апсида, снабдена със синтрон. Намерени са останки от мозайка в подовете на олтара и на наоса(фиг.4а,б,в,г).



а) б) в) г)

Фиг.4а,б,в,г. Снимки от частично разкритата раннохристианска базилика намираща се под сградата на читалище "Братство" (бившия младежки дом); [3]

4. Османски период - Джамията "Фатих Султан Мехмед"

Джамията "Фатих Султан Мехмед" в Кюстендил (известна още като "Шалдърванската джамия") е разположена в централната част на града находяща се на бул.“Цар Освободител“. Построена е през 1531 год. Джамията е изградена от дялани каменни блокчета и тухли. Минарето се издига на около 30 м височина - осемстенно призматично тяло(фиг.5а,б,в). Джамията не е действаща и е архитектурно-строителен паметник на културата с категория национално значение.[5]



а) б) в)

Фиг.5.а,б, в) Снимки на сегашното състояние на джамията "Фатих Султан Мехмед" в Кюстендил; [6]

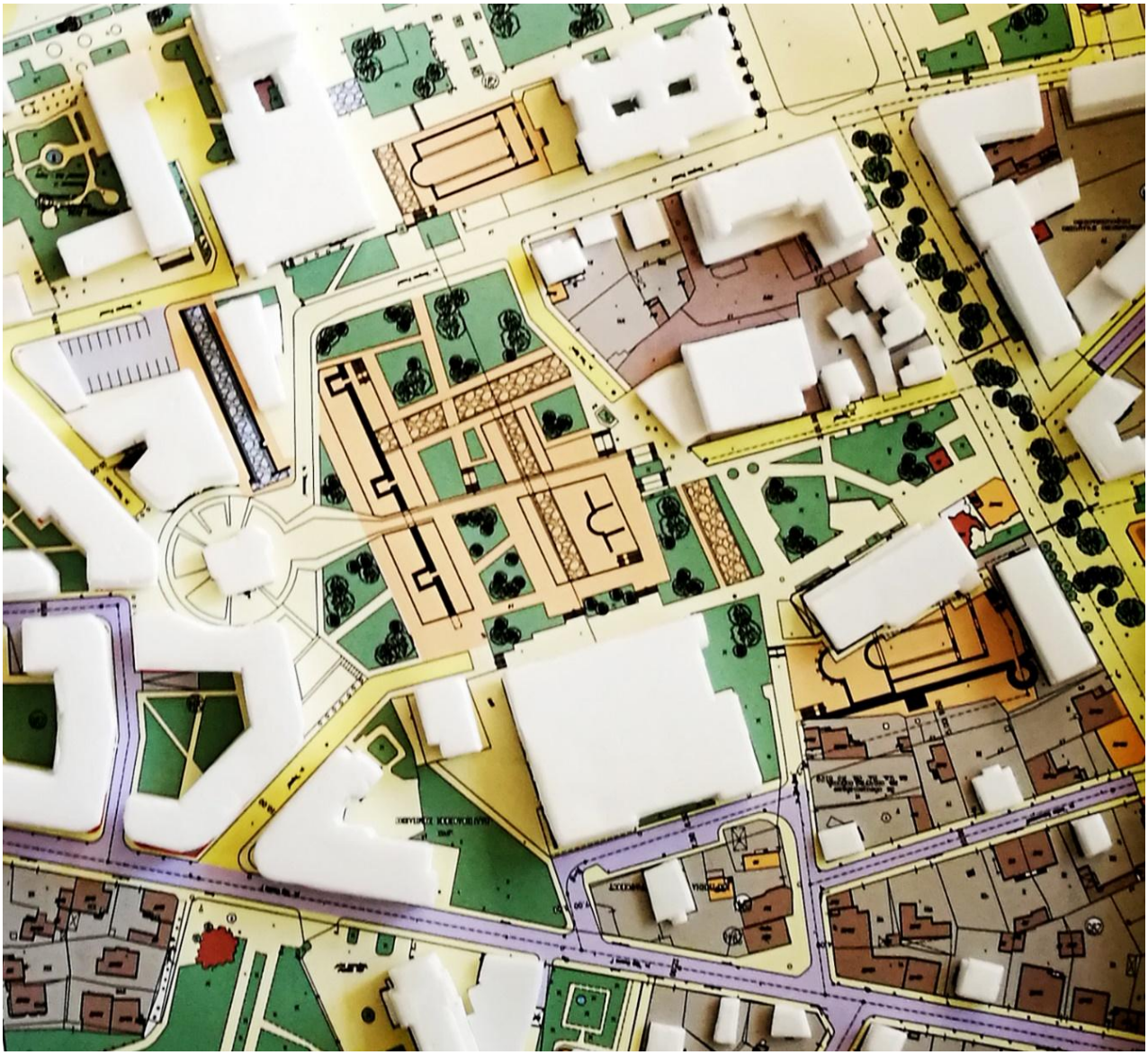
5. Изводи

Представените изследвания с периодизация на културно историческите паметници, ясно фрагментират нови устройствени аспекти при обновяването на ЦГЧ на град Кюстендил. Разглежданите три периода създават възможности за цялостното третиране на паметниците (консервация, реставрация и експониране), както и свързването им в културно- исторически маршрути. Представени са проектопредложения разработени в катедра „Градоустройство, теория и история на архитектурата“ – ВСУ „Любен Каравелов“, Архитектурен факултет(фиг.6,7,8,9,10).



Фиг.6. Обемно проучване с възможности за експониране на археологическите паметници от древна Пауталия, непосредствено до галерията на „Владимир Димитров Майстора“

Проучването е извършено с ръководител доц. д-р арх. Д. Власарев. Решението включва и обособяване на археологически маршрут в зелената зона пред галерията както и пространството пред „Дервиш баня“(фиг.7).



Фиг.7. Обособяване на археологически маршрут в зелената зона пред галерията както и пространството пред „Дервиш баня“

Извършено е обемно проучване с ръководител доц. д-р арх. Д. Власарев на кръгообразното пространство около Дервиш баня с възможности за експониране на част от крепостната стена на древна Пауталия и раннохристиянските базилики(фиг.7 и 8).



Фиг.8. Пример за посочване на състоянието на археологически разкопки непосредствено до западната фасада на галерията на „Майстора“, с оглед обновяването на територията по посочените обемно-устройствени проучвания.

Извършено е устройствено проучване с ръководител доц. д-р арх. Д. Власарев на джамията "Фатих Султан Мехмед" в Кюстендил (известна още като "Шалдърванската джамия"), с устройствена конверсия на бул.“Цар Освободител“ (фиг.9а,б).



а)



б)

Фиг.9а,б. Джамия"Фатих Султан Мехмед"("Шалдърванската джамия") в Кюстендил, -а), устройствена конверсия на бул.“Цар Освободител“ –б)

Предвидена е и разработена нова пешеходна зона с търговска и рекреативна функция в гр. Кюстендил(фиг.10).



Фиг.10. Нова пешеходна зона с търговска и рекреативна функция

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[2] http://poseti.guide-bulgaria.com/
[3] https://www.google.bg/
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НАПРАВЛЕНИЕ 2

Документиране, интерпретиране, презентиране и публикуване на културно-историческото наследство; Увреждане на недвижимите културни ценности в обкръжаващата го околна среда; Предпазване от промени във вътрешния климат и ефекти от туристическата инвазия; Биодеградация на културно-историческото наследство.

TOPIC 2

Documentation, interpretation, presentation and publication of culture heritage; Degradation of culture heritage in surrounding environment; Indoor climate and tourism effects; Biodegradation of culture heritage.

BRIDGES IN THE PREANTIQUITY WORLD

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***Abstract**: This paper reviews topics, related to the inception and development of bridge construction in the ancient pre-antique times of the Near East, North Africa and Europe. Emphasis is put on the importance of the early civilizations of Mesopotamia, and Assyria in particular, to that development. The following work includes analysis of the cyclopean masonry stone bridge structures from Mesopotamia, Minoan and Mycenaean civilizations, Hittite empire, as well as their later translation to the Apennine peninsula, where they are at the basis of the construction practices of the Etruscan.*
In-depth analysis of the different types of bridge structures formed in Anatolia follows - stone-wood, stone with stone slab pavement and arched stone, with corbel arches and later semi-circular and other types of arches. Based on the latter two, analytic overview of the formation and development of the arch is made, pointing the Mesopotamian region as its birthplace, in context of the inception of the Assyrian culture.
Analytic overview, based on particular examples, of the appearance of several bridge structural elements follows – breakwater, shuttering orifice, cantilever and impost at a vault's heel, trapezoid structural elements for vaults, foundations with giant squared stone blocks, linked via metal fastenings, immersed in lead, pilaster reinforcing of bridge superstructure, guard tower-gates on one or two sides of the road, etc., which challenges the views of their roman origins.

***Key words**: pre-antique bridges, ancient architecture, Assyrian, Hittite, Minoan, Mycenean, Etruscan, Persian, Egyptian*

As early as the Iron Age, if not earlier (ar. 1300 – 1100 BC), the Assyrian kings begin road construction on a large scale in Asia Minor, particularly in the mountainous areas. Among the ruins of the well-developed cities of Ur and Uruk (ar. 2000 BC) there can be found preserved road sections covered in cobblestone. The technology, however, is much older as it was applied in the most ancient settlements of Mesopotamia around 6100 - 5400 BC. In any case, a number of Assyrian reliefs from IX – VIII c. BC show cobblestone roads, which means that this was the most widely spread type of flooring at the time. During VIII – VII c. BC in Nineveh the carriageways are already covered in large, thick, properly-formed stone slabs on top of an underlying gravel layer. In the region of Mesopotamia at the time the routes are, in some cases, covered with bricks fixed with

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asphalt on a base layer of stone slabs fixed with asphalt too. This leads to the emergence of the durable multilayered road profiles, which are to be further developed in Rome.







Assyrian road. Loot from Lachis being carried, 701 B.C. Stone relief.
Assyrian road. King Shalmaneser travels. Bronze relief from Balavat.
Cobblestone road to Beth Saida in the river Jordan's delta, IX century B.C.

The Assyrian imperial road system of II millennium BC naturally came to include the construction of bridges as well. In those early times, however, many of those are pontoon ones – a practice which continued into the following Neoassyrian period (911 – 608 BC) as evident in certain texts of Herodotus. Certain Assyrian texts mention such bridges but it becomes clear they are private property. [1]

The bridges in Asia Minor from II – I millennium BC are, in most cases, stone-and-wood ones (or brick-and-wood, later with stone-lined piers), i.e. composed from stone or brick piers and a wooden deck lying on top of them. One of the most famous is the Babylon one, built by Nabopolassar I (626 – 605 BC) as evidenced in an inscription by his son Nebuchadnezzar II. At the time Babylon is capital of the Neoassyrian empire. In the second part of his geographical description, Diodor of Sisily attributes its construction to

Queen Semiramis - that means IX c. BC instead of VII c. BC. His claim is based on an old legend, which in its turn suggests the presence of a previous, more ancient bridge.

The bridge of Nabopolasar I, has got stone abutments, on top of which, according to Herodotus's description (ar. 484 BC), lies wooden flooring composed of spanning modules. Evidence can also be found in the hagiography of prophet Ezekiel, who was taken into the city by Nebuchadnezzar III (ar. 599 – 588 BC) together with other Jews. According to him, the bridge spans the Hovar river (i.e. Euphrates) crossing the city centre, and connects the king's palace on the eastern coast with, what is above all, a temple zone on the other side.

Herodotus, in his *History*, says that the construction was the deed of Nitocris – a name behind which, according to historians, stays Nabopolasar I. However, the description contains many technical details related to the construction and the name of Nitocris, which is why I think this is the name of the architect who built canals and a huge water reservoir before he constructed the bridge. And when the diverted river began filling it so that its bed dried up – Herodotus writes, he began a new project in it "in the first place lined the banks of the stream within the city with quays of burnt brick, and also bricked the landing-places opposite the river-gates, after which, with the materials which had been prepared, he built, as near the middle of the town as possible, a stone bridge, the blocks whereof were bound together with iron and lead. In the daytime square wooden platforms were laid along from pier to pier, on which the inhabitants crossed the stream; but at night they were withdrawn, to prevent people passing from side to side in the dark to commit robberies."

After the finalization of the construction, the author concludes, the river "was turned back again into its ancient bed". [2]

The historical excavation of the bridge took place as early as 1920. It is 116 m long (according to Diodor – ar. 700 m, which turns out to be a strong exaggeration), with 7 piers and two abutments. The foundation was made of dressed stone, connected by means of metal block ties filled with lead and built upon vertically with brick masonry, most probably protected with a stone-slab coating – as per the contemporary technology. The river piers measure 8,53 m in width and 19,81 m in length, this including the elongated, rounded off break-waters. According to Diodor, the deck is around 9 m wide made from palm log modules on top of a wooden cedar-and-cypress board (i.e. two layers in a row). Herodotus text makes it clear that the bridge has tower-gates on both sides.

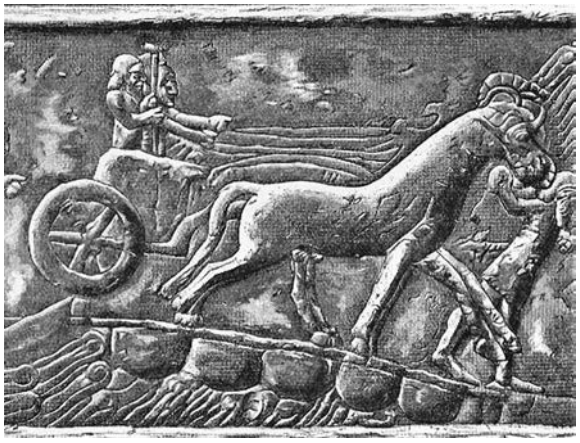




The bridge of Babylon. Reconstruction.
The bridge of Babylon – foundation. Photo by Wetzel.

Upon the excavation of the Neoassyrian city of Imgur-Enlil (present-day Balawat), founded by king Ashurnasirpal, there were discovered bronze strips from the city gates binding with relief images of the Assyrian kings' deeds. The main figure is that of Shalmaneser III (858 – 824 BC). Some of the relief images show bridge construction work. One such illustrates the construction of a bridge under the guidance of a war architect – the bridge has dressed stone abutments and wooden flooring; another shows the king passing over the bridge in a war chariot, the third – the army following him. The bridges are illustrated without protective handrails and the stone abutments have rounded-off break-waters, i.e. as with the Babylon bridge. Therefore, the squared stone blocks abutments and the break-waters are an invention much older than the bridge in Babylon. It is namely their Assyrian rounded-off shapes, most of which are found in Asia Minor, that are the prototype of the Roman ones centuries later.





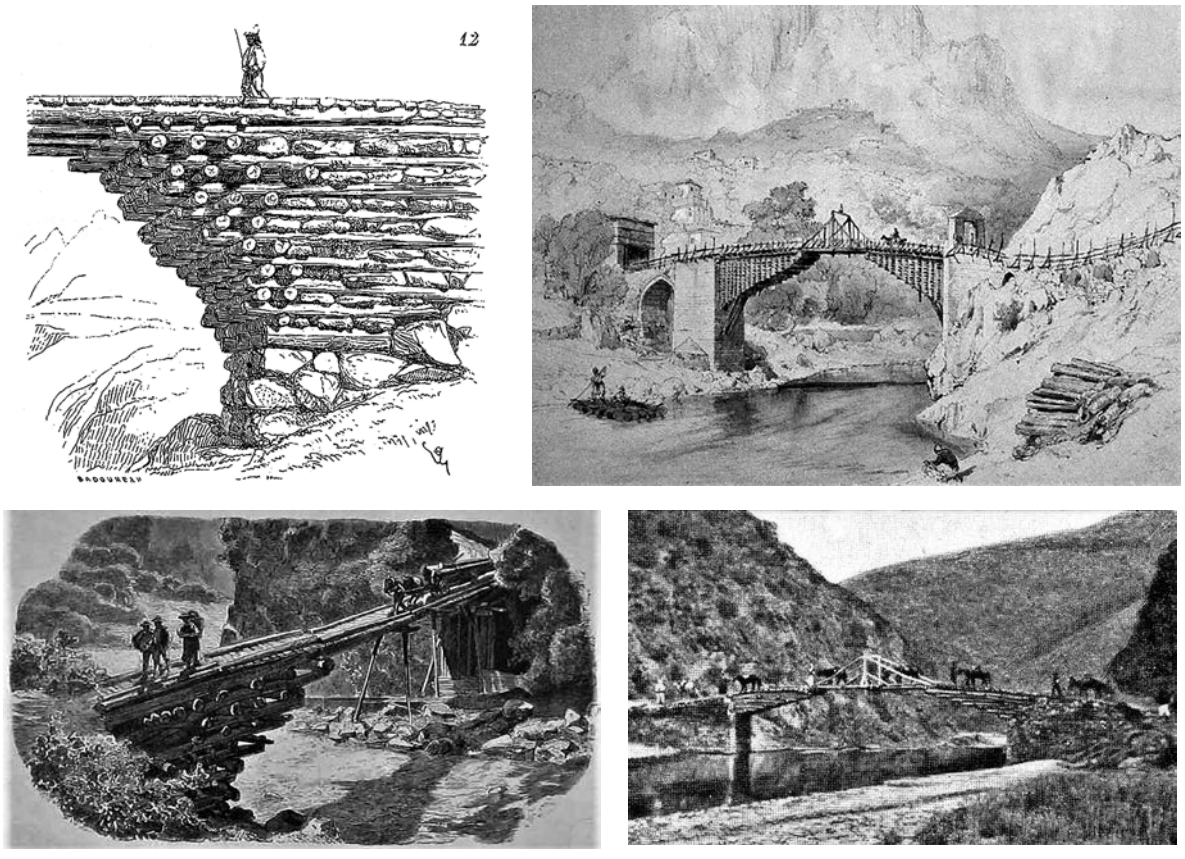


Bridge construction site. Assyrian bronze relief from the Balawat gates.
Shalmaneser III crosses a bridge. Assyrian bronze relief from the Balawat gates.
Shalmaneser III's army crosses a bridge. Assyrian bronze relief from the Balawat gates.

According to Herodotus in his campaign against Kyrus, Croesus (527 – 541) crossed the Halys river 'by the bridges which exist there'. In addition, he looks into another 'general belief of the Greeks' that the Phoenician mathematician and philosopher Thales 'divided the stream' into two so that the army can cross over. In fact, what stays behind this hazy and even illogically

expressed statement is, undoubtedly, a hydrotechnical division of the river into two beds, in order to construct smaller bridges by subsequently redirecting the stream under each one. This provides an explanation of the two bridges Herodotus writes about. The complicated large-scale construction manner makes it clear that the bridges are not pontoon ones, but permanent ones, most probably in stone and wood, similar to the Babylon one.

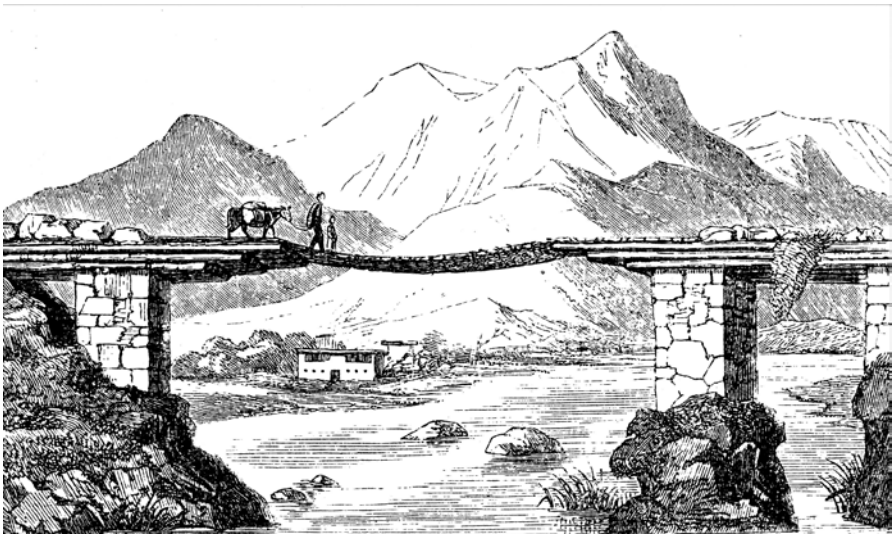
A specific type of wooden bridge deck survived through the centuries in northern Mesopotamia – it is based on top of stone abutments by means of several layers of logs protruding console-wise. Viollet-le-Duc defines it as an ancient construction convenient for wooden bridge-building and used by various civilizations, including the European one. The consoles consist of several layers of longitudinal rows of logs fixed onto one another, with each upper row protruding over the lower one so that the foremost side of each opposite-end pier provides the static thickening of the whole construction. An improved alternative variation includes cross rows in addition to the longitudinal ones, which consolidates the console. I think, however, that this improvement is a later one. Viollet-le-Duc adds to his text a drawing of the console of such a bridge from Gaul. I am familiar with the same type in Albania – from photos by De Grande and Loius – Jaray (1909). I have also seen the same type of bridges on photographs and drawings of Euphrates (XV c.), in the vicinity of the ancient Armenian capital Ani and at least three over the Zab river.



Gaulish wooden bridge with cantilevers. Illustration by Viollet-le-Duc.
XV century Anatolian wooden bridge with cantilevers over the river Euphrates by Agat (Egin) near Ani. Watercolor from 1849.
Swiss wooden bridge with cantilevers by Ticino Faido, 1878 engraving.
Albanian wooden bridge with cantilevers by Vaumat [Vau i Matit] over the river Fan. Photo by Gabriel Louis-Jaray, 1909.

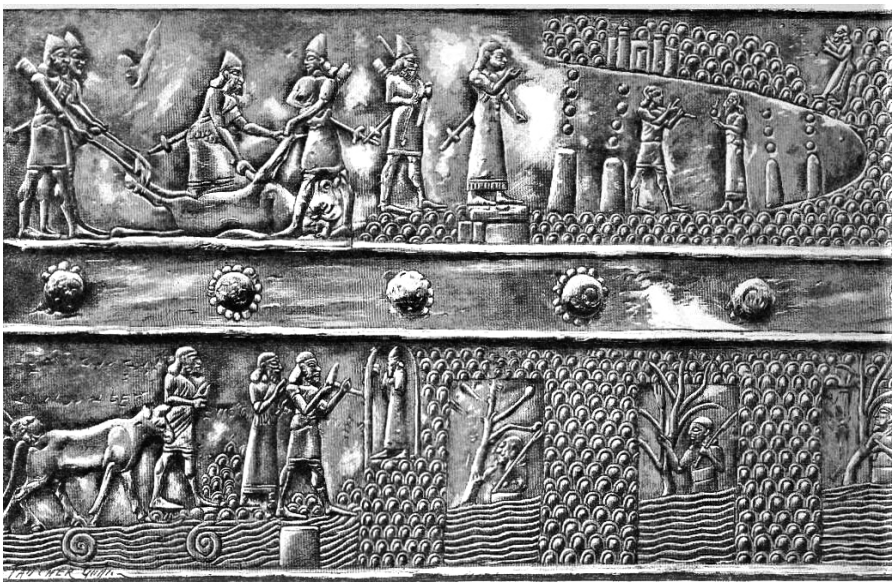
In the context of the development of this construction idea, the stone bridge drawn by Henry Layard in 1849 is extremely important – it is located in close proximity to his

excavation works in the Assyrian capital of Nimrud. Here, the construction idea of the multilayered wooden consoles was executed in stone. The abutment here is cyclopean masonry dressed on the face, while the deck above them is constructed from long thick stone slabs on consoles composed of the same material. It is evident how the archaic stone-and-wood type of bridge evolves from the wooden decks and consoles to constructions entirely of stone slabs.



Stone bridge with stone slab pavement near Nimrod over the river Zab by Mosul, Iraq.

Undoubtedly, this is one of the types of bridge illustrated on the binding strips in Balawat dated at the time of Ashurnasirpal II (884 – 858) and Shalmaneser III (859 – 824). It is constructed entirely in stone with four column-like abutments on top of which lies a stone deck, undoubtedly composed from slabs. The first of the columns seems to be a gate-tower with a guard standing in it judging by the pointed helmet – the same type of helmet worn by the other soldiers on the fragment.



Stone bridge with stone slab pavement. Assyrian bronze relief from the Balawat gates.

This bridge system, undoubtedly, draws on the ideas of bridges composed from stone-block supports and a deck of large, thick slabs lying on top of the supports. Today, there are few remnants of such cyclopean masonry, and the only one still intact and in use

(of course after multiple repairs) I am aware of is the 55-metre Tarr Steps over the Barle river, close to Somerset, England, dated between 1400 and 1000 year BC. Formatted 2-tonne slabs lie on blocks planted vertically in the river bottom. In XIX c. Samuel Smiles will point to several more in the same region and will firmly state that „No such structures resembling these bridges have been discovered in any other part of Great Britain, not even in Brittany…The only bridges they have anything in common with can be found in Ancient Egypt, to which they bear a striking resemblance".This means that this construction idea was applied in Egypt, besides the Two Rivers.



The 1400 - 1000 B.C. stone bridge „Tarr Steps" over the river Barle, near Somerset, England.

In France as well there can be seen bridges with masonry, hence higher, piers and a stone-slab deck, each of which is several tonnes. Such bridges date from the Middle Ages, but as evident from Tarr Steps, their origin dates much back in time, although they were constantly renovated in their archaic look during the Middle Ages.

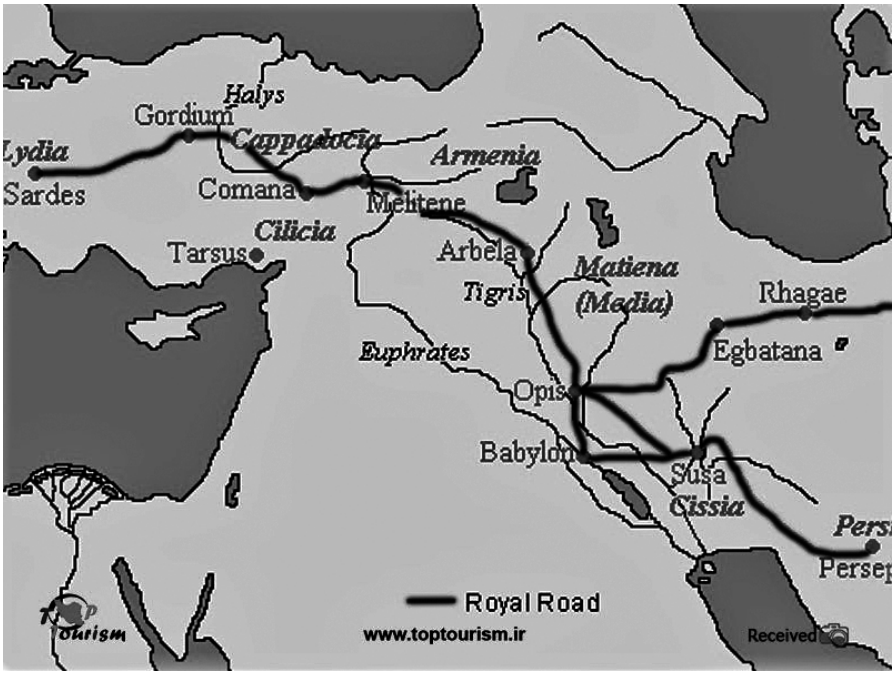
Such was the solution, but at the smallest height possible, of the much later Brauron bridge near Athens (480 BC), spanning the Erasinos river and leading to the temple complex of Artemis. It is a large deck composed of 10 rows of well-wrought elongated squared stone blocks lying on 5 partly submerged in water rows of upright blocks placed transversely. The idea provides a representative pedestrian route to the temple.



The V century B.C. stone bridge with stone slab pavement by the temple of Arthemis In Brauron, near Athens.

Everything looked at so far undoubtedly outlines three manners of construction of the most ancient bridges in Asia Minor – **wooden-pontoon, stone-and-wood and stone ones with a deck of gigantic slabs**. Undoubtedly, the woodlands also saw completely wooden ones. There is no conclusive evidence of vaulted stone bridges with the exception of one in Hatusha (the Empire of the Hittite), but dating it, apart from a hypothesis based on remnants of gigantic stone masonry, is quite a complicated task. Based on the Nimrud bridge and what Samuel Smiles says about Egyptian ones of the same type, it is also possible to talk of a deck of thick slabs.

After the Assyrians, the Persian shahinshah Darius I the Great (522 – 486) besides his war pontoon bridges (built, in fact, by the Greeks), remained in history as the creator of a large-scale road system. Herodotus describes it in detail by regions, stating that a "kingly road" connects the cities of Sardis (between Izmir and Ephesus) in Ionia, with the capital Suza, equipped along the entire route of 2699 km with 111 "royal stations and excellent inns". The text does not say what bridges were built along the route but in addition to the mention that part of the rivers "have to be crossed by means of boats" it also alludes to the presence of at least three bridges. The description of what is, definitevly, bridges reinforced with tower-gates can be found in texts explaining that "On leaving Phrygia the Halys (present-day Kızılırmak, Turkey – note, N. T.) has to be crossed; and here are gates through which you must pass, so you can traverse the stream; a fortified outpost guards it…" and that "at the borders of Cilicia, you will cross (the river Euphrates – note – N. T.) through two sets of gates and guard posts." Consequently, what is referred to is bridges consolidated with tower-gates – the type we know from the Balwat gates' metal binding and the Babylon bridge.



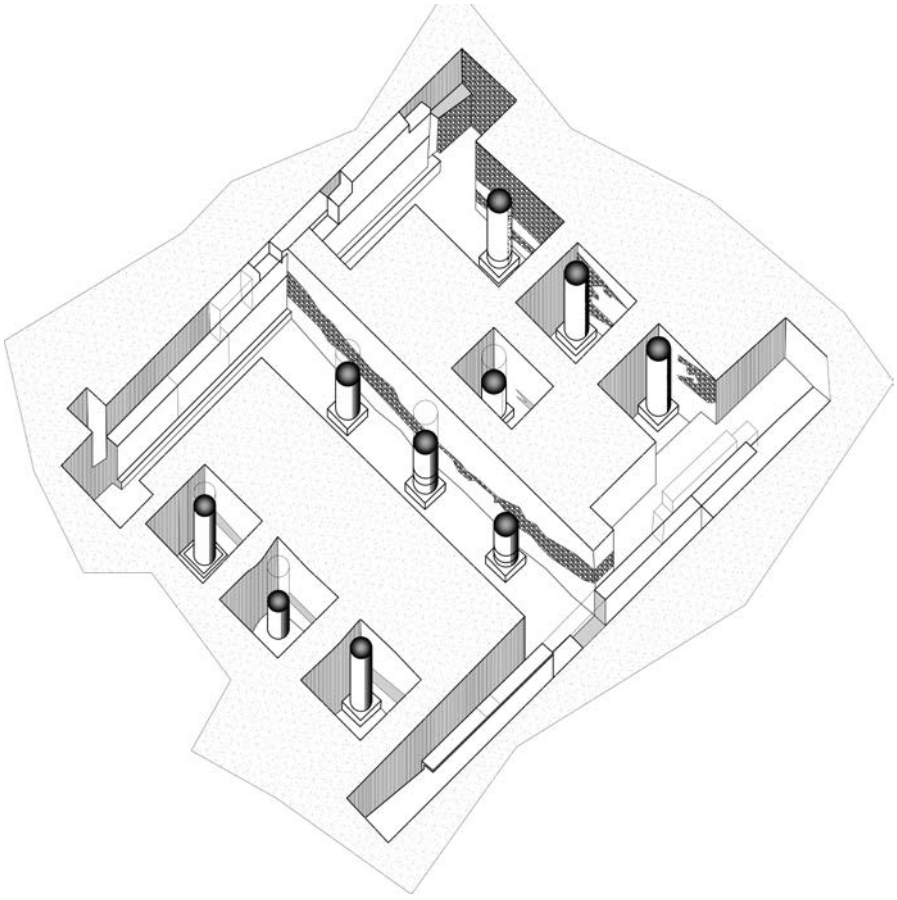
Map of the "Royal road" of Šāhanšāh Darius II (522 – 486 B.C.).

The 10-acre bridge lying on the imperial road at Amida (present day Diyarbakır) is considered to have been re-built by the Legio V Parthica, (founded by Diocletian, 284 – 305) in the place of the Darius' bridge. This is subsumed from the text of Ammianus Marcellinus too.

It can be assumed that the earliest Persian bridge constructions, similar to the Assyrian ones, were wood-and-stone ones, but it cannot be ruled out that the later ones might have been massive brick ones, possibly with stone coating. If we take into account

the famous inscription about the construction of the palace of Darius in Suza, which says that its builders were Syrians, Ionians, Mydians, Egyptians and Sardinians (i.e. from Sardinia – note – N. T.), all of them said to be masters in stone processing and masonry, each of the two thesis could be possible.

Modern science believes that bridge-building for the Persians became practice as late as the time of the Achaemenids (550 – 330 BC), which is when evidence for this can be first found. After Darius' bridges, a more interesting one than the ones established archeologically is the one in the gardens of the palace in the capital Pasargadae (present-day Farce province in Iran), spanning a canal more than 2 m deep. According to Stronach, the composition of the piers consists of three rows at a distance of 3, 90 m, each consisting of five stone columns on top which lie wooden platforms. Thus, the total length of this representative bridge measures 15, 65 m and a width of 16m. It is considered to date back to the IV c. BC. Its construction, though, isn't unique as it has its parallel in the Brauron bridge (480 BC) over the Erasinos river. Due to this similarity and the use of stone slabs, it is possible that this Persian construction is the work of builders from the Lydian coast.

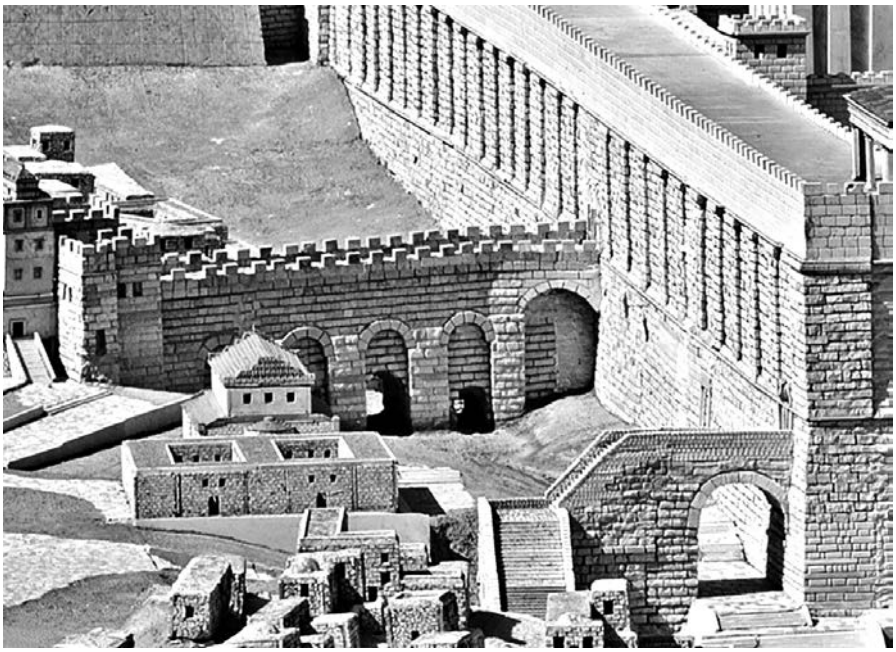


V century B.C. stone bridge with stone slab pavement in the palace garden of Pasargadae. Plan by Stronach.

After the Syrian lands were annexed to the Achaemenidian Empire ar. 540 BC, Persia saw a more noticeable presence of Syrian master builders. The Syrians remained foreigners until 333 BC when Alexander of Macedonia defeated Persia and after his death in 312 BC, a Hellenic Syrian Kingdom was founded by his diadoch Seleucus I Nicator (305 – 281 BC).

Probabaly the only bridge which is not part of the picture of the bridge construction practice of the epoch and place is the bridge over the ravine Tirolion between the Sion hill and the Jerusalem temple built by Solomon simultaneously between 957 and 950 BC (or

1014 – 1007 BC). The masters came from the Phoenician city of Tyre, but as Hiram Abiff – the main architect sent from there – is said to be Assyrian, it follows that at least some of them were his fellow-citizens. In the XIX c. there can still be seen remains from the bridge's round arch (so-called Wilson's arch) built from cyclopean dressed blocks, which is, rightfully so, considered a part of the bridge. In any case, it belongs to a later stage of the building history of the temple after it was destroyed by Nebuchadnezzar II in 586 BC. This is evident by the fact that the large stone blocks – remains of the Solomon's wall - have foundations beds carved to accommodate the abutments of the vaulting. Judging by the level of bridge building in the Two Rivers at the beginning of I millennium BC, the Solomon's bridge was, undoubtedly, of the stone-and-wood type or with a stone-slab deck on top of high, huge squared stone block abutments. The open vaulted bridge is most probably part of the reconstructed in 535 – 515 BC so-called Second Jerusalem Temple.





The bridge of the second temple in Jerusalem (535 – 515 B.C.) Model with marked intact remains.
The bridge of the second temple in Jerusalem (535 – 515 B.C.) – Wilson's Arch.




Semi - circular arch (round 2000 B.C.) from Tell al-Rimah near Nineveh, Iraq.
Semi - circular arch (round 1800 - 1750 B.C.) from the gate of Laish, currently Tel Dan, Israel.

In the Two Rivers and Egypt, however, the arch is a familiar construct as far back as III millennium BC. While in Egypt the early arches are corbel, real vaulted fairways have been excavated in the big Mesopotamiam city of Ur (dated 2100 BC). The arch of Tel Dan Mudbrick Gate also dates to XVIII c. BC, today in Israel. Two and three-layer arches have also survived in the ruins of Babylon and its colony at the Tigris River – Ashur (today in Iraq), from the times when they were part of the Assyrian Kingdom. [3] It is exactly in those regions that trapezoid elements were first created for the purposes of arch construction. So it can be positively said that the semi-cylindrical and pointed vaulting were constructed in Mesopotamia not later than the end of III c. BC. The mathematical explanation of the construction though ancient authors attribute to the Greek philosopher Democritus from the Thracian Abdera (V – IV c. BC), when it was already applied in the construction of the Thracian stone tombs.

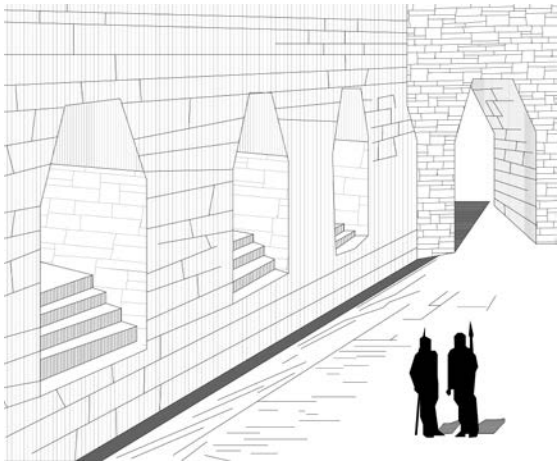


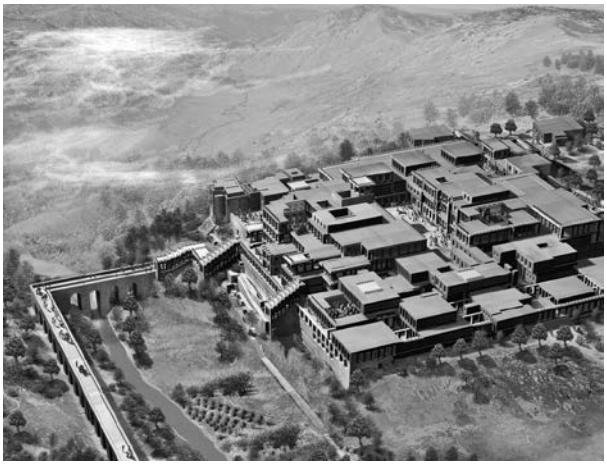

Semi – circular arch (21 - 18 century B.C.) from E-Dub-la-mah, entrance edifice for the ziggurat of Ur, currently Tell al-Muqqayar, Iraq.
Arched aqueduct of the Nippur temple, in modern Nuffar, Iraq. Photo by John Henry Haynes, from the 1893 excavation site.



The domed tomb "Golyama Arsenalka" (V century B.C.) in "The Valley of Kings" near Kazanlak, Bulgaria.

During the last millennium BC, the arch continues to be present in the Assyrian architecture but increasingly in stone. The Assyrian bas-reliefs show illustrations of vaulted city gates as well as hemispherical domes originating, for example, from Nimrud, Nineveh and other cities. Their existence has been established archaeologically – at that vaultings ranging from one to the more stable two, three or more layers of bricks or squared stone blocks lying on top of each other. Therefore, it comes as no surprise that it is the Syrians who will later introduce the column-arch architectural system in culture of the Roman empire. [4] Being among the main builders of the Ancient World and later of Rome in particular, they seem to have spread the round arch in VI – V c. BC as a new for Europe and Africa architectural and construction element replacing the old corbel arches. [5]





The stone bridge with corbel arches (round 1500 B.C.) in Knossos, Crete. Reconstruction by Sir Arthur Evans.
The stone bridge with corbel arches (round 1500 B.C.) in Knossos, Crete. Modern reconstruction

In parallel to the construction of stone-and-wood bridges and stone ones with spanning slabs in Asia Minor, after 1700 BC the Minoan culture of Crete manages to construct bridges entirely out of stone. One of the most ancient ones, often dated along with the palace between 1700 and 1400 BC (according to Evans – 1900 BC, today – 1500 BC) was created as a connection between the Main and the so-called "Small Palace" in Knossos. It spanned the Kairatos river and was built at the time of reconstruction of the palaces. The bridge is a long structure with stone piers constructed as columns of up to 10 m in height and connected, according to Evans, by means of corbel arches of a trapezoid silhouette. I could not discover evidence of this, but in the early construction cultures of Asia Minor and Eastern Europe from II millennium BC, the form is triangular, pointed, sometimes with bent sides and quite rarely trapezoid. Such gates can be found in different parts of northern Asia Minor and they are contemporary to the bridge in Knossos. Therefore, anything is possible, but for me the more acceptable one is the stone-and-wood construction.

A similar, but more primitive is the manner of construction of all early Mycenaean stone bridges in southern Greece, mainly Peloponnese. Their masonry though, unlike the gigantic squared stone blocks bridge in Knossos, is archaic cyclopean.

In the region of Argolid (Argolis) – land to the important in antiquity cities of Argos, Mycenae, Tiryns, Terzen, Epidaurus and many others, there are preserved remains of a road system with bridges dating back to II – I millennium BC. These are the oldest entirely preserved European stone-vaulting bridges. They have a primitive cyclopean masonry of unwrought or semi-wrought boulders, with pointed corbel-arched fairways. So far there is documental evidence of 30 such bridges, but unfortunately almost all of them have been destroyed during the layout of the roads during XIX and XX c. A larger one is the "Kazarma" bridge (1300 – 1190 BC) near Argatino-Argolos village in Arcadia, on the war road Mycenae – Epidaurus, with a length of 22 m, width of the vaulting of 5, 60 m, of the superstructure – ar. 2,5 m and height of ar. 4 m. At around 1 km west of the "Kazarma" bridge there is located "Petrogefiri" bridge, which is slightly larger – 5, 20 m wide at the foundations and 2, 40 m of the deck. Its architectonics is identical with that of "Arcadico". A Mycenaean cyclopean bridge can be found in the region of Likotrupi along with part of the deck of large gravel mixed with slabs. The fairway of this bridge is small of rectangular section.





The Mycenaean cyclopean masonry stone bridge "Kazarma" (1300 – 1200 B.C.) by the village Arkadiko Argolida, near Nafplion, Peloponnese, Greece.
The Mycenaean cyclopean masonry stone bridge "Petrogefiri" (1300 – 1200 B.C.) by the village Arkadiko Argolida, near Nafplion, Peloponnese, Greece.

Near Mycenae itself there is preserved coastal superstructure – part of a much larger bridge. The corbel arch and the other part of the structure are missing. The preserved

part measures 12 m, while the total length is ar. 25 – 30 m. And since this belongs to the qualitative for its times construction work in the capital, the cyclopean masonry is much finer with horizontally aligned rows.



The Mycenaean cyclopean masonry stone bridge "Agios Georgios" (1500 – 1400 B.C.) by Mycenae (Mykēnē) near Argos, Peloponnese, Greece.

The origin of the builders of the stone bridge in Hatusha is unknown, but for the time being it is considered that the earliest samples of cyclopean stone corbel arches and vaultings that have been preserved or archaeologically studied are the ones in the bridge-building practice of the Minoan culture. Those were applied in palace, tomb and fortress construction as well as in bridge construction throughout II millennium BC. The preserved bridges from Peloponnese, for example, date back to ar. 1300 – 1100, which is the time of the Hatusha bridge unless the latter is a stone-and-wood one.

This type of cyclopean stone construction appears in II millennium BC at other places in the then Mediterranean World – Thracia (possibly earlier), the Hittite Empire, Etruria, Mesopotamia, Caucasia , North Africa, the Iberian peninsula, etc. [6] Maybe this fact confirms the contested by some scholars texts by Herodotus and other ancient authors on the migration of the Hellenes (but also Thracians) towards Asia Minor and Etruria; on the Lydean origin of the Etruscans and the migration of the so-called 'Sea peoples' from the Phoenician coast and North Africa towards the Hittite – a migration that took place mainly in XIII BC. [7] Gelanic of Lesbos (of Miletus, ar. 485 – ar. 400 BC) will say that after being banished by the Hellenes, some of the Pelasgians from Peloponnese settled in Tyrrhenia, where they mixed with the local population – the Tyrrhenians and as Dionysius of Halicarnassus later writes (60 – 7 BC), this new people was called "Etruscans". Towards the end of IV c. BC at the latest, the Hellenes did not only have trade relations with Sardinia, but colonists of their own in the Phoenician towns founded in VII c. BC. Together, they will stimulate the development of construction practices there. Thus, the whole material practice of the Etruscans whose cultural upsurge began in VI c. BC is strongly influenced by the Greek and Phoenician cultures. [8] According to certain written

and archaeological evidence, what contributed to this was the Phoenician, the Sardinian and, beyond doubt, the Assyrian masters working in various industries, including construction. Therefore, it is of no surprise that the first Etruscans monumental fortifications are of the megalithic Mycenaean – Thracian – Hittite type. This type of fortifications together with the pointed corbel-arched gates, completely identical with the Mycenaean and Hittite ones, have been preserved near Arpino, Frosinone province. The city (present-day Civitavecchia), known under the name Arpinum under the Romans, dates back not later than VII c. BC as far as these ruins are concerned. [9] Consequently, the same architectural manner of thought as in the Mycenaean and Hittite civilizations can be seen when it comes to the masonry technique applied in wall and gate construction. [10] Having in mind that the oldest samples of megalithic structures on the Apennines date to ar. XI – IX c. BC on the one hand, and the great similarity with the much older cyclopean masonry on the Balkans as far as the northern territories of Asia Minor on the other, they are highly likely to be imports from there.

Indeed, in present-day Italy I have no information of preserved bridge constructions dating from this early Etruscan epoch, but I have found evidence that at a later stage, it seems, probably IX – VII c. BC, there were built stone bridges of the Mycenaean type at that, with cyclopean masonry and fairways, formed by protruding stone blocks, adding up to approximately triangular or another rounded-off shape at the point - maybe similar to the gates at the time and, sometimes, with slightly curved sides. Evidence of the similarity between building apertures and bridge fairways in Etruria can be found in two drawings by Carlo Labruzzi from 1789 showing bridges on the Via Apia route between Velletri (Velitrae) and Lanuvio (Lanuvium). The bridges shown are early Roman bridges of squared stone blocks, as well as inbuilt in them earlier ones, constructed of boulders with cyclopean dry masonry. It is clear to see how the one at Velletri stands between the facades of the newer one, constructed of huge boulders, its fairway being pointed - of the Mycenaean – Asia Minor type. The other one, at Lanuvio, was also inbuilt between the two outer walls of a Roman bridge, constituting its filling. Its archaic cyclopean construction can be seen behind the newer bridge arch.

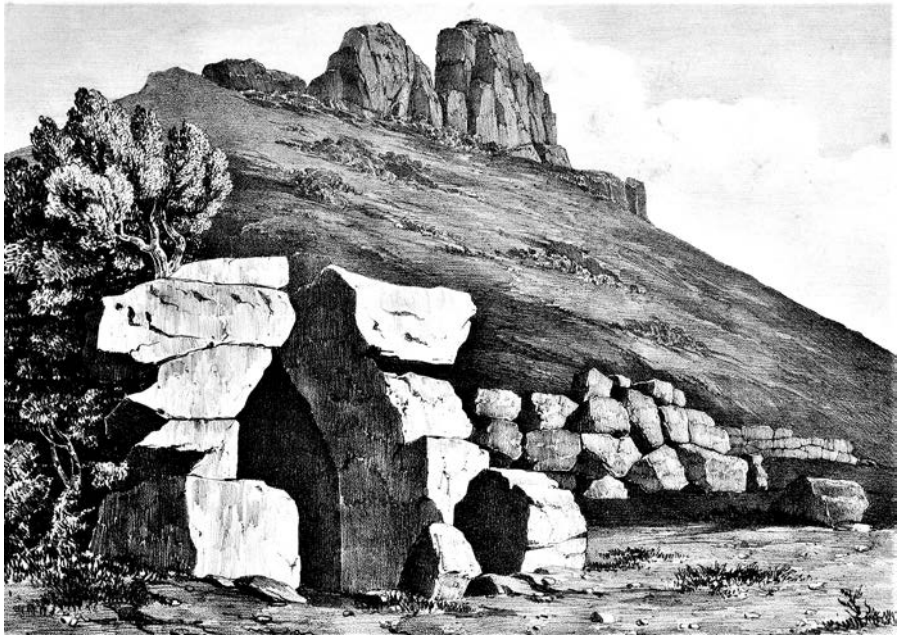




Etruscan cyclopean masonry stone bridge (IX – VII century B.C.) near an early roman one on Via Appia near Veletri. Engraving by Carlo Labruzzi, 1789.
Etruscan cyclopean masonry stone bridge (IX – VII century B.C.) built in an early roman one on Via Appia near Lanuvio. Engraving by Carlo Labruzzi, 1789.

Another bridge corbel arch, still intact in the first half of XIX century, is also dated VII – VI c. BC – in the vicinity of Terracina and known to us from an engraving by Edward Dodwell during his trip in Etruria in 1834. The facility is constructed from gigantic dressed stones, hence the fairway has the clear shape of an elevated pointed

pseudo-vault. The bridge has smooth dressed work surfaces, such as, for example, those of the Foca and San Pietro gates of the pre-antiquity rampart of Segni. Some early XIX c. engravings I am aware of define the facility as part of an underground aqueduct, but even if so, this does not in any way change the fact that this stonemasonry manner of corbel arches construction had already replaced the cyclopean structures of the previous centuries.



Ruin of a bridge or an aqueduct (VIII – VI B.C.) near Terracina, Italy. Engraving by Edward Dodwell, 1834

The most intriguing question in the realm of massive bridge construction is the existence of a cyclopean type of corbel arches, similar to the Mycenaean ones, on the territory of Asia Minor, at that as part of the early construction practices on these lands. Unless this was some cultural contact with the Minoans ahead of its time, it probably took place ar. XIII – XII c. BC as a result of the settling, first on the Asia Minor coast, of the so-called "Sea peoples", among which prevalent were the Greeks coming from the continental and island territories. The Hittite, for example, mention their neighbors – the Achiyawaians, a name behind which stay the Greek Ionic colonies from the northern coast of Asia Minor, with whom they had trade relations.

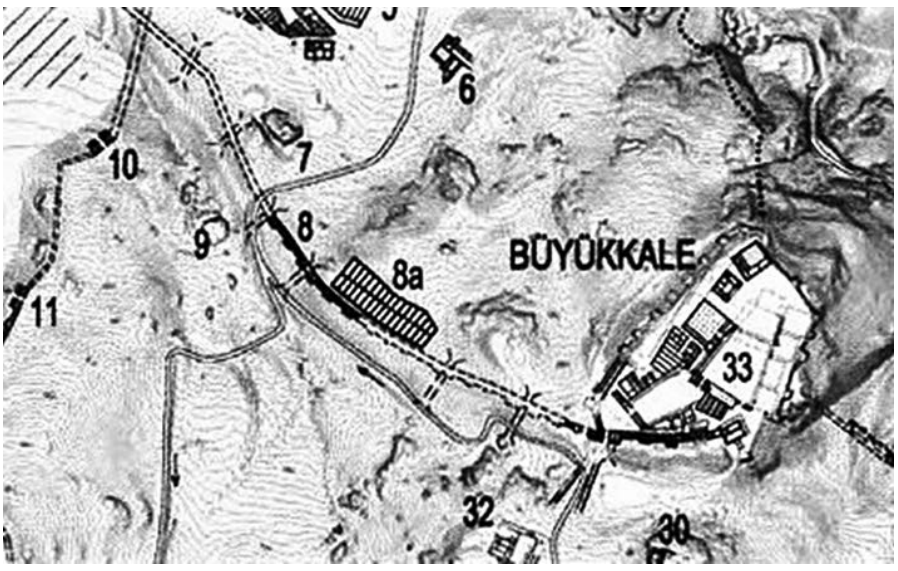
The emergence of cyclopean architecture in the Hittite Empire is, at first thought, inexplicable if we take into account the fact that its coast is in a direct contact not only with the Ionians, who had migrated there, but also with the aqua territories of Crete and the Greek islands, part of which, actually, belong to it. Northern Asia Minor saw, however, as well Thracians migrating to it, who also used cyclopean masonry in II millennium BC. In addition, the Pelasgians who settled on Peloponnese and were the first to build it up in cyclopean masonry buildings and facilities, were Thracians too. Therefore, the question about the way of the cyclopean architecture from Europe to Asia Minor or vice versa is still unanswered. It can be said, though, that this large territory from the southern Balkans to northern Asia Minor and the Caucus had a unified architectural profile in the XIV – XII c BC.

When, after a series of destructions in the wars against, mainly, Egypt, and the desolation following its being conquered by Amenhotep III, the Hittite king Suppiluliuma I (1344 – 1322 BC) made the ruined city of Hatusha (present-day Boğazkale, Turkey) capital again. He then started monumental stone masonry which gave it a new profile. This first fortified territory was the core of the so-called Inner city. Later, under the reign of King Hatushili III (1267 – 1237 BC) after the conclusion, at last, of a peace treaty with

Ramesses II and above all, under his son Tudhaliya IV (1237 – 1209 BC), south of the initial city there was built the twice larger, so-called Outer ("Lower") city.

If the start of the construction of Hatusha is clear due to the many cuneiform tables found in the ruins as well as some external evidence, as to the Outer city, it can be referred to the period 1267 – 1209 BC. Toward the end of the last period of this construction there can be dated the construction of stone- foundation bridge which spanned the ravine dividing the city into two parts. Some authors think that the number of bridges over the ravine was much higher, but so far only this one has been archaeologically established. All public constructions in Hatusha are of cyclopean masonry, while the gates are constructed as pointed corbel arches, decorated with Assyrian palladiums in relief. The excavated bridge piers positioned quite close, in view of the level of the terrain on both sides, are indicative of spannings of the Mycenaean corbel arch type rather than a wooden road, but this cannot be said with absolute certainty, so it could as well be a stone-and-wood construction.



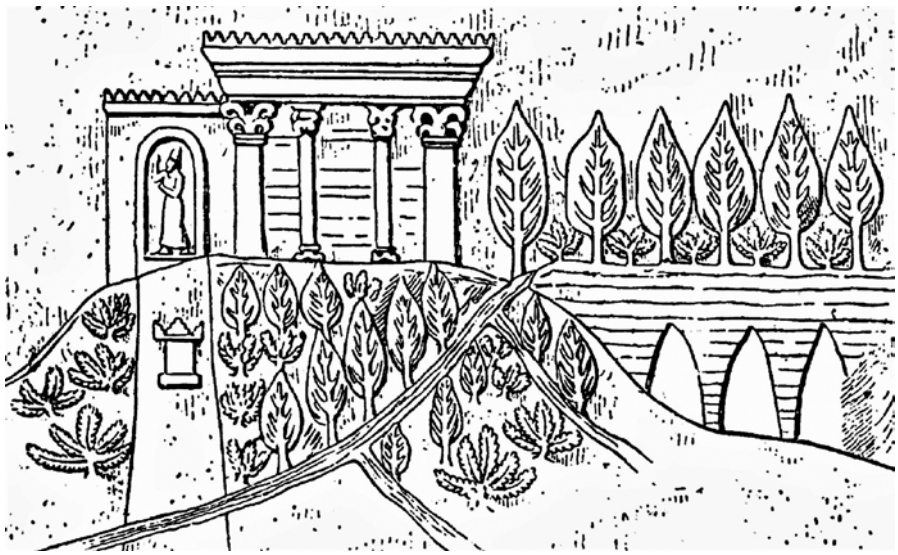


Hittite empire. The bridge (1267 – 1209 B.C.) between the Citadel and lower city Ḫattuša. Reconstruction.
Hittite empire. Bridges (1267 – 1209 B.C.) between Upper and Lower city Ḫattuša. Archaeological plan fragment.

This particular type of corbel arches can be seen in other ruined bridge remains whose height has been preserved to a greater extent in the Mesopotamiam countries. All that I know of are more recent than the Minoan ones, but if we judge by the fact that there were excavated cyclopean masonry constructions and corbel arches on these Asia Minor territories, dated XIII – XI c. BC, we can hypothesize that at the time there were also built bridges with the assumed cyclopean characteristics of the Hatusha bridge.

The first reliable evidence of stone bridges in Asia Minor is of as late as VIII and VII c. BC, when in addition to adobe and bricks, dressed stone only is used.

An Assyrian relief, categorized in literature as originating from Dur-Sharrukin (present-day Khorsabad, near Mosul, Iraq), but labeled, in British Museum, where kept, as coming from the Northern palace of Nineveh (present-day Kouyunjik, Turkey) shows constructions made under King Sargon II (Acadian Sarru-kinu), ar. 721 – 705 BC) from the period after 721 BC, when the Assyrian ruler created a palace complex in his new capital Dur-Sharrukin. The image shows a stone aqueduct with pointed corbel arches stepping on slender pillars. In close proximity to the city there are archaeological excavations of parts of splendid bridge constructions from huge dressed squared stone blocks with pointed corbel arches which probably belong to this same aqueduct. Some of the literature on the question, however, describes them as remains of a bridge dating 717 – 705 BC and connecting the king's palace with the temple of Nabu. This temple, however, is illustrated in the relief as adjacent to the aqueduct, which is in turn irrigating the garden in front of it, which makes this thesis dubious.



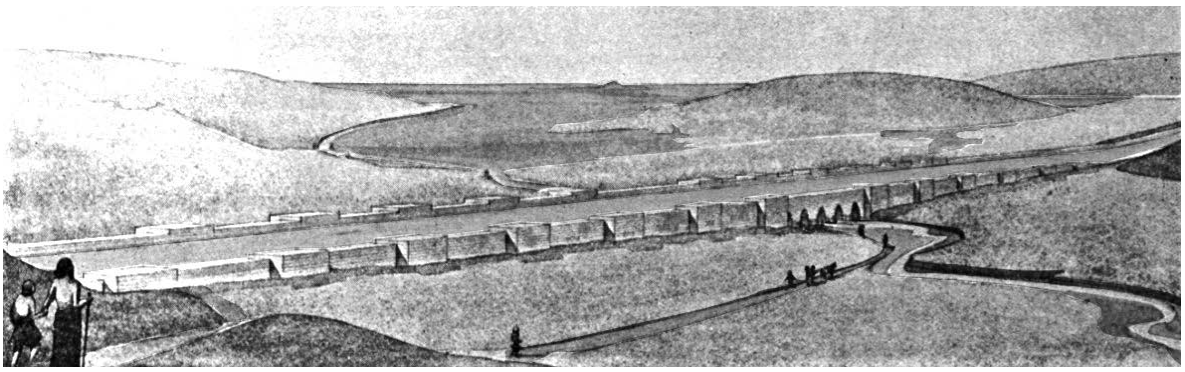




Assyria, Nineveh. The Aqueduct (717 – 710 B.C.) supplying water for irrigation of the temple's gardens. Sketch of stone relief.
Assyria, Dur-Sharukin. The aqueduct or bridge (717 – 710 B.C.) by the temple of Nabu.
Assyria, Dur-Sharukin. The aqueduct or bridge (717 – 710 B.C.) by the temple of Nabu.

The remains of another excavated facility with pointed corbel arches and intended as spanning the riverbed of the Khosr river in front of the park gate of Nineveh undoubtedly show it was a bridge construction since it lies on the royal military road of Sennacherib dated ar. 690 BC.

The largest pre-antiquity aqueduct was also excavated ruined – near present-day Jerwan (Iraq); it provided water for Nineveh (present-day Kouyunjik, Turkey) at a distance of 40 km. It was built by king Sennacherib I (704 – 681) along with the palace and the city walls with 15 large gates in front of the main ones, from which short bridges were constructed over the ditches. The aqueduct itself is ar. 300 m long and, just as the one in Dur-Sharrukin, has five corbel arches spanning the river. The preserved piers as well as the whole body of the facility is made of huge finely dressed squared stone blocks, with the slopes of the shape of the corbel arches being achieved by cutting out the curvatures on the face of the squared stone blocks. The width is considerable – ar. 24 m, which means that the aqueduct transferred huge quantities of water. [11]

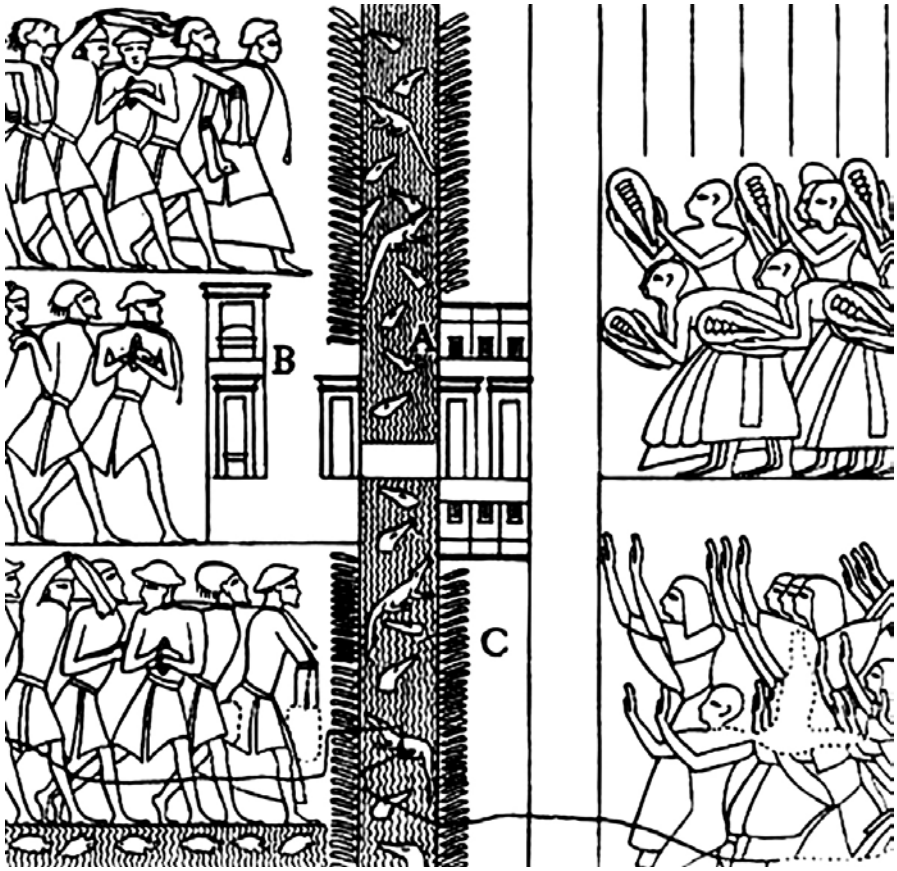




Assyria, Jerwan. Aqueduct of Senahirim I (704 – 681 B.C.). Reconstruction by Jacobsen.
Assyria, Jerwan. Aqueduct of Senahirim I (704 – 681 B.C.). A picture from the ruins.

All Assyrian facilities built after X – IX c. BC were constructed with finely dressed large-format stone. Accroding to Choisy this new stone-carving technique has its origin in Mesopotamia during the Iron Age, when, in his words, durable tools appeared, allowing such a technique. [12] Also, according to him, an example of the transition towards the new technique is a type of cyclopean masonry with horizontal levelling layout of the stones every other two rows, which, he goes on to say, is also used in the Sardian tombs in Lydia and Lycia. [13] However, in Asia Minor well-dressed gigantic squared stone blocks can be also seen in the supporting walls of the Solomon's temple (X – IX c. BC) – the work of Assyrian and Phoenician builders; in addition, the above mentioned VIII c. bridges are also of squared stone blocks.

But even in Egypt it is the old part of the Khafra Temple (Khefren) from the Giza complex at the time of the Old Kingdom (2520 – 2494 BC) that is considered to be the earliest squared stone block masonry. It cannot be, however, stated with certainty whether the first Egyptian bridges were stone ones or stone-and-wood ones like the Mesopotamiam ones since the only certain evidence Egyptology has of the ancient bridge building at the place is what is depicted in relief in the hypostyle hall of the Amon Ra temple in the Karnax Temple Complex in Luxor._It depicts a moment from the return of Seti I Pharaoh (1291 – 1279 BC) after the war with the Hittite, in particular his arrival in the town of Tjaru (also Tharu, or Sile) in the Nile river delta, where a festive welcome awaits him. The town, which is the exit point for the eastbound military campaigns of the pharaos of the XVIII dynasty (XV – XIII c. BC) is depicted as located beyond one of the sleeves of the delta, teeming with crocodiles; over the river there is a bridge fortified with a tower-gate on the eastern side, while on its western side there is a gate, flanked with two other towers. Unfortunately, the bridge is depicted from an aerial perspective, which is why the structure is unclear. The presence of a guard tower-gate on the bank across the town could also mean that it was of the stone-and-wood type, similar to the Assyrian bridges mentioned above. The town, located north-east of the present-day Ballah lake and 4 km away from El-Qantara el-Sharqîya on the Sinai Peninsula, is still being excavated today. What has been established, though, is the fact that it was constructed with adobe and is, for the time being, dated at the time of the successor of pharaoh Seti I – Ramesses II (1279 – 1213 BC).



Egypt, Horus way at Taru. Fragment from a sketch of relief of Seti I (1291 – 1279) in the hypostylos of the temple of Amun in Karnak, Luxor.

A comparison between the Mesopotamian architectural works of III –II millennium BC and the Egyptian ones clearly shows certain parallels between them. Recently, many buildings bearing similarities in their architectonics have been collected in

a website by Francesco Rafaelle. Even without it, though, there are pronounced parallels between the brick constructions in Mesopotamia and Egypt. An expressive example of those are the constructions dating from the Third Dynasty and onwards – ranging from the adobe mastabas in the vicinity of Beit Khallaf to the town of Girga at the Nile river in Upper Egypt. In fact, it is there that a small vaulting has been preserved, constructed from trapezoid bricks similar to the ones in the Mesopotamian town of Ur, which are its contemporaries. A large number of brick constructions with two- and multi-layered vaulted structures appear after pharaoh Seti I as well, for example at the hamlet of the workers who built the tomb of Ramesses II at the end of XIII c BC. Undoubtedly, this second phase in the brick construction in Egypt is related to the fact that Seti captured masters from Asia Minor, most of whom, probably, Assyrins. This was to continue into the II and I millenium BC. Actually, holding masters of different industries in captivity, above all builders, is an ancient military practice. Therefore, on grounds of the brick construction in the Tjaru fortress, it can be hypothesized that not only it, but also the bridge and its east gate were created by people from Asia Minor, with the installation itself being stone-and- wood.

North of Egypt, at the Knossos palace on the Crete island, there can be found early masonry in squared stone blocks and stone details dated ar. 1700 – 1400 BC.

Therefore, the appearance of the squared stone blocks technique cannot be referred to the Iron Age and dates back to the Bronze one. It seems that it is not so much the invention of the iron stone-carving tools, but rather the invention of metal block-ties welding the squared blocks together through fixing the ties with or without melted lead that has contributed to the mass dissemination of construction with dressed stone in the Mediterranean. [14]

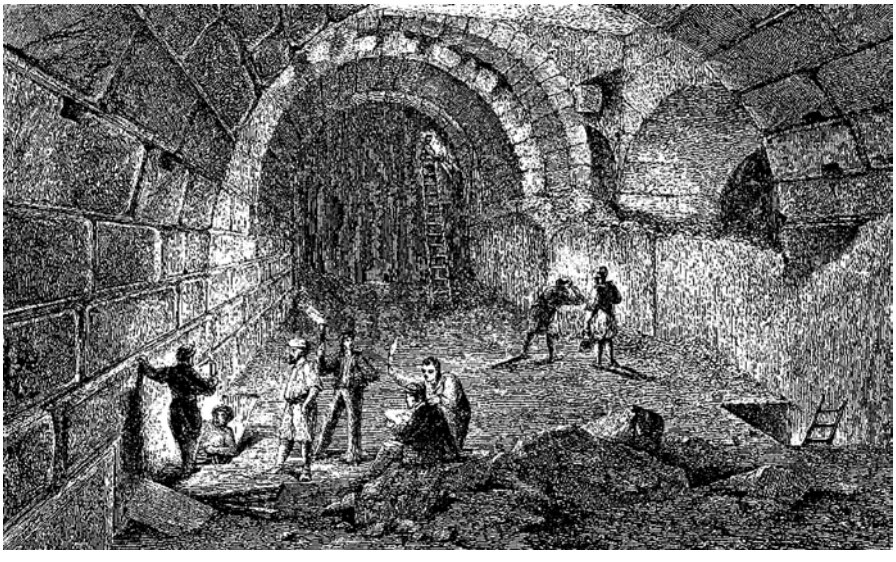
Some scholars believe that iron appears in Asia Minor and the Mediterranean at the time of the migration of the "Sea people", ar. XIII c. BC after the fall of the Mycenaean civilization. And while at the beginning the squared stone blocks masonry utilizes entirely the dry masonry technique of the old cyclopean masonry, at the time of the Neoassyrian kingdoms (ar. 1000 (934) – 605 (609) BC), when the dressed stone becomes the main building block, there appears a technique in which dressed stones are welded by means of metal block ties fixed into aperture with melted lead. Such elements were first established archaeologically at the foundation of the abutments of the construction in Babylon.

Initially the squared stone blocks technique is used in the construction of bridges and aqueducts with corbel arch fairways. Then, however, I deem it absolutely possible that not later than VIII c. BC in Syria there were also built the first constructions, apart from the defensive wall, with semi-cylindrical vaultings. It is then that at the dissemination of arched bridges and aqueducts begins. [15] The arched aqueduct of ziggurat in Nippur dates from VIII – VII c. BC.

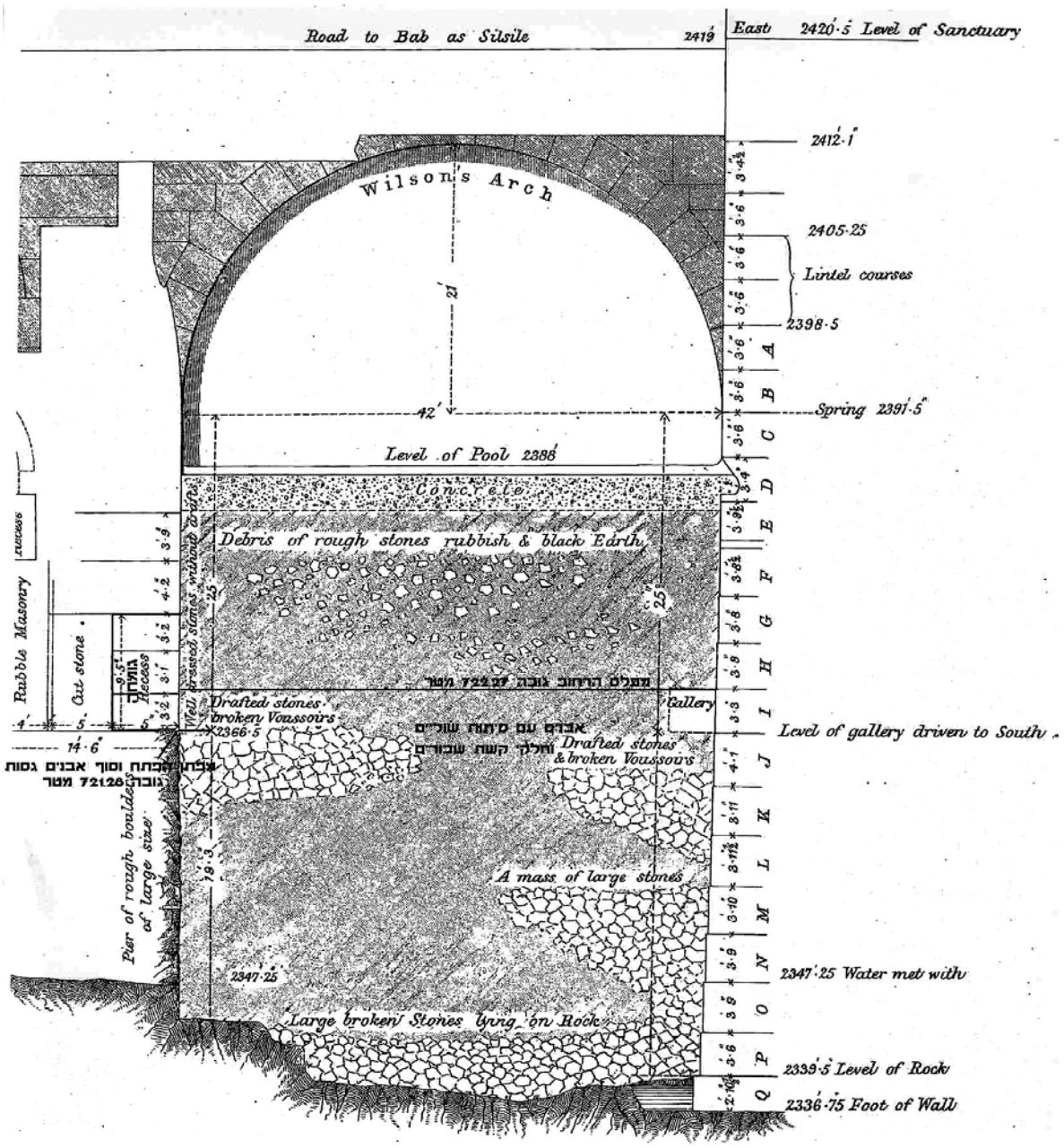
Early stone arches, constructed from trapezoid-shaped squared stone blocks were discovered by Austen Layard (1845 – 1847) in the so-called Southwest palace (IX – VIII c. BC) in Nimrud, as well as at some of the gates.

It seems that the reconstructed bridge at the Second Jerusalem temple was exactly such an earlier arched bridge structure; the reconstruction was part of the reconstruction of the shrine by the Jews in the period 538 – 516 BC, inititated by the Jerusalem Persian satrap Zerubbabel, as mentioned in the Book of Ezra (ch: 3:8.9). For the temple complex this is a new structure, as evidenced by the grooves carved in the older gigantic blocks which have survived and which belonged to the original Solomon's supporting wall; grooves made to mould a bed for the footing of the bridge arch. Many writers believe that the bridge belongs to the third temple of Herod as its southern and western supporting walls were reconstructed and enlarged. However, the part with the gigantic squared stone blocks at the level at which there is a groove for the bridge arch abutment, apparently belongs to the first temple, while the squared stone blocks in the enlargement are smaller in size and better preserved. [16] The statements which attribute the

construction of the walls enclosing the hill to Herod are based on the common view that when he starts the construction of a new, third Jerusalem temple, there was nothing left from the previous one. However, Flavius Josephus (37 – ar. 101) in Book 11 of his "Antiquities of the Jews" (Antiquitates Judaicae), firmly states that the idea of the king is to "make it larger in compass, and to raise it to a most magnificent altitude, as esteeming it to be the most glorious of all his actions". Then he goes on to add "as it really was". The text makes it clear that what was reconstructed was only the temple stepping on the uppermost level supported by the walls ("…took away the old foundations, and laid others, and erected the temple upon them, being in length a hundred cubits, and in height twenty additional cubits…", i.e 44 x 44 x 52,80 m), together with new gates, a monumental portico and some representative other buildings; the old walls fortifying the hill under all of these buildings were upbuilt to add the enlargement. Flavius goes on to say "There was a large wall to both the cloisters, which wall was itself the most prodigious work that was ever heard of by man…". He explains in detail how this wall was built on order by "Solomon…the first of our kings" from "rocks…and bound them one to another with lead… immovable for all future times". The third temple, according to Flavius is connected to its surroundings by four walls and from "the enclosure…the road descended down into the valley by a great number of steps, and thence up again". The foundations of one of them, the so-called Robinson's ladder were well studied as early as XIX c. and it was established that the flights of stairs were supported by a massive squared stone blocks arch system. Flavius make no mention of a bridge, but his description contains a paragraph on one of the gates, which in his words 'led to the king's palace, and went to a passage over the intermediate valley' and this goes to confirm my statement about the existence of a bridge there. Indeed, in his other main work "The Jewish War" (Bellum Judaicum), in his description of the conquering of Jerusalem by Gnaeus Pompeius (The Great) в 63 BC, Flavius comments on "the bridge over the depth that joined (the temple – note, N. T.) and the city" as still existent. Consequently, my hypothesis about the construction of the so-called Wislon's bridge not later than the time of the Second Temple, is not without justification. It is also possible that there were used foundations, or even entire abutments from the Solomon's bridge. According to the findings from the archaeological excavations, it had no fewer than seven arches and has been the subject of numerous visual reconstructions since at least from XVI. Here, for the first time in the history of bridge-building there were established square apertures in the footing of the preserved vault – apertures which were meant for the formwork during its construction. Today the Wilson's arch is low above the level of its adjacent terrain.



Jerusalem, Solomon's Temple (second temple). Wilson's arch – engraving of Robinson and Wilson's finding, 1857.



Jerusalem, Solomon's Temple (second temple). Wilson's arch – section of the excavated finding.

In summary of the achievements in bridge-building until VI c., there can be said that the Assyrian builders created not only the stone-and-wood bridge, but also the stone ones with a road of huge slabs as well as the squared stone block bridges – first constructed with corbel arches, subsequently – with real, semi-cylindrical ones. They were constructed with bricks, and later on with large-size finely dressed stone elements. Those stone elements are of trapezoid section when it comes to arches and vaults (a system which was invented ar. II millennium BC for the purposes of brick vaults and arches construction) and fixed in one, two or more layers one above the other along the radius of the section. This means that that it was not false, but real structural vaults with clearly expressed control over the horizontal thrust. [17]

With the semi-cylindrical vault of the bridge at the Second Jerusalem Temple we can also see rows of square apertures on both sided of the arch intended for the

framework to step on during the bridge's construction. The bridges on the territory of Syria dating to the IV – I c. BC show constructions with rows of cantilevers or imposts, instead of apertures, which were upbuilt in single or entire rows of squared stone blocks around the footings of the vaultings. Around III c. BC such constructions will appear in Etruria as well, most probably as an import from Asia Minor via the coastal colonies.

With the Babylon bridge there was also established the invention of the break-waters with a rounded-off top as is the particular case. Depictions of such break-waters can be seen, though, two centuries earlier on the casing of the gates in Balawat, which means that their emergence can be dated not later than IX – VIII c. BC. The idea was, undoubtedly, taken from the pontoon bridges with plank roads onto boats, the bows of which are oriented against the river current.

The foundations of the Babylon bridge show the use of large squared stone blocks welded with iron clamps filled with lead in the apertures. According to Flavius, though, this technique was used several centuries earlier by the Assyrian architect in the construction of the Solomon's temple. There were also invented the foundation piles as well as the pilaster consolidation of the body, which we know of from Jeruan.

From the military roads of Egypt under Seti I at the end of XIII c. BC to Persia at the time of Cyrus the Great bridges consolidated with gate-towers were also built, but probably such were built at other places in Mesopotamia, which was a leader in the art of fortification since, at least, the end of the third millennium BC.

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[2] Херодот. История. Част първа. С., 1986, 91-93
[3] Ashur is capital of the first Assyrian Empire of Samshi-Adad (1813 – 1781 BC), but during the Neoassyrian period between 884 and 859 BC it was changed to Kalhu (Nimrud). Ashur remained the main religious centre.
[4] Tuleshkov, N. Architecture in the Mediaeval Christian and Islamic World. Sofia, 2007, 12-13
[5] The idea that the arch is an invention of the Etruscans, adopted by the Romans, is a wrong one. With the Etruscans it appears due to the Assyrian and Mycenaean participation ar. XI – IX c. BC as a false one; in its hemi-spherical form it appears as late as VI – IV c. BC in the mature stage of their cultural development, again, due to the participation of masters of different professions from Greek and Asia Minor.
[6] It is fair to say that the question about the origin and dating of the cyclopean masonry still attracts a variety of statements. They are considered to be work of the mythic Atlantis people, migrants from Asia and in particular, India, who lay the foundations of the Minoan culture, or the local "protogreek" tribes (the Pelasgians). It is said that there are ancient cyclopean masonry remains all over the world – Asia, Europe, Africa, America and even Australia and Japan and Polynesia – similar in their technique and assembly at that. The architecture of the Incas also show the typical false arches and vaultings of pointed shape. It is possible that the migration of the "Sea people" reached America in XIII – XII c. BC, if not earlier.
The name "cyclopean masonry" we owe to Pliny the Elder, according to whom the stones of this masonry are so heavy that only the mythic cyclops could create them.
[7] According to historiography, the "Sea peoples" come mainly from the territories of the Mycenaean civilization, which was destroyed by the invasion of the Dorians from the north. As a consequence, part of the Achaeans migrated to the Hettite empire and other

territories. Apart from the Hellenes, Asia Minor also saw the migration of Thracians. Crete is considered to be the motherland of the present-day Palestinians.

[8] This is true of all spheres of cultural and public organization of the Etruscans, following the Greek system of the towns-poleis, twelve of which united in VII c. BC in the union of Etruria which was the beginning of their cultural and military upsurge.
[9] Some authors date the structure to the XIII c. BC, while the preserved gate is here known under the name the 'Gothic Arch'. According to Giuseppe Ludgli (1890 – 1967), the construction is from VIII – VII c. BC, but I think it is older - X – XI c. BC.
[10] The fact that the megalithic constructions on the Apennine peninsula have Minoan and Mycenaean origin is maybe proven by the specific for these cultures stone-carved solar type of cross placed in the centre of the key stone of the Saracena gate in Signi – we see on an engraving from XIX c.
[11] Jacobsen, Th. The Water Supplies of Nineveh. –In: Sennacherib's Aqueduct at Jerwan, Chicago, 1931, p. 36
[12] Шуази, О. Всеобщая история архитектуры. М., 2008, с. 291
[13] The same source. The town of Sard (Sardis) is capital of Lydia (present-day Manisa region, Turkey) – successor of the Hettite Empire after XII c. AD. It is believed to have been founded by Gyges (716 – 678 BC) in 687 BC. Alternatively, this is not a preceding technique, but only its later replica of VII c. BC.
[14] Nowadays, studies have proven that the bronze tools are not only more durable, but depending on manufacturing technology, they could be harder too. Therefore, the transition from the Bronze to the Iron Age is related not so much with quality differences in the metal used, as with the depletion of the copper deposits.
[15] De Lony as well as other authors believe that the first vaulted bridge appeared in Mesopotamia in VI c. BC (De Lony, E. Context for World Henitage Bridges. – In: Ajoint publication with TICCIH, 1996). This is not so, however.
[16] These walls were enlarged and extended upwards multiple times, but large lower parts of the side with the bridge were preserved because they have durable masonry. The footing of the bridge (the so-called Wilson's Arch) is embedded in their structure, while the footing of the second, so-called Robinson's Arch is embedded in the most recent masonry of relatively small-sized squared stone blocks dating to the time of the third temple as part of a later staircase. The fact that it was an upward extension is evident from the fact that the wall blocks have grooves carved to accommodate the wall footing. The first study of the arches was published in 1856 by the American Biblical studies expert Robinson, and later – by Wilson and Warren from the English sapper parts after research and architectural tracing in 1864 – 1865. (Robinson, E. Biblical Recearches in Palestine and the Adjacent Regions. Vol. 3, London, 1856; Wilson, Ch., Ch., Warren. The Recovery of Jerusalem. London, 1871).
[17] Unlike the corbel vaults where the masonry forms the configuration by means of gradual upbuild of layers of squared stone blocks. Therefore, in this case, the load of the whole construction is only vertical.

CALIFORNIA'S TRANSPORTATION INFRASTRUCTURE RENAMED TO HONOR GOVERNMENT PROFESSIONALS

Greg Kaderabek[1], Steve J. Lee[2], Alfred R. Mangus [3]

Professional Engineers in California Government[1,2,3]

Abstract: *The renaming of California's transportation infrastructure after actual government professionals who design, build, or maintain highway or freeway transportation systems, rarely occurs. Today bridges, SRRA (Safety Roadside Rest Areas), vistas, overlooks, tunnels, tubes, and 5-mile sections of freeways and highways are renamed after individuals. Over 24,000 bridges exist in California. A table lists the 48 government professional honorees with the type of facility. The focus of this paper is an executive summary about seven selected facilities plus a brief biography of each honored professional. Harlan D. Miller (1880-1926) was the State Bridge Engineer for the California Highway Commission from 1923-1926. The Dog Creek Bridge was renamed the Harlan D. Miller Memorial Bridge. The Posey Tube, the world's first immersed underwater reinforced concrete vehicular tube opened October 27, 1928 in Oakland, CA. It was named after George Addison Posey, Alameda County Surveyor (1883-1932). Fredrick W. Panhorst (1893-1974) was California State Bridge Engineer from 1931-1960. The Russian Gulch Bridge was renamed to honor Panhorst. The Pine Valley Creek Bridge was renamed the Nello Irwin Greer (1922-2002) Memorial Bridge. Marilyn Jorgenson Reece (1927-2004) was the first woman in California licensed as a civil engineer and the lead design engineer for the San Diego-Santa Monica Interchange in Los Angeles. The Tuolumne River Bridge was renamed after James E. Roberts, a Caltrans bridge engineering leader (1931-2006). The California State Engineer Memorial Interchange was named to honor the 35+ State of California Engineers and Land Surveyors killed in the line of duty.*

Key words: *bridge; California, engineers, freeway, highway, Greer, Interchange, memorial, Miller, Panhorst, Posey, professional, Reece, renaming, Roberts, SRRA (Safety Roadside Rest Area), surveyors, tube, tunnel*

1. Introduction

The renaming of infrastructure after individuals is a recent idea. The renaming of the Miller Bridge was in 1926 followed four years later by the Posey Tube. The renaming happened because they were beloved by their colleagues. Today there are about 700 transportation infrastructures renamed after individuals and or groups of individuals. An act of the California legislature is required to rename a piece of transportation

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infrastructure. There are about 12,500 Caltrans bridges and 12,500 bridges owned by others. Thus, renaming is really a rare occurrence.

Table 1. List of Honorees

#	Individual	Renaming	Life Span	Age	Honoree	Job Title
1	Abarr, William D.	Bridge - No. 05-0006	1934-1983	49	killed	Heavy Equipment Operator
2	Allen, Lowell C.	Bridge - No. 04-0240	1929-2014	85	career	Senior Bridge Engineer
3	Baker, Shawn	Freeway I-5 (5 miles)	1963-2013	50	killed	Equipment Operator 2
4	Henry Edgar Beck, Jr.,	Bridge - No. 04-0072	unknown	unknown	career	Foreman
5	Benedict, Harold "Hod" Wilson	Bridge - No. 57-0633	unknown	unknown	career	Senior Bridge Engineer
6	Binger, Robert L.	Bridge - No. 42-0342L	1931-1997	66	career	District 6 Director
7	Bowers, Dana	Vista Point	unknown	unknown	career	Supervising Landscape Architect
8	Broeske, Daniel	Highway 101 (5 miles)	died 2005	unknown	killed	Trans. Engineering Technician
9	Brown, Delbert A.	Bridge - No. 04-0286	1931-1995	65	career	District 1 Deputy Director
10	Caltrans Workers Memorial Bridge	Bridge - No. 05-0006	Varies	Varies	killed	Varies
11	Combs, Larry	Safety Roadside Rest Area	unknown	unknown	career	Lead worker
12	Harry Crabb	Tunnel No. 19-0079G	alive	alive	career	State Service Manager 2
13	Dekema, Jacob	Freeway I-5 (28.5 miles)	born 1915	100+	career	District 11 Director
14	De Neal, Donna	Bridge - No. 57-0633	1960-1995	35	killed	Equipment Operator
15	Dinsmore, William J. C.	Bridge - No. 04-0129	1877-1951	74	career	Foreman
16	Dunaway, Douglas	Bridge - No. 58-0337	1959-2010	51	career	Senior Bridge Engineer
17	Evanhoe, Michael	Interchange SR-85 + US 101 Bridge - No. 37-0611G	1923-2002	79	career	Executive Director California Transportation Commission

18	Erreca, John Chuck	Safety Roadside Rest Area	1910-1981	71	career	Director of Public Works
19	Feliciano, Michael "Flea"	Highway 101 (5 miles)	died 2004	unknown	killed	Maintenance Lead Worker
20	Forbes, Carlton E.	Freeway I-5 (5 miles)	1929-1980	51	career	Chief Engineer, Caltrans
21	Gonzalez, Richard	Bridge - No. 57-0784	1959-2011	52	killed	Equipment Operator 2
22	Greer, Nello Irwin	Bridge - No. 57-0692	1922-2002	80	career	Senior Transportation Engineer
23	Hart, Alan S.	Freeway I-80 (103 miles)	1907-1994	87	career	District 3 Director
24	Helwer, Sam	Highway 101 (5 miles)	1914-1991	77	career	District 3 Director
25	Hemenway, Bernard A.	Bridge - No. 04-0094	1907-1980	73	career	Maintenance Tree Foreman
26	Hoffman, David	Bridge - No. 57-0487	1957-1987	30	killed	Associate Engineer
27	Huss, Robert Harvey	Bridge - No. 44-0453	died 1975	unknown	killed	Maintenance Worker
28	Jones, Joseph "Robert"	Freeway I-5 (5 miles)	1973-2013	40	killed	Equipment Operator 2
29	Kane, Clyde	Safety Roadside Rest Area	1981	unknown	career	District Engineer
30	Larsen, Harold G.	Vista Point	1925-1972	48	career	District 1 Engineer
31	Leonard, William E.	Interchange I-210 + I-15	1923-2008	85	career	Member California Highway Commission
32	Lucius, William	Highway 101 (5 miles)	1914-2004	90	career	Chair California State Transportation Board
33	Lund, Bert	Overlook	died-1970	unknown	career	Senior District 10 Engineer
34	Meyer, John G.	Overlook	unknown	unknown	career	District 10 Engineer
35	Miller, Harlan D.	Bridge - Abandoned	1880-1926	46	career	State of California Bridge Engineer
36	Obeso, Jaime	Highway 8 (5 miles)	unknown	unknown	killed	Maintenance Lead worker
37	Panhorst, Frederick W.	Bridge - No. 10-0151	1893-1974	81	career	State of California Bridge

						Engineer
38	Posey, George Addison	Tube - No. 33-0106R	1883-1932	49	career	Alameda County Surveyor
39	Raine, Philip S.	Safety Roadside Rest Area	unknown	unknown	career	Chief of the Division of Highways
40	Reece, Marilyn Jorgenson	Interchange	1927-2004	77	career	Senior Transportation Engineer
41	Roberts , James E.	Bridge - No. 32-0018	1931-2006	75	career	Technical Advisor to the Director
42	Smith, Gary Wayne	Highway 99 (5 miles)	died 2010	unknown	killed	Maintenance Lead worker
43	State Engineers Memorial Interchange	Interchange I-5, US50	varies	varies	killed	varies
44	Sturgeon, Vernon I.	Highway 101 (5 miles)	1915-2004	89	career	Chief Deputy Director of the Department of Public Works
45	Thomas, Everett	Highway 101 (5 miles)	1911-2002	91	career	Assistant District Engineer in District 1
46	Van Den Broeke, Roger	Plaque	died 1993	unknown	killed	Equipment Operator
47	Weston, Patricia Ann	Highway 99 (5 miles)	1951-2009	58	career	Transportation Planner
48	Wilkie, John	Safety Roadside Rest Area	unknown	unknown	career	Highway Maintenance Superintendent

2. Survey of the biography and the bridges

2.1. A brief biography of Harland D. Miller

Harlan D. Miller, PE was born May 5, 1880 in Ohio and died October 26, 1926 in Sacramento. He grew up in Santa Barbara and attended the Case School of Applied Science in Cleveland, OH and received his Civil Engineering Degree in 1915 (*17*). Miller worked for a Cleveland contractor. Next he worked for the New York Central Railroad (*18*). He was employed by the State of New York and left after being the chief engineer.
He was a partner of Watson, Davis, and Miller of Cleveland. In 1921, he was hired as Assistant State Bridge Engineer by the California Highway Commission. He was promoted to California State Bridge Engineer (1923–until his death in 1926). He was the first formal California State Bridge Engineer coordinating with AASHTO and other state bridge engineers. His staff was two office engineers, four designers, 12 draftsmen, 10 construction engineers, one cost estimator, and one specification writer. Miller required all highway bridges in California to be designed by government employees, to match the geometry of

the road and emphasized aesthetics. Miller directed his staff to design curved and skewed bridges to be in harmony with the curving highway geometry. A few days prior to his death, the California Highway Commission renamed this bridge to honor his dedication and service to the state of California. Miller was told of this unprecedented honor prior to his death. Harlan wrote a thank you letter to the Board prior to his death.

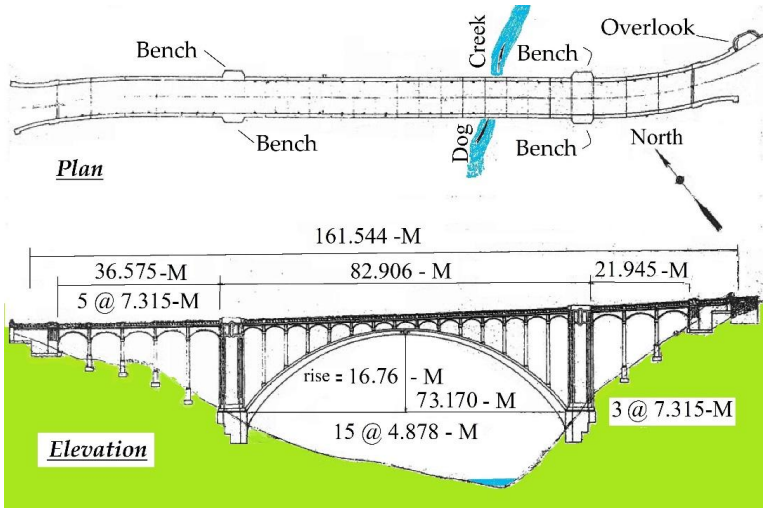


Fig. 1. The Harlan D. Miller Memorial Bridge.

2.2. A brief description of the Harlan D. Miller bridge

The 73.17-m main span open-spandrel arch bridge was design by Harlan located at the mouth of Dog Creek as it flows into the Sacramento River near Vollmers (Figure 1). The rise of the pair of reinforced concrete fixed, parabolic ribbed arches is 16.76 meters. The superstructure of the arch has 15 spans of 4.878 meters to more uniformly distribute the weight. At the ends of the arches are massive concrete piers about 10.7-m wide. This helps resists the horizontal thrust of the arch. The approach structures have spans of 7.315-m to minimize the number of columns plus footings. It is a cast-in-place reinforced concrete bridge built with timber falsework (*10*). The total length is 141.43-m with a deck width of 7.32-m or 2 lanes of traffic. The bridge also has concrete benches inlaid with blue ceramic tiles as part of the barrier system. The primary vehicle at the time of construction were Ford Model-T cars and trucks. Travel speeds were slower and people would stop to picnic on the bridge for views. The resident engineer was Charles E. Andrew, PE (1884-1969) who was promoted State Bridge Engineer, upon Miller's death. The bridge, abandoned in 1957 is adjacent to its replacement 4-lane concrete arch bridge with about 9.656 kilometers of new freeway (*8*). It is about 250 meters east of I-5 (*25*). Many photographers like to post their photos of this abandoned bridge on the internet. The current bridge owner is the US Forest Service and it is both a US and California Historic Landmark, and visitors may walk across it.

2.3. A brief biography of George Addison Posey

George Addison Posey was born in Mexico September 28, 1883. Posey received his BSCE from UC Berkeley in 1906 and was a member of Tau Beta Pi, the honorary fraternity for Civil Engineering Students (*5*). Posey worked for several employers until 1909, when he joined Haviland and Tibbets Engineers of San Francisco. In April 1913, he was appointed Chief Deputy County Surveyor of Alameda County. In 1921, he was appointed County Surveyor of Alameda County due to the death of the county surveyor.

Afterwards he had to run for re-election. Posey was allowed to maintain a consulting

firm until 1927 when the conflict of interests laws were changed, prohibiting the side business. George did land surveying and civil engineering including structures around Oakland, CA. Posey designed the Liemert Concrete Arch Bridge, which was built for the Liemert Brothers who developed the Oakmore Highlands in Oakland. Posey's most important infrastructure contribution was the concrete tube. It was the first immersed tube built out of reinforced concrete**.** Posey committed suicide on August 3, 1932 even though he only had indirect involvement on a free lot scheme (Posey did some land surveying for the convicted offenders.). In 1932, the community felt that that this vital tube should be renamed after Posey. Caltrans purchased this tube.

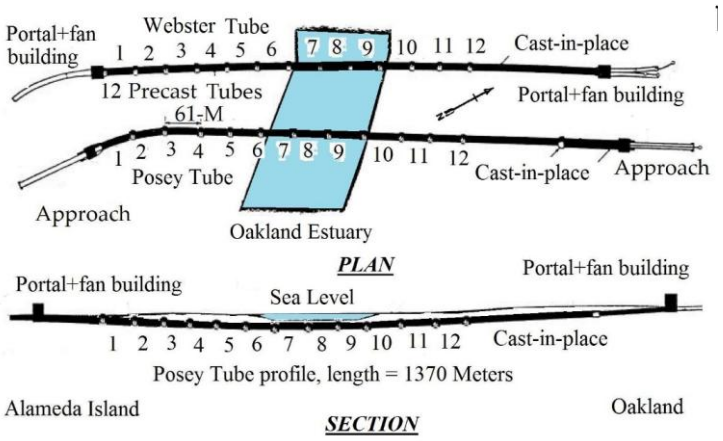


Fig. 2. The George A. Posey Tube site plan.

2.4. A brief description of the George Addison Posey tube

Alameda was originally a peninsula but was converted to an island via dredging. The inner harbor became a very important location for cargo facilities serving Oakland and California. Alameda became an important residential community served by the Webster Street Swing Bridge. There was much interruption and conflict between automobiles and cargo vessels. Posey and his team selected a concrete immersed tube as the best solution to separate both vital transportation needs (Figure 2). The Posey Tube is a circular reinforced concrete immersed underwater tube linking Oakland to Alameda Island (*4*). Posey used the ventilation system developed for New York City's Holland Tunnel (Figure 3). The Holland Tunnel was renamed after the Project Engineer, Clifford Holland, who died during the project. The reinforced concrete tube was prefabricated and lowered into place after dredging (*7*). The fans are encapsulated by an art deco façade building (*27*). The Prelinger Archives www.Prelinger.com include a black and white silent documentary clip of the first section of the tube being floated from Hunters Point across San Francisco Bay to its location. The Posey Tube opened October 27, 1928, and the swing-bridge was removed. This tube was renamed after Posey's death in 1932. It provided two-way traffic until the 1963 opening of the copied Webster Tube by Caltrans. Due to the 1989 Loma Prieta Earthquake, both tubes leaked sea water at the joints between tube sections. Eventually, the tubes were seismically retrofitted (*2*). The Posey Tube is now used only for two lanes northbound traffic. An average of 86 people per day used the narrow sidewalks in 2006.

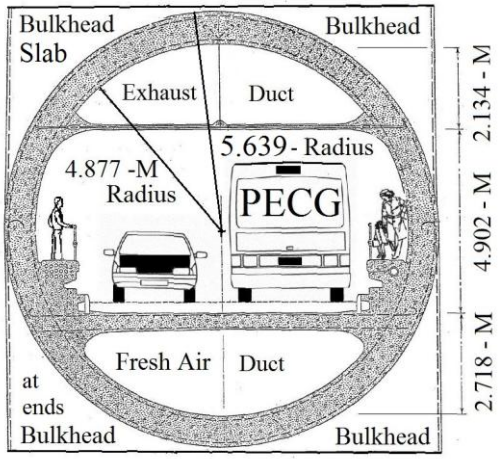


Fig. 3. The George A. Posey Tube cross-section.

2.5. A brief biography of Frederick W. Panhorst

Frederick W. Panhorst was born 1893 in Mexico and died 1974 in Sacramento. He grew up in Illinois and received his BSCE in 1915 from the University of Illinois, Urbana (*23*). He worked as a civil engineer in Missouri, Kansas, Texas, and Washington. He was hired as resident bridge engineer in 1921 by the California Highway Department. He rose through the ranks and was acting State Bridge Engineer in 1931(*12*) (*13*). Charles Andrew, the 2nd California State Bridge Engineer was transferred to manage the San Francisco Oakland Bay Bridge. Panhorst supervised a staff of 114 employees in 1931. Frederick became the 3rd California State Bridge Engineer. Important bridges completed under his supervision were Bixby Creek (1931), Russian Gulch (1940), Klamath River at Orleans (1940), Pit River at Shasta Lake (1941), Noyo River (1948), Four Level Interchange in Los Angeles (1949), Arroyo Seco Arch (1953), Carquinez Steel Truss (1958) and East Los Angeles Interchange (1960). There were 1,000 employees under his supervision when he retired in 1960. California State Resolution Number 75 passed March 9, 1960 to thank him for his 40 years of state service. The Russian Gulch Bridge was renamed the Frederick W. Panhorst Memorial Bridge in 1974. The California Senate renamed the Bridge via concurrent Resolution 145. Panhorst was Chief of the Bridge Section of the California Division of Highways from 1931 to 1960, and served the longest tenure as California State Bridge Engineer. Frederick also served as an officer in the American Society of Civil Engineers (ASCE) (*19*). Panhorst donated his professional papers to his alma mater the University of Illinois, Urbana.



Fig. 4. View of the Frederick Panhorst Memorial Bridge from the Pacific Ocean.

2.6. A brief description of the Frederick W. Panhorst Bridge

This 2-lane bridge crosses the Russian Gulch and is part of Highway 1 that follows the California coastline. It replaced a timber trestle bridge completed in 1910 about 150 meters upstream. The timber bridge was designed for a six-horse team, since trucks and autos were rare at the time of design. A new roadway alignment of about 850 meters was required, and it allowed for faster vehicle speeds. The bridge was designed by Henry E. Kuphal. Its reinforced concrete open spandrel arch main span is 73.2 m long, and its rise of arch is 25.9 meters. The bridge's total length is 160.6 m (Figure 4). It is a cast-in-place reinforced concrete bridge built with timber falsework. This carries an average of 10,600 vehicles per day. This bridge cost $109,000 and was constructed 1939-1940. The contractor was R. G. Clifford and the resident engineer was George A. Greene. This bridge had stronger rock to resist the thrust of the arch forces. Thus, all of the bridge columns have similar shape. It is interesting to compare this design to the Miller Bridge, a similar sized structure. Both bridges have a superstructure divided into fifteen spans above the arch. The Russian Gulch Bridge was dedicated by California Governor Culbert Olson on June 9, 1940. The Panhorst Memorial Bridge is a beautiful concrete arch bridge at a spectacular California coastal site (*15*). The FHWA refers to this today as a "contextual solution" and this bridge is well photographed with images posted on the internet.

2.7. A brief biography of Nello Irwin Greer

Nello Irwin Greer was born March 9, 1922, in Phoenix, AZ and died August 15, 2002. He was residing in Victorville, CA for a few years, then moved to St. Johns, AZ where he spent his formative years. Greer fought during World War II and served under General George S. Patton, Jr. at the Battle of the Bulge (*6*). Greer began his career with the Department of Transportation (Caltrans) in 1948, advancing to an engineering position and working on many state freeway projects in San Bernardino. In 1960, Greer moved to District 11 in San Diego, where he lived until retiring from Caltrans in 1977. Some of his projects include State Highway Route 163 from State Highway Route 8 north to the Genesee exit, the design of the first bridge on Friars Road with 13 lanes of traffic, as well as various other projects throughout the county. The Pine Valley Project was by far the most important of Greer's infrastructure projects. Originally, Caltrans wanted the project's freeway to run through the town of Pine Valley, CA. Overcoming numerous obstacles to complete it, Greer moved the project to the location where it currently exists. The construction of the Pine Valley Bridge received many engineering awards. Senator Hollingsworth was the author of the resolution to rename this bridge (*6*).

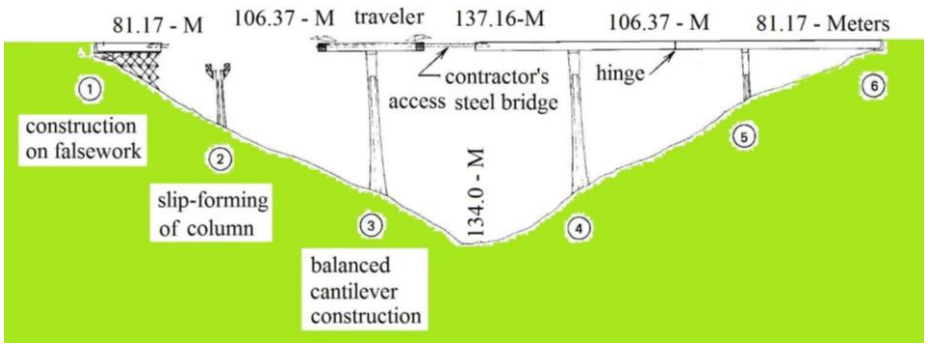


Fig. 5. The Nello Greer Memorial Bridge.

2.8. A brief decription of the Nello Irwin Greer Memorial Bridge

This bridge was the first segmental concrete bridge of either precast or cast-in-place concrete, designed and constructed in the USA (Figure 5) (*14*) (*15*) (*16*). European designs were studied and the goal was to minimize falsework. The main columns were slip formed then the superstructure was built from the columns. Bridge aesthetics was important and studied in great detail. Thus, scale models were built. The superstructure is split into two lanes of traffic in each direction separated by a gap. This gap provides space to widen the bridge to three lanes in each direction. The California Department of Transportation produced a short movie of 5 minutes for the general public explaining how the natural valley was minimally disturbed with the segmental method (*28*). The FHWA refers this bridge as a "contextual solution." A longer documentary of 20 minutes was entitled "*The Bridge*." It features narration by Caltrans Division of Structures staff, with short discussions on the topic of bridge design by Bert Bezzone and Ostap "Joe" Bender (*9*). Bridge architecture is discussed by Bill Wells and bridge construction by Caltrans Resident Engineer Ralph Sommariva. Film credits recognize Lead Highway Designers Norm Larsen, Nello Irwin Greer, and Highway Resident Engineer Tom Drake of Caltrans. The General Contractor was SJ Groves of New York City. The prestressing contractor was Dyckerhof and Widmann, Inc. (Dywdag), Munich, Germany. The post-tensioning design of the superstructure was by Dr. M.C. Tang who was an employee of the post-tension company Dywdag.

2.9. A brief biography of James E. Roberts

James Roberts was born November 30, 1930 in Missouri and died July 6, 2006 in Sacramento (*1*). Roberts' family moved to California when he was a child. His father was a carpenter and later became a contractor. As a UC Berkeley student, in 1951, obtained a position as an engineer's assistant to the California Department of Highways (CDOH). He graduated from UC Berkeley and was commissioned as a First Lieutenant in June 1953. Mr. Roberts was called up by the military to serve in Korea, in the winter of 1953, as a US Army platoon leader in a civil engineering battalion. In 1955, after 2-years active duty, he returned to California to continue his employment with the CDOH and worked in the area of bridge construction. Roberts was a Bridge Resident Engineer 1955-1965, Bridge Designer 1966-1967, Bridge Design Section Supervisor 1967-1971, Administrative Assistant to the Deputy Director 1972-1975, Chief Engineer of the Equipment Division 1976-1981, Sacramento Light Rail Transit Project Director 1981-1985, Chief of the Structure Design Office 1985-1987, Chief of the Division of Structures 1987-1994 and Chief Engineer and Director of Engineering Services (mainly bridge engineering, about

2,300 employees) 1994-1999. Caltrans had about 1,000 bridge engineers when Roberts served as California's State Bridge Engineer. His final position was Technical Advisor (primarily Bridge Engineering issues) to the Director for the California Department of Transportation. In 1972, Roberts served as State President of PECG (Professional Engineers in California Government). He retired as a colonel in the US Army Reserves in 1985 and from Caltrans in 2003.

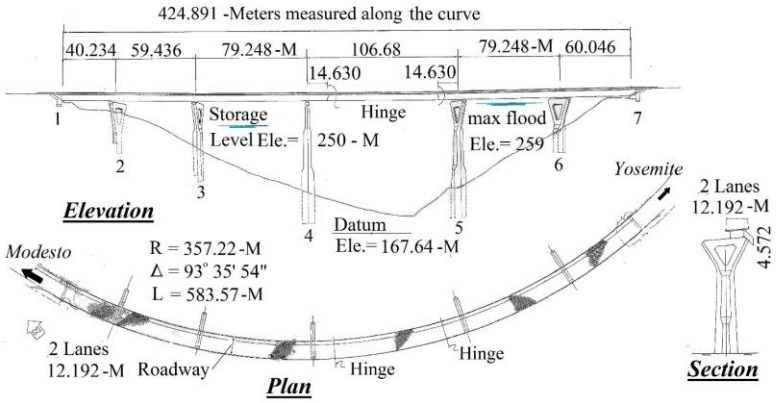


Fig. 6. The James E. Roberts Memorial Bridge.

2.10. A brief description of the James E. Roberts Memorial Bridge

The purpose of this bridge was a new Highway 120 alignment made necessary by the construction of the Don Pedro Dam. Roberts planned the Tuolumne River Bridge on a horizontal curve with a 357-m radius curve to meet the site requirements in December 1967 (Figure 6). Richard "Dick" Dokken was lead design engineer, and aesthetics were by W.S. Ludow. Plans, Specifications, and Estimates (PS&E) were completed in November 1968. Resident Engineer for Bridge Construction was Herb Jensen, with John F. Harrington, as assistant Resident Engineer. Conventional concrete was used for construction of the foundations and the hexagonal columns and lightweight concrete was used for the bridge deck. The superstructure was high strength curved steel girders. The structure was designed to be widened from 2-lanes of traffic to 4-lanes of traffic. The general contractor built switchbacks into the canyon walls, which would be submerged by the lake. Next, the steel girders were erected. Final girder pieces were lifted from the canyon floor and placed upon the cantilever tips of the north and south girders. The bridge opened to traffic on time and on schedule in 1971 as coordinated with the Don Pedro Dam completion. The bridge designers received a welding award with a cash prize in 1972 (*3*). The American Institute of Steel Construction (AISC) presented another award in the medium span category. The AISC Jurors wrote "This *gracefully sweeping bridge fits beautiful into its setting. The clean curving superstructure and the sculptured piers are very pleasing*" (*24*).

2.11. A brief biography of Marilyn Jorgenson Reece

Marilyn Jorgenson was a South Dakota native who earned a bachelor's degree in civil engineering from the University of Minnesota in 1948 (*11*). Marilyn moved to Los Angeles shortly after graduation. The same year, she went to work for the California Division of Highways (later became Caltrans) as a junior civil engineer in Los Angeles. In 1954, after six years of required experience to sit for the Professional Engineer's Exam, Marilyn became the state's first licensed female civil engineer (*22*). In 1957 she married Alvin Reece, a civil engineering graduate of the University of Colorado who also worked for the Division of Highways in Los Angeles.

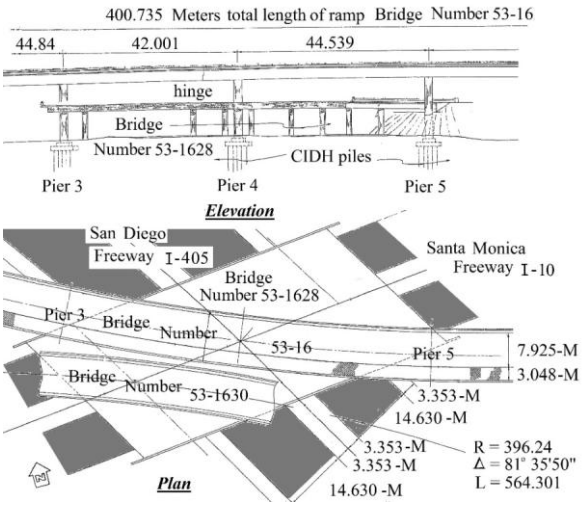


Fig. 7. The Marilyn Jorgenson Reece Memorial Interchange.

In 1962, she received the Governor's Design Excellence Award from Governor Pat Brown for the San Diego-Santa Monica freeway interchange. Shortly after, Reece became the Division of Highway's first woman resident engineer for construction projects. During her 35-year career, Reece served as senior engineer for the completion of the 210 Freeway through Sunland in 1975; at the time, the largest construction project Caltrans had ever awarded at $40 million. Reece taught engineering classes at CSU Long Beach after retiring in 1983. During Women's History Month in 1983, the Los Angeles City Council honored Reece for making significant contributions to the city. Senator Debora Bowen authored Senate Concurrent Resolution 72 renaming this series of bridges or the I-405/I-10 Interchange in honor of Reece (Figure 7). Kirsten Stahl followed her parents, becoming a civil engineer for Caltrans.

2.12. A brief description of Marilyn Jorgenson Reece Interchange

The interchange is three levels and basically symmetrical. The north-south freeway I-405 is aligned to the northwest, so the interchange is slightly skewed. The 3-level Santa Monica-San Diego freeway interchange, which opened in 1964, was the first interchange designed in California by a woman engineer. Ms. Reece told *The Los Angeles Times* in 1995 that she put her "heart and soul into it" and that she designed the interchange with aesthetics in mind which was reprinted in 2004 (*11*). Ms. Reece said, "It is very airy. It isn't a cluttered, loopy thing," she said, adding that specifications to keep traffic moving at high speeds necessitated the long, sweeping curves. "That was so you didn't have to slam on the brakes, like you do on some interchanges." Urban critic Reyner Banham, author of "Los Angeles: The Architecture of Four Ecologies," admired the "wide-swinging curved ramps" connecting the two freeways. "It is more customary to praise the famous four-level (interchange in downtown Los Angeles);" he wrote, but the I-10/405 Interchange "is a work of art, both as a pattern on the map, as a monument against the sky, and as a kinetic experience as one sweeps through it," (Figure 7). The structural engineering was performed by the Bridge Department in Sacramento. The bridges are post-tensioned multi-celled prestressed concrete that were cast-in-place and built on falsework. This interchange was undamaged during the 1994 Northridge Earthquake. It was seismically retrofitted to increase its robustness.

2.13. A brief description of the California State Engineer Memorial Interchange

The I-5/US 50 Interchange in Sacramento was renamed the California State Engineer Memorial Interchange in 2012 (Figure 8). State of California Assembly Member Bonnie Lowenthal authored Assembly Concurrent Resolution 100. This resolution renaming this interchange to honor the 35+ state government engineers and related professionals who died in the line of duty. We are very sad that, during the creation of this paper that Caltrans was devastated by the death of Structures Construction employee Mr. Oscar Vargas. He died, when he lost control of his truck while working a night at a project along I-8 near Imperial County. California State flags were flown at half-staff in memoriam. Mr. Vargas, 54, served the people of the state of California for the past 29 years, most recently as a Transportation Civil Engineer. He worked on many of the bridges throughout San Diego and Imperial counties, including the iconic San Diego-Coronado Bridge and the historic Cabrillo Bridge in Balboa Park. His extensive knowledge and willingness to share will continue to live on as he helped train many of our current bridge engineers. He was a friendly man with countless co-workers and peers across many organizations that knew him and will miss him. He is survived by his wife, Teresa, and two adult children, Oskar and Christian.

The publically listed employees as of 2015 (*26*) are by name, title (year killed): A Achtert, Assistant Resident Engineer (1924), Albert B. Castro, Surveyor (1931), Fred Balsz, Surveyor (1934), Raymond Buland, Assistant Highway Engineer (1935), Charles Pope, Construction Engineer (1938), Frank Wesley Poast, Assistant Highway Engineer (1948), Alan L. Robbins, Surveyor (1953), Walter Howard Johnson, Assistant Bridge Engineer (1956), Lyle E. Bilyue, Assistant Highway Engineer (1962), Frank James Smith Assistant Highway Engineer (1962), Arnold H. Lenox, Surveyor (1963), Donald W. Anderson, Assistant Engineer (1964), Robert Adams, Associate Engineer (1965), Leroy Exum, Construction (1965), Bruce Cole, Assistant Bridge Engineer (1966), Richard McQuillan, Surveyor (1968), William McLaughlin, Assistant Highway Engineer (1969), Thomas R. Brungardt, Construction (1970), Jerry Corey, Surveyor (1971), Carl P. Tardito (1971), Emory Price, Highway Resident Engineer (1973), David R. Tobey, Engineering Student Assistant (1978), Daniel C. Wong, Engineering Geologist (1979), James C. Esola Jr., Assistant Transportation Engineer (1981), Roland V. Keller, Assistant Transportation Engineer (1983), Byron Works, Engineering Geologist (1984), David A. Hoffman, Associate Engineer, Tsutomu (Ben) Tani, Assistant Transportation Engineer (1989), Oscar Lanatta, Civil Engineer (1990), Robert J. Essenpreis, Transportation Engineer, Civil (1993), Hemul Patel, PE, Transportation Engineer, Civil (1998), Moul Duc Tran, PE, Transportation Engineer, Civil (2008) (Unknown, 2014) and Oscar Vargas (2015).

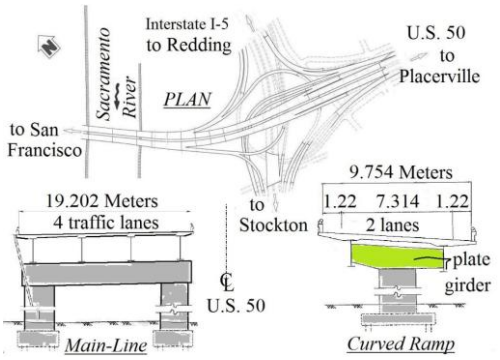


Fig. 8. The CA State Engineers Memorial Interchange.

2.14. A brief description of the State Engineer Memorial Interchange

This interchange merges Business 80 with I-5 and US-50 on the east banks of the Sacramento River in Sacramento. Highway US 50 runs east towards the Sierra Nevada Mountains to South Lake Tahoe. It was part of the first Interstate built within the city limits of Sacramento and carries an average of 190,600 vehicles per day. I-5 is the north-south freeway and parallels the east banks of the Sacramento River. The east-west freeway starts from the west as I-80 from San Francisco and becomes US 50 inside the city limits of Sacramento. The series of bridges or interchange was designed by various types of state engineers, (Traffic, Highway, Bridge, Electrical, and Mechanical). The series of bridges or ramps comprised of steel plate girders with reinforced concrete decks, replaced city streets and stop lights. The main line crossing of Business 80 over I-5 consists of four traffic lanes supported by four steel plate girders with a reinforced concrete deck. The various exit and entrance ramps are curved two steel plate girders with a reinforced concrete deck.

3. Conclusion

The renaming of infrastructure after individuals is a rare occurrence. The renaming of transportation infrastructure after government professionals indeed happens even less. Many of us take transportation granted because we expect roads and bridges to be safe and reliable for us when we travel. Thus, a reliable transportation system is vital for our quality of life. Those who worked on our highways and bridges should be recognized and honored because they are the true heroes of our seamless and reliable transportation network. The 13,000 engineers, surveyors and related engineering professionals who work for the State of California have one vision in mind when they design and inspect our infrastructure; the safety of Californians and everyone who visits our Golden State. The Professional Engineers in California Government (PECG, www.pecg.org) shares this vision along with developing clean energy and green technology.

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THE HISTORY OF THE TIMBER ARCH SYSTEM, DEVELOPMENT BY PHILIBERT DE L'ORME, AND ITS APPLICTION IN THE CONTEPORARY STRUCTURES

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***Abstract**: The present paper attempt to make analysis of several examples of wooden structures built, using the system invented by Philibert de l'Orme in 1561. In the field of timber structures his system is known as especially arched systems with mechanical joints. The case studies in the paper illustrate the different ways in which the construction system proposed by de l'Orme can be used.*

***Key words:** wooden structures, timber arch systems, mechanical joints,P. de l'Orme*

1. Introduction

According M. R. Campa[1,2], the treatise "*Le nouvelles inventions pour bien bastir et a petits fraiz*" by Philibert de l'Orme (1514-1570), was published for the first time in 1561 and then inserted towards the back of the "Premier Tome de l'Architecture" in 1568 (Fig. 1)[4,5,6]. It is a milestone in the history of wood inventions as it contains different conceptions of how wood can be used. Anyone who wishes to study wooden roofing, has to consider the theories of this French architect, according[1,2].

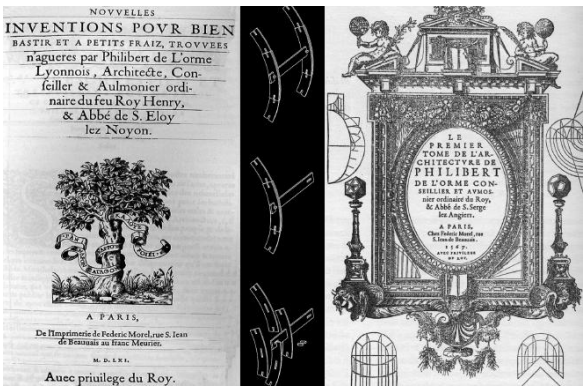


Fig.1. Frontispieces of Ph. de l'Orme's treatises and the components of the invention[1,2].

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In an historical context, Philibert de l'Orme was more than just an architect, he was also a counsellor and chaplain to the King Henry II (1547-1559), which he pointed out in all his work[1]. The roof vaults described by Philibert de l'Orme are always of the same modular basis, which are adaptable to form different shapes and sizes. He describes a twin series of shaped wooden boards, put together in a staggered way to give greater resistance to the roof structure, forming the curve of the arch. Every piece of wood has a hole in the centre to allow the insertion of crossbars that bind the arches together. The crossbars are then fixed to the boards by some wooden wedges, called "keys"(Fig. 2). This base module changes according to the forms and dimensions of the vault. For example, wider spans subsequently require additional variations of the dimensions of each component. De l'Orme suggests the adding of crossbars to the sides of the boards to ensure that they will be adequately formed. Another element, which varies in relation to the dimension, is the "rafter end", which is the piece that links the vault to the building. This is used to make the positioning of the tiles easy, and consequently, better water drainage(Fig. 3)[1,2].



Fig. 2. Ph. de l'Orme's vault

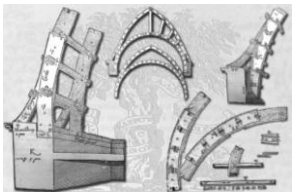


Fig. 3. "Rafter end", lateral crossbars and components

As these variations are, in the main, the sizes and the number of connections between the components parts, the module does not really vary from the original shape. It is only changed to adapt according to new production processes(Fig. 4). The carpentries built in this way are used to construct roofs or floors: the latter are treated in the second book of the treatise (Fig. 5)[1,2].

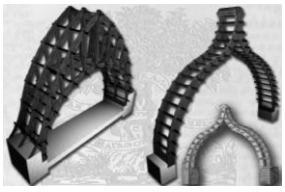


Fig. 4. Different types of vaults

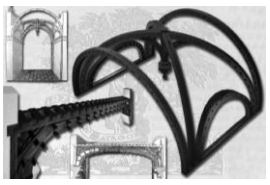


Fig. 5. Ph. de l'Orme's roofs and floors[1,2]

With regard to roofs, de l'Orme illustrates the different types of vaults that can be constructed according to the rooms that need to be roofed. The project for a royal basilica was intended to show the potentialities of his small piece carpentry (Fig. 6)[1,2].

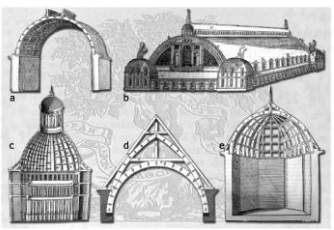


Fig. 6. Different examples of roofs produced by Ph. de l'Orme: at Anet Castle (a); in the projects for a royal basilica (b) and a dormitory; at the Tuileries (d) and at Saint-Germain en Laye (e)[1,2].

With the aim to prove, according[1], the advantage of the invention of the his system**,** Ph. de l'Orme claimed[4,5,6], that it was necessary to create a new technique in order to realize wooden vaults, with the main advantage being that the problem of raw materials is sharply reduced.

2. The influence of the De L'ORME System on the following Architecture in Europa

To understand the extent to which de l'Orme carpentry has influenced architecture since 1561[4,5,6], it is imperative to first consider the life of the French architect, because the success of a literary or architectural work often depends on the personal events of the author's life[1]. After Henry II's death, François II (1559-1560) became king and de l'Orme was no longer high in the Monarchy's favour. He lost his office as royal architect, being replaced by Primaticcio, and was compelled to return to Rome. Moreover he found he had accumulated a lot of enemies during the period spent at the royal court and this impeded his return to life at court (de l'Orme 1561, letter to the reader). For this reason de l'Orme's works and ideas were not spread in an appropriate way. The "Nouvelles Inventions" were published during the heights of renaissance age, when all Europe was full of the rebirth of classical culture. It was the period of Vitruvio's and Leon Battista Alberti's studies, and the period in which the rigour of Palladio's architecture started to be successful, before spreading all over Europe. It was not really the right period to encourage the success of a building technique that was more reminiscent of the wooden roofs of the medieval churches, than of the domes by Michelangelo and Brunelleschi! To demonstrate that de l'Orme's treatise had no *immediate* success. The French treatise wasn't rediscovered until the nineteenth century, during the renewed interest in the gothic when illustrious people, such as Jean Baptiste Rondelet and Armand Rose Emy[1], referred to it in their works. Then other famous architects, such as Etienne Louis Boullée, actually referred to him.

2.1. The Influence of de l'Orme system in Italy

Italy with its deep-rooted attachment to the classical models is the less suitable country to accept a completely different cultural tradition[1]. As a matter of fact in Italy there are no traces of the "Nouvelles Inventions" in architecture. And, with regard to the treatment of this work after the Renaissance, there is no evidence of its consideration. This is because the seventeenth century in Italy was the century of Baroque style: a style of excesses, expedients, decorations more than structure. To have some references to the French building techniques we have to wait for the latter half of the eighteenth century, the Enlightenment age. In the Tuscany area a generation of enlightened architects emerged, that was linked to a new culture: the so called "Scienza Nova" by Galileo's appellation. Their principles present a new idea of architecture, which is based on solid resistance and static structure. A lot of these architects, at that time, were entrusted by the Lords to produce some engineering works. In another chapter, we read some technical notes that reflect an interest in the de l'Orme building technique. *It means that in Italy, there was undoubtedly a spreading of his theories, but without any real interest.* There was only a cultural exchange between Italy and France. By the end of the eighteenth century that De l'Orme as a man and as an architect had been rediscovered thanks to the building of the roof of Halle aux Blés. The Halle aux Blés was made according to the principles of carpentry in small pieces. All this happened in 1782, a clear fifteen years before Del Rosso[3] published his treatise. Considering that the Florentine architect worked to spread the French theories in Italy, he must surely have known of de l'Orme, and it is therefore unclear, even implausible, that he never succeeded in hearing of the "Nouvelles inventions". When his work is analysed, it is clear that there are many common points with

de l'Orme's works. We can affirm that "Della facile costruzione…" is not just a treatise, but it is a handbook. A practical guide for building that doesn't intend to deal with everything about architecture or to dissert on the philosophical problems linked to it. But it intends only to give practical instruction on a well-defined topic, without taking into consideration any impressive projects. The *invention* considered by Del Rosso[3] is more substantially an archetype of lamellar wood building. In fact, *they match precisely* de l'Orme's boards that instead of being put on their edge, are laid-out flat, and linked together by iron fasteners. (Del Rosso 1797, p. 11-12) **(**Fig. 7)[1]. An innovative element that contrasts to the French *invention*, in the technique described by Del Rosso, is the *tapering* of the structure towards the centre. The comparison of the two books highlights various aspects of the work when analysed. First of all, it is immediately clear that Del Rosso's aim was to give an easy instrument that can be also used by a layman, i.e. not fully qualified architects, builders and carpenters. Anyone could apply his *invention* (is not necessary to underline the same aims of de l'Orme as it was also not only aimed at qualified architects). Even the usefulness of the project reflects the Frenchman as his forerunner.

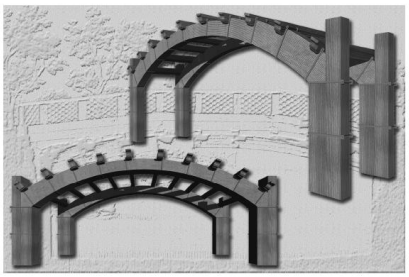


Fig. 7. The *invention* of Del Rosso.

The main stated advantage points are, likewise, the easy availability of raw material and the saving in making it (Del Rosso 1797, p. 12, 14)[1]. Convinced of the principle according to which a building can be realized by anyone in any context, the author simplifies the rules of the building (1797, p. 15). Like de l'Orme, Del Rosso also takes great pride in the strength of his *invention* (1797, p. 6, 12), and, to give more credibility to his projects, he refers to his experience (1797, p. 13). It is true to say that Del Rosso is ahead compared to de l'Orme in the development of lamellar wood (see the affinity with Emy's patent). But he follows the French architect, in all of the architectural aspects, and wood comes back from being just a poor material, used only to realize something without artistic value, *a simple country thing*.

Analysis shows that de l'Orme's treatise had a great influence because he introduced a conception of architecture as an art for everyone, no-longer only for qualified people and for this reason it was necessary to adopt a new language far away from that of Alberti's, which was clearly too theoretical. Generally, Del Rosso took this theme to extremes and his book looks and reads like a handbook more than a treatise on Architecture.

2.2. Influence of the De l'Orme's system in France

Unlike Italy, the cultural and social French world was surely a more successful field for the spreading and the development of the theories of De l'Orme[1]. Although, the Italian culture and techniques had been the starting point of the "Nouvelles inventions", the revision of these theories was without doubt influenced *by the gothic spirit belonging to French culture.* Of course, it was even more evident in architecture, in which gothic

culture never stopped having an influence even though only on style or only on critical aspects of the artists. In this way, many critics refer to a survival of the gothic style, in spite of the new classical wave of the Renaissance. If the architecture described by de l'Orme in the "Nouvelles inventions" is analysed, it is evident that it has a clear middle age style: in forms, in aesthetic features, and in the structural aspects, because he had judged positively this building technique. De l'Orme's comments about the gothic structure were made back in 1634 by François Derand in "L'architecture des voutes, ou l'art des traits et coupes des voutes" and, through this work, they influenced all architecture of the seventeenth century. Derand's work was then published again in the eighteenth century by Amédée François Frézier encouraging, in this way, the spread of de l'Orme's theories. To show the gothic style in France during the Renaissance and Baroque Era, it is necessary to refer to a text by Jean François Felibien written in 1699 "Dissertation touchant l'architecture antique et l'architecture gothique" in which gothic works are criticized regarding dimensions, proportions, structural aspects. And also to a text written by the abbot J. L. Cordemoy in 1702 "Memoires critiques d'architecture" in which he judges positively the building aspects of gothic styles but not aesthetic features. Amédée François Frézier agrees with Cordemoy, and, in his "Traité de stereotomie à l'usage de l'architecture" in 1739, praises gothic style for its precision and for the accuracy of the vaults technique, but not for the aesthetic features. Therefore, at the beginning of the eighteenth century the gothic architecture was appreciated for its structural and functional aspects but not really for its aesthetic features. Not until the end of the eighteenth century, was the gothic style rediscovered and with it De l'Orme's personality and work revaluated.

2.3. De L'Orme in France from the late sixteenth century to the end of the seventeenth century

According[1] between de l'Orme's death and the end of the eighteenth century we know that he was not forgotten in French academic world, but it is not known if his technique was applied in building. In the information from the land register started during Napoleonic age in 1809, we find references to some buildings, mainly houses, made according de l'Orme's technique. This kind of building was quite fragile and breakable: so it is likely that the structures were actually more in quantity, but only a small percentage survived. The reason for the small amount remaining is that, as told in the treatise, the *invention* spread through the workers in poor places, whose buildings were less durable. As the same author says, the *invention* spread not only in France but also abroad, but not enough information exists to clearly show that examples of these buildings are located only in France. An important example that demonstrates that de l'Orme's *invention* was known abroad is the church of La compagnia di Gesù in Cordoba, Argentina. Built starting in 1650, it is part of a bigger building that included the university and the monastery (Fig. 8).



Fig. 8. The church of La compagnia di Gesù

Among the most interesting elements of the building are without doubt the vault and the dome: made to the relevant size and form and, obviously, made of wood[1]. The person *charge* of the project was the Belgian Father Felipe Lemer, who had spent many years in Europe working in a shipyard and this experience was fundamental for the conception of the work. The considerable dimension of spans and the impossibility to use a roof of a pressing type were the ideal conditions to apply the de l'Orme's carpentries considering also the lack of qualified workers at disposal. The result that can be seen, even today, is a product that surely originated from the French *invention*. However, it is a little more advanced, for example, in the tri-dimensionality of the work, in contrast with the bi-dimensionality of the French one: [1] the great difference between de l'Orme and Lemer is that de l'Orme conceives the structure statically and in its surface, on the contrary Lemer conceives it dynamically in the space. To explain it better, if we imagine to overturn de l'Orme's structures we destroy them, on the contrary Lemer's structure can be overturned, inclined and moved and it always remains unbroken. So for this reason we can defy it a roof keel overturned just for the conception that reminds to a structure resistant to any stress, while if it was only a formal reference to the keel Lemer's extraordinary work would be underestimated(Laner 2001, p. 15-16)[1]. The work just described demonstrates that de l'Orme's *invention* had a great diffusion, but a very different one from that expected by the French architect: this diffusion was not just through the academic world but also through the world of craftsmanship.

2.4. The rediscovery of de l'Orme in France at the end of the eighteenth century

According[1] the revival of the gothic style at the end of the eighteenth century produced the rediscovery of the "Nouvelles inventions" applied to all architectural fields. In particular two events contributed to the rediscovery of de l'Orme building technique: the great attention paid to the military during the revolutionary age and, above all, the building of the Halle aux blés in 1783. After French revolution of 1789, episodes of war broke out continually, and it was necessary to find great open spaces to let the cavalry practice all year round. Engineers, looking for ways to produce big spans, consulted the old books about architecture so rediscovering de l'Orme's building technique. The project revised by General Detournelle following the guidelines of carpentry of the sixteenth century, but published in the nineteenth century, was the point of reference for the building of the riding schools of the First Empire. The most important of these, with 26 metres of span, was in Rennes, at Carrousel quarter, but it has been destroyed. However, two roofs of this kind still exist. The first is the Royal riding school in Saint-Germain en Laye with 8 metres span, and the second one in Fontainebleau, at Sénarmont, with 22 metres of span. The

barracks Rochambeau, at Mont-Dauphin (Hautes Alpes), built between 1819 and 1823 based on Captain Massillon's project according to de l'Orme's design also deserves mentioning. Following the barracks model, all the country areas were soon filled with barns and farms, built in de l'Orme's technique, during the nineteenth century. These building required a great deal of light, and as they couldn't use a lot of materials, the building technique using small pieces satisfy all these needs. The buildings still standing now, in France, like the Grandlhac and Serc farms, both in Chanac, on the Causse de Souveterre, where these buildings are used as barns and sheepfolds. At first these buildings were constructed with double vaults, as testified by the considerable thickness of the walls downstairs. The superior vault has been rebuilt, without doubt, using de l'Orme's carpentry, because it offered a cheaper solution compared to the rebuilding of the vault. Located at the entrance of Mende, the "Grange de l'Adoration" has the double function of house and barn. Its dimensions are 20.5 m x 7.5 m and all the arches are composed by three wooden boards layered together. It's interesting to consider what kind of wood was used for the building, as it was a type of poplar tree called "carolin". This tree grows near the river Lot, where the predominant winds give it a natural bending structure. To creating the bend that produces the curved board that form the arches of de l'Orme's structure is costly in time and materials. However, this natural bend becomes an advantage because it doesn't cause the waste of material that you would have cutting a curved piece from a straight board. Moreover, if you cut curved pieces from a straight board, you cut towards the contrary direction of the woods grain, weakening it and producing less resistant pieces (Fig. 9).

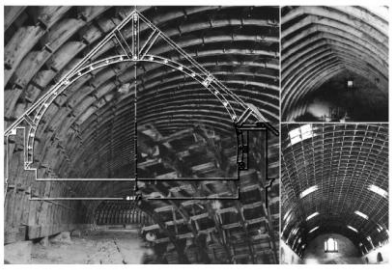


Fig. 9. The barracks Rochambeau, the "Grange de l' Adoration" and the riding school of Fontainebleau[1]

According[1] the building analysed so far have been part of architecture that is considered *minor* because it used low cost material, giving more importance to the *utilitas* than to *venustas*. To appreciate de l'Orme's *invention* through building with a clear aesthetic value it was necessary for it to be used in a more public context where it might be seen by a the public at large. Such an occasion was offered by the building of the Halle au Blé dome in 1783 by Jean Guillaume Legrand and Jacques Molinos. The building had been erected between 1763 and 1769 by Camus and Mézières and it needed a roof that covered a span of 39 metres. For this reason, the two architects thought it best to revive the carpentry illustrated in the "Nouvelles inventions", constructing a dome with wooden arches joined together at the top by a glass lantern (Fig. 10). The vault was made more spectacular by glass bands inserted inside the roof so producing an extraordinary lightness. The Halle aux blés became the best example of the qualities of de l'Orme's carpentry: the lightness and the great building potential. With Halle aux blés, the popularity of the *master builder successor of the middle agse* spirit grew enormously. In fact, at the end of the eighteenth century many architects were inspired by de l'Orme and mentioned him in their treatises. Among them were the more famous Etienne Louis Boullée and Jean Baptiste Rondelet. Boullée used the small pieces carpentry in some of his most famous works such as in the

dome for the Royal Library, a project of the 1785, and the Hall of the Royal Lottery in 1788. These buildings utilise all the potentialities given by the *invention*. As evident from the stateliness of the rooms, although in a different way from Halle au blé, it doesn't keep the carpentry in view, but it is covered with lacunars ceiling as recommended by de l'Orme himself. What persuaded Boullée to apply his *invention* to his project? It was not, of course, only the production of Legrand and Molinos but his education. Above all a "strict education is necessary to the mechanical aspects of art" (Pérouse de Montclos 1997, p. 21)[1]. This was given to him by his father who made him an exponent of the technical renewal process during the later half of the eighteenth century, when forgotten building techniques were being rediscovered. The *visionary* architect grew up according to the principle "Ars sine scientia nihil est" that "had been the first principle of French school since the age of the cathedral" (Pérouse de Montclos 1997, p. 180). The information reported by Montclos must be mentioned, because in his opinion, a Boullée ancestor, Martin Boullée, was de l'Orme's partner, and he continued to carry out his work on the Tuileries staircase, when de l'Orme died. It's not strange that Etienne Louis Boullée was one of the greatest promoters of the rediscovery of de l'Orme's technique, whose potentialities were exalted by the union of the greatness of his project. From a mere theoretical point of view, the first critique re-examination of de l'Orme's work is due to Jean Baptiste Rondelet and his "Art de Batir" of 1802. In truth the portrait that emerges regarding Henry II's architect is not particularly exalting, because Rondelet aims to dampen the enthusiasm surrounding De l'Orme at the end of the eighteenth century, thus emptying the meaning of his *invention*. On one hand, he tried to demonstrate that the small pieces building system was not invented in "Nouvelles inventions", but that it had its roots in antique and renaissance Italy. Then on the other hand, he took criticised the *invention*, and underlined its defects.

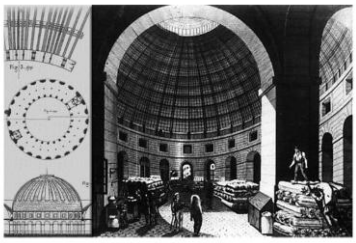


Fig. 10. The dome of Halle au blé.

To back his claim that De L'Orme was not the inventor of small-piece wooden carpentry but that it had older origins than 1561, the year of publication of de l'Orme's treatise, Rondelet tracked down some models in the domes of some churches in Venice. Firstly he pointed out that St. Marco, was made using a double wooden dome and in the Roman centrings formed and superimposed double or triple layers of boards. He referred also to Sebastiano Serlio's work, where small pieces wooden roofs are mentioned. Among these are the vaults of the Tournelles Palace in Paris and some others, seen in Italy, produced by using Vitruvio's method. Going deeper into the *invention* Rondelet underlined how it is not as cheap as the "Nouvelles inventions" suggested. Furthermore, he contended that the hole in the middle of the boards is not advisable because of thermal expansion, suggesting that it's better to hook the crosspieces on the sides of the boards, as de l'Orme did with the big size arches. Finally, there is a fine example of *invention* for a small pieces wooden roof, that the author considered more effective. It is a building solution used by Sir Lacase, a building contractor in Paris, who suggested a carpentry similar to de l'Orme's, but adding two innovative elements. The first is about the joint between the boards that becomes more elaborated (called by Rondelet "assemblées à trait de Jupiter"). And the

second is about the arch profile that suffers from a tapering along its axis **(**Fig. 11**)**. Anyway, even if Rondelet criticised his predecessor, the fact that he paid great attention to him in his treatise, gives Philibert de l'Orme a pre-eminent position in the national architectural situation, as a father of the modern French architecture[1].

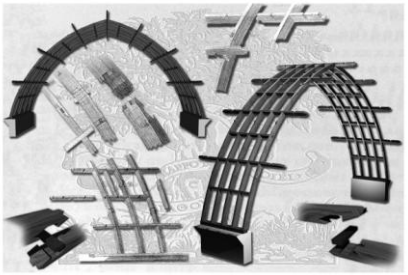


Fig.11. The Lacase's *invention*.

3. Examples of wood structures built using constructive system development by Ph. de L' Orme

In 1561 the French architect Philibert de l'Orme published a paper, entitled: "New inventions for low cost building", in which he describes a construction system to build roofs and floors using wooden boards, which were shorter and thinner. The arches, used to support the timber constructive elements of the roof or floors, were built by assembling a double series of curved boards [4,5,6], placed on edge and "transversally perforated in the middle and at the ends, like a mortise and tenon joint"[1](Fig.12 and Fig.13). The arches were constructed by juxtaposing the boards with timber wedges rammed into the holes at each end. All the arches in the system are joined and united using crossbeams, which length can be as long we like or depend on the length of wood, which we can find", placed perpendicularly to the boards and attached to the latter with wooden pins "which will be two and a half inches in width and one inch thick".

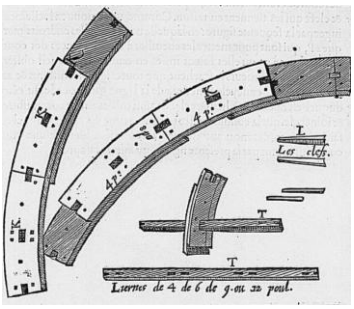


Fig. 12. Ph. de l'Orme method for the assembly boards, placed on edge

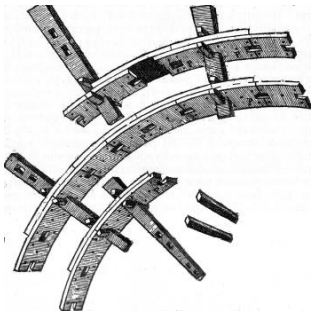


Fig. 13. Detail of the assembly boards, placed on edge

3.1. The wooden roofing system proposed by de Ph. de l'Orme

De L'Orme's invention, according[1], has numerous advantages which he himself cites in the last chapter of his paper. The most important is the possibility to build structures covering large spans using smaller timber elements, assembled using metal pieces; these structures could easily be maintained by replacing the elements which, for any number of reasons, no longer performed their assigned task. Furthermore, the lightness of such structure allows to reduce the weight *burden* on the perimeter walls that can, in this way, have a little thickness. The system invented and promoted by de l'Orme was very popular in France; but it was however forgotten until the end of the eighteenth century,

when in 1782 it was used to build the roof (a dome with a 40 m diameter, built only with de l'Orme arches) over the Halle au Blé in Paris. A few years later many architectural texts and treatises describing this construction system began to be published (for example, the *Theoretical and practical building treatise* by Jean-Baptiste Rondelet). There are many examples of wooden structures which use de l'Orme's method and yet interpret it differently. For example, the roofs of several industrial buildings built in the second half of the eighteenth century in Italy and Spain; the latter are examined here in-depth to illustrate their similarities and differences. Knowledge about these buildings, according [12], is crucial to influence the work performed not only to ensure successful long-term conservation while respecting their structural design, but also to suggest and possibly influence new design projects by exploiting what the past has taught us and what, in this case, is more topical than ever

4. Arched wooden roofs made by the timber arch system, development by Ph. De L'Orme

4.1.The former Trombetta wool mill.

The structure of the wooden roof of the former Trombetta wool mill, built in the 1870s, has a series of arches, arranged on an interaxis, approximately three metres from one another[8]. The structure was built during the refurbishment of the top floor of the building, originally with a flat roof, to create an area where the wool was stretched on tenters. De l'Orme himself also stressed that the use of the kind of roof he invented made it possible to exploit the space under the eaves which are "very comfortable for many things, unlike the ones normally found there", because "you can created beautiful rooms or halls underneath this kind of roof" [8].



Fig. 14. Trombetta wool mill, The wooden arch structure (photo Mattone)[8]



Fig. 15. Trombetta wool mill, Detail of the arches (photo Mattone)[8]

The arches were built by placing, side by side, three order of boards (a few centimetres thick and approximately two metres long), cut into semicircular pieces and placed on edge with staggered joints, and assembled with bolts and dovetail joints at the ends. Unlike de l'Orme's system, in this case the arches are further away from one another; in addition, the structure has fewer transversal wooden crossbeams. The only function of the crossbeams, simply placed next to the boards, is to maintain the reciprocal distance between the arches. The entire system is made more stable by the use of reinforcements (almost posts) where the arches join the pitches of the roof. Wooden crossbeams were inserted when one of the arches warped; since they were attached to the arch and to one of the perimetral masonry pilasters, these crossbeams ensured structural solidity[8].

4.2. The former Gallo wool mill

Another example of an arched timber roof is located in the former Gallo wool mill (now abandoned) built in the second half of the eighteenth century in Sagliano Micca (Biella)[8]. It has a series of arches built with a triple order of boards placed next to one another and fastened with screws, bolts and wooden connector plates; the roof rests on the rafters supported by the arches. Compared to de l'Orme's design, the arches are not attached to one another, however, this doesn't appear to have affected the stability of the structure which is still well preserved. The fact that this kind of construction system was inexpensive is one possible explanation why it was used to support a simple two-pitch roof; it was economical not only roof[8].



Fig. 16. Gallo wool mill, Sagliano Micca, The wooden arch structure (photo [8])



Fig. 17. The former Gallo wool mill, Sagliano Micca, Detail of the assembly system, (photo [8])

4.3. The Obrera Mataronense Pavilion

Between 1883 and 1885 Anton Gaudi designed a pavilion (to be used as a bleaching hall) for the Obrera Mataronense Society[8]. When designing this structure Gaudi probably used as reference both traditional Catalan architectures, such as the *drassana* in Barcelona and the novices' dormitory in the monastery of Santes Creus, and French models which he discovered when reading the treatises by Jean-Baptiste Rondelet and Armand Rose Emy, as well as the *Dictionary of French Architecture from 11th to 16th Century* by Viollet-le-Duc.

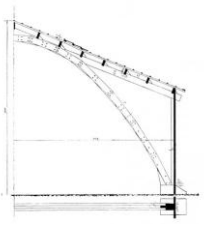
a)  b) 

Fig. 18a,b; The Pavilion of the Societad Obrera Mataronense, Matarт, Spain[8]
Detail of the wooden arch structure (R. Pane, Antoni Gaudi,Comunita Editions, Milan, 1964**)- a).**
The wooden arch structure- **b);**

The pavilion, in fact, had a roof supported by twelve parabolic arches with a triple order of wooden boards (made with a strong pine-tree called "melis")[8]. The boards were 22 cm wide and had a maximum length of 1.5 m; the external boards were 6 cm thick, while the internal boards were 10 cm thick; all the boards were assembled using bolts. Single pieces length varies in order to better comply with the parabolic bending. The rafters supporting the roof rested on the arch and on a springer, also wooden, which ensured that the arch did not move horizontally. The use of a parabolic arc allows Gaudi to

reduce the horizontal thrust at the support. The loads are in fact traced to the vertical or nearly allowing a reduction in the thickness of the perimetral walls. Nethertheless, the insertion of parabolic arcs is optimal when the loads are uniformly distributed on them. In the pavilion designed by Gaudi the load due to the roof is on the contrary concentrated almost near the top of the arch. This kind of load seems to ask for a different type of structure. The sequence of parabolic archs which characterize the pavilion of the Obrera Mataronense Society even if certainly gives it great formal intensity, however is not the optimal choice from the static point of view [8].

5. Selected spatial system for timber roof structures in Czech Republic

Systems made from curved or framed ribs are used for roof structures over circular, rectangular and irregular ground plans[9,10]. Curved ribs can be manufactured in two ways. Either as arched ribs of glue laminated cross section or as members composed of lamellas connected by mechanical connectors (on the de Ĺ Orme principle). The spatial systems formed by mechanically connected arch ribs might be, under certain conditions, suitable for structures with smaller spans. The ribs can be manufactured even in firms equipped with usual machinery. Spatial trusses consisting of curved lamellar ribs have been theoretically analysed. The analysis took into account various types of connectors such as different variants of joint bolts, dowels, nails, toothed plate connectors, connections using steel plates or punched metal plate fasteners(Fig.19a,b,c).

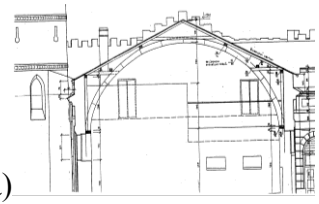
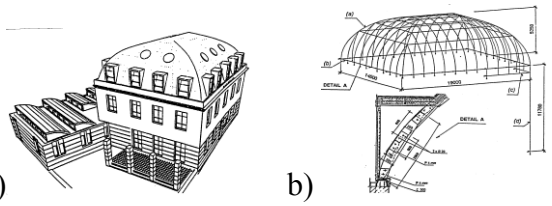
a)  b)  c) 

Fig.19a,b,c ; Roof structure of the riding hall in the Letovice castle (ČR), arches de L'Orme system- **a)**; Typical structural detail, the cross-section is made from two vertical planks, connections by means of hard wood dowels and tightening bolts- **b,c);**

After evaluating more options, a spatial system from arched ribs was developed (Fig. 20a,b; 21a,b; 22a,b). It was fabricated and assembled for a school building (Brno, Czech Republic, 1995). Timber ribs are hinge-supported on a steel wall beam. Theoretical span of the structure is 14,5 m. Basic load-bearing elements are arched ribs designed as two lamellas with cross sections 50 x 200 mm, corner ribs are formed by three lamellas with identical cross sections. The lamellas were interconnected by bolted joints 16 mm in diameter. In the regions with maximum bending moments, the connections are solved as steel plates. The load-bearing layer of the roof coat is formed by boarding, the ceiling by gypsum fibreboards. Deformations of individual ribs were tested on selected segments, overall deformations of the structure were evaluated during the assembly and in different periods of time during the serviceability of the structure. Thus, the assumption of sufficient structural rigidity was confirmed. Maximum theoretical value of deflection (48,3 mm), corresponding to 1/300 of the span, proved to be satisfactory. In comparison with other types of structures, it displayed favourable economic factors, simplicity of fabrication and speed of assembly. These structures can find application for roofs of houses if a more complex geometrical roof shape is required. After evaluating more options, a spatial system from arched ribs was developed (Fig.20,21and 22). It was fabricated and assembled for a school building (Brno, Czech Republic, 1995). Timber ribs are hinge-supported on a

steel wall beam. Theoretical span of the structure is 14,5 m. Basic load-bearing elements are arched ribs designed as two lamellas with cross sections 50 x 200 mm, corner ribs are formed by three lamellas with identical cross sections. The lamellas were interconnected by bolted joints 16 mm in diameter. In the regions with maximum bending moments, the connections are solved as steel plates. The load-bearing layer of the roof coat is formed by boarding, the ceiling by gypsum fibreboards. Deformations of individual ribs were tested on selected segments, overall deformations of the structure were evaluated during the assembly and in different periods of time during the serviceability of the structure. Thus, the assumption of sufficient structural rigidity was confirmed. Maximum theoretical value of deflection (48,3 mm), corresponding to 1/300 of the span, proved to be satisfactory. In comparison with other types of structures, it displayed favourable economic factors, simplicity of fabrication and speed of assembly. These structures can find application for roofs of houses if a more complex geometrical roof shape is required.



a) b)

Fig.20a,b; Architectural design of the roof vault for the school building in Brno – Sobesice- **a);** Spatial load-bearing system with arched ribs roofing a school building- **b)**



a) b)

Fig. 21a,b,c; Detail of the curved rib and the support on the steel eaves beam, the original damaged and unsatisfactory roof structure was removed after the erection of the new structure- **a)**; The erection of curved ribs over the original roof structure by means of mobile crane -**b)**; Assembly state of the new roof vault, corner ribs are supported on a steel Vierendeel beam, some joints of lamellas must be stiffened by means of steel plates- **c)**



22a,b The roof structure after the reconstruction- **a)**, interior of a school room in the roof space- **b)** (Brno – Sobesice, 2000)

6. Conclusions

According [8] the maintenance and preservation of wooden structures which are a significant part of historical building heritage requires suitable knowledge of both the material on which intervention is intended and the construction techniques used to build it. Understanding its structural design, characteristics, and construction specificities is crucial in order to correctly safeguard this aesthetical/cultural heritage of knowledge and values which in some cases has, unfortunately, been irremediably lost. Although the structural design and small elements of these roofs differ, their assembly and the techniques used to build them are the result of remarkable operational knowledge and construction expertise which should be studied in order to facilitate not only the work required to ensure "the working life, and permanence of signs that hand down a message", but also to acquire the extensive knowledge which can gainfully guide the design of new constructions. As emphasised by Edoardo Benvenuto, "there can be no innovation that has not already been historically provided"[1]. Knowledge is therefore important not only to preserve the past, but also to design the future[8]. Nevertheless, this kind of result can only be achieved by preserving the documentary evidence and artefacts of buildings, their construction materials, and construction and structural specificities since the latter constitute valuable information and data which may be useful in new projects. In certain cases, however, the preservation of these structures may require replacement of degraded elements[7,8,9,10]. This type of intervention, already suggested by de l'Orme, allows to preserve the structures without altering in any way the structural meaning of the building[7,8,9,10].

Acknowledgement

This paper was supported by the project: Past, Present and Future of Timber Structures in Buildings-Symbols, with great Culture and Historic Significant for our Country." Granted from Ministry of Education in Bulgaria(2017).

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FUNCTIONAL REVITALIZATION OF THE ARCHITECTURAL HERITAGE MONUMENT AFTER RESTORATION AND CONSERVATION - SUMMER HUNTING PALACE "SITNYAKOVO"

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***Abstract**: The Summer Palace 'Sitnyakovo' in the Rila Mountain is an architectural, cultural and historical monument of important national value. The estimated date of its construction is between 1890 and 1904. It is built under the rule of Tzar Ferdinand as a hunters' palace of the Bulgarian monarch. It is characterized with its functional, volumetrically-spatial composition, which is new to the Bulgarian building tradition. The artistic and cultural value is determined by the elegant solution of the interior in the 'Secession' style, contemporary to the time of building of the palace. The analysis of the architectural complex is done on situation, land shaft solution, volumetrically-functional division of the palace building (in zones with representational halls; and a zone with residential functions), building elements and materials and their condition at the moment. On this ground the authors are offering the necessary needs for restoration and conservation of the unique architectural complex, as well as the contemporary enrichment of its function in terms of tourism and museum exposition.*

***Key words**: Summer Hunting Palace 'Sitnyakovo', architectural heritage, revitalization*

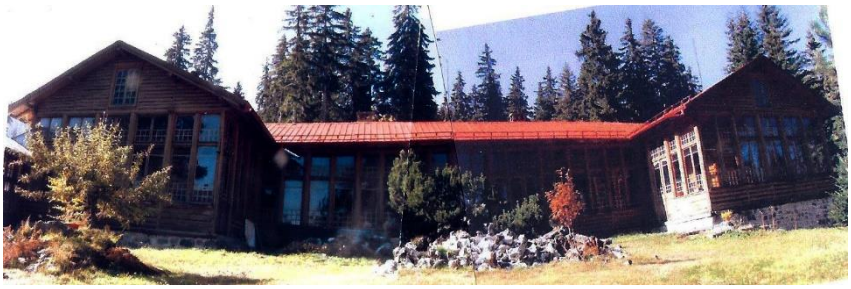


Fig. 1. General view [1]

Functional revitalization of the architectural heritage monument includes the construction of: 1.musean historical complex for the epoch of the Bulgarian monarchy;

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2.Museum of Nature for the ecological high mountain region of Rila - Chamkoria; 3. Visitors center and administration for the management of the complex and the surrounding ecosystem; 4. Restoration of the activity of the existing palace kitchen by adding additional places for consumption of ecological products; 5. "guest house" - hotel, located on the site of the destroyed stables.

The new features envisaged in the project will cause the summer hunting palace "Sitnyakovo" to become a living cultural-historical and cultural-research center for the Rila Mountain in Bulgaria.

1. Introduction

The summer hunting palace in 'Sitnyakovo' region, located in the Rila mountain, is an architectural, cultural and historical monument of important national value. The estimated date of its construction is between 1890 and 1904. It is built under the rule of Tzar Ferdinand as a hunters' palace of the Bulgarian monarch. [3]



Fig. 2. General view collage

1.1. The architectural construction value of the "summer hunting palace" is determined by the unique functional and volumetric composition of the building, unknown to this moment in Bulgaria.

By the initiative of King Ferdinand in 1913, the palace holds a conference of the Balkan monarchs who sign the "Balkan contract".

1.1.1. The functional scheme corresponds to the richer living standards. There are separate groups of rooms: living room, bedrooms and sanitary facilities, reception halls for representative functions, premises for the servicing staff. The various functions are grouped into separate building volumes connected in a common architectural composition. In the development of the site were used functional schemes inherent in the German (Austro-Hungarian) construction tradition.

1.1.2. The buildings in the complex are constructed of a block wooden structure, characteristic of the northern building tradition. The main construction material is beams of Siberian larch. This type of construction fully harmonizes with the environment of centuries-old pine forests.

1.2. The artistic and cultural value of the summer hunting palace is determined by the unique solution of its preserved authentic interior space in the 'Secession' style.

1.2.1. The decoration of the walls and ceilings is fulfilled in the contemporary style of the "Secession" - contemporary to the time of building of the palace. Similar examples in our architecture are extremely rare.

1.2.2. The preserved furniture, fireplaces and household facilities are unique for this period of our cultural and historical heritage.



Fig. 3. Secession Masterpieces

1.3. The rich collection of hunting trophies documents the ecological diversity of the Rila National Park over the past century.



Fig. 4. Hunting trophies

1.4. Location of the "summer hunting palace" – Sitnyakovo region

1.4.1. Sitnyakovo Palace is situated among the unique nature of Rila park - a dense pine forest at an altitude of 1900 m. The architectural ensemble is built in the middle of the tourist route between Banska Bistrica and the hunting lodge of the Saragyol lakes. Beside it passes the main road between Borovets resort and Musala peak.

1.4.2. The hunting palace consists of four separate building volumes on a steep slope. The one-story buildings unobtrusively blends into the picturesque natural environment.

1.4.3. The courtyard landscape consists of rock gardens, water mirror and forest rare plant species.



Fig. 5. The courtyard landscape

2. Functional analysis of the existing building structure.



Fig. 6. Architectural functional and compositional schemes of the buildings in the complex "Summer hunting palace Sitnyakovo". Buildings 1, 2, 3 and 4. [1]

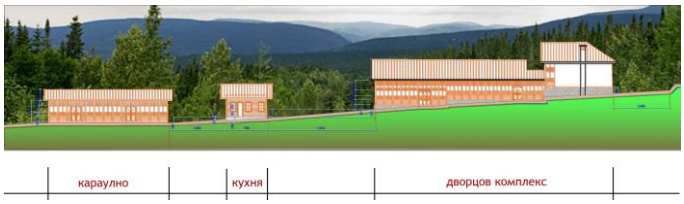


Fig. 7. General side view /siluet/

2.1. A Palace Building with representative functions [1]. The reception hall, the dormitories to the royal family and the guest rooms are compositionally united by an external glazed corridor, richly decorated with hunting trophies. The symmetrical façade composition and the location of the building in the highest part of the terrain correspond to its primary and functional significance in the general architectural complex.

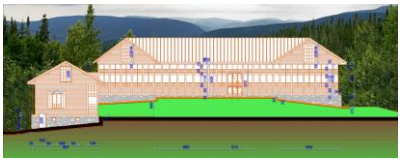


Fig. 8. Palace Building with representative functions/siluet/ .

2.2. A palace building with residential functions [1] . It is built on the principle of the corridor system. The main functional unit is the canteen with adjacent auxiliary rooms. The decoration of the walls and wooden ceiling in the dining room is designed in the typical style of the epoch "Secession".



Fig. 9. View of Palace Building with residential function /siluet/

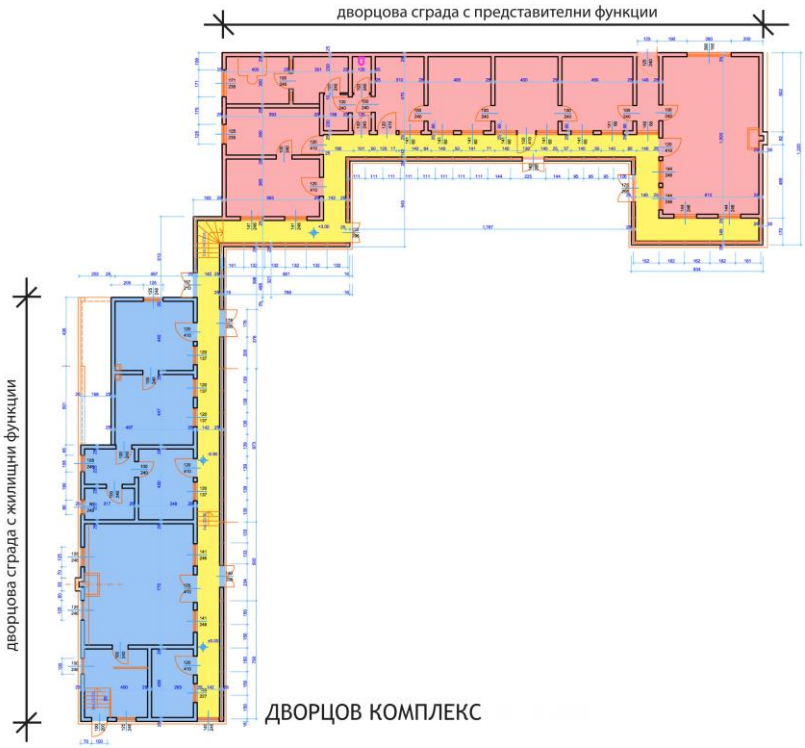


Fig. 10. Plan of the Palace Building 'Sitnyakovo'
with representative function (in red) and residential function (in blue)

2.3. Kitchen – it is located in a separate building volume [1]. It is equipped with a fireplace, ovens, stove, grill, and two 200-liter boilers united around the centrally located chimneys. The kitchen facilities are unique and should be preserved in their original form.



Fig. 11. Kitchen

2.4. Security building. The authentic functional scheme is not preserved [1]. At present the building is divided into separate bedrooms with private bathrooms. The original facade of the guardhouse is preserved.



Fig. 12. Guardhouse

2.5. On the territory of the palace was built a **stable**, which was subsequently destroyed [1].

3. Architectural and constructive concept for preservation, restoration and constructive strengthening of the existing building volumes of the Summer Hunting Palace "Sitnyakovo".

3.1. Architectural concept for conservation, restoration and exhibition of "Summer hunting palace" - Sitnyakovo

3.1.1. Building measures to preserve the architectural monument**.**

Restoration and reinforcement of the existing stone foundation and plinth.

Replacement of the main load-bearing beams that are compromised by the decay processes of the material. Replacement of 50% of the flooring with a suitable new coniferous material.

Replacing the main bearing elements of the enclosure and inner walls.

Complete replacement of the interior walls.

New wooden cladding of the roof, waterproofing and new shingles.

Replacement of the existing electrical installation.

Restoration of the existing heating system from tiled fireplaces and provision of additional heating, suitable for maintaining a certain microclimate to preserve the wooden structure.

3.1.2. Restoration and preservation works for the palace.

The palace's structural wooden facades to the inner courtyard have to be restored due to their uniqueness after a thorough analysis of their physical condition.

All decorative elements on the walls and ceilings should be removed before replacing the main structural elements. Upon completion of the reconstruction of the building they will return to their places, fully restored.

Mobile furniture and hunting trophies will be restored and conserved. Missing furniture elements should be restored in the general historical context.

3.2. The construction of the building ensemble of the hunting palace "Sitnyakovo" is entirely carried out by a wooden bearing construction system.

The outer walls and part of the inner walls are load-bearing, constructed by the system "block" beams - logs from "larch", connected by wood joints. The roof structure is wooden, made of lined material - larch. The roof structure is cladded with wood planks. The floor is made of beamwork of the same material on which the floorboard is mounted.

As a result of a number of factors - weathering and, most notably, unprofessional repairs in the past decades, decay processes have developed in large areas of the wooden structure [4] - wood fungus appears, some of the foundations are subdued.



Fig. 13. Decay processes of some of the foundations



Fig. 14. Decay processes - wood fungus

All decaying or rotting wooden elements of the building should be replaced - from the floor through the walls to the roof, as well as measures should be taken to properly remove the water from the roof and the adjacent terrain to prevent the foundations from being flooded.

In order to replace the damaged and / or rotten wooden parts from the structure of the building at a height of up to 2 m from the floor (according to the prescriptions in part "Biology"), a temporary metal supporting structure should be constructed, representing a spatially reinforced block.

This structure must be able to absorb the load from the roof and that part of the supporting walls that will not be replaced.

The construction will deliver the loads to the ground by means of temporary mounting foundations.

The metal construction is dismantled after the final replacement of all compromised or endangered elements of the walls and the roof structure, after which a new floor structure can be installed in the respective reconstructed section.

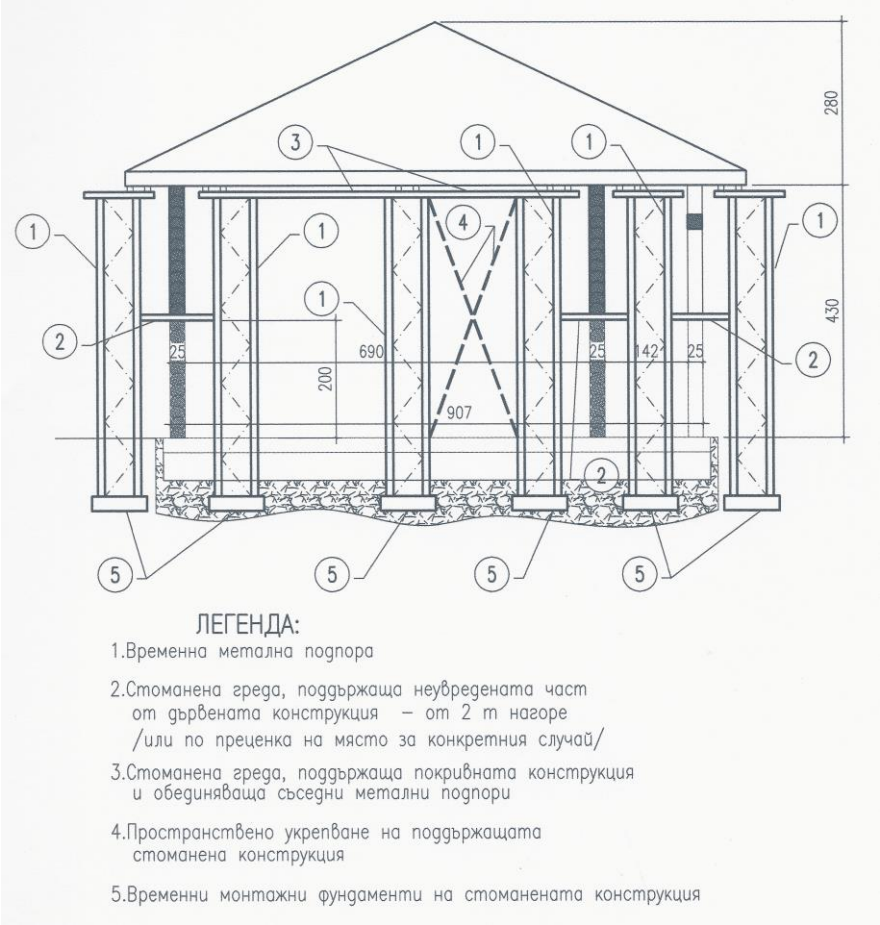


Fig. 15. Principal construction scheme for the reconstruction of the Summer hunting palace "Sitnyakovo"

4. Functional revitalization of the monument of the architectural heritage "Summer hunting palace "Sitnyakovo" after restoration and conservation.

After the analysis of the architectural and constructive structure of the Summer Hunting Palace "Sitnyakovo", the following functional, volume-spatial composite architectural elements are differentiated.

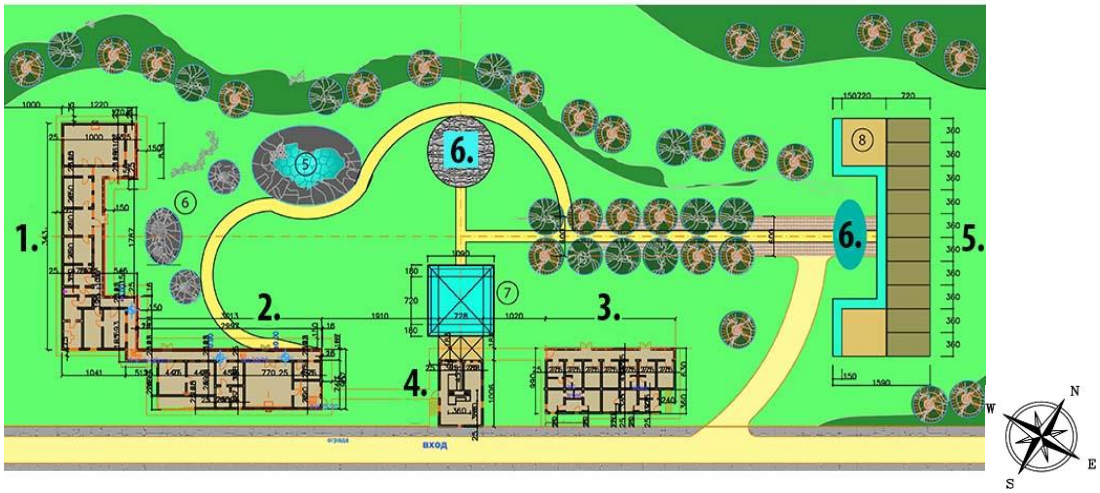


Fig. 16. General plan of the funcional revitalization of the Summer hunting palace in Sitnyakovo

4.1. Palace building with representative functions. The reception hall, the dormitories to the royal family and the guest rooms are compositionally united by an external glazed corridor, richly decorated with hunting trophies. The symmetrical façade composition and the location of the building in the highest part of the terrain correspond to its primary and functional significance in the general architectural complex.



Fig. 17. Palace building

The existing palace structure after reconstruction is suitable for a museum historical complex of the Bulgarian Monarchy.

4.2. A palace building with a residential function and a peripheral communication space - a corridor (gallery).

The reconstructed south-west wing of the historic complex is suitable for an ethnographic and natural-science museum of the ecological high mountain region of Rila Mountain - Chamkoria.



Fig. 18. Interior composition with hunting trophies



Fig. 19. Interior composition with furniture in Secession style

4.3. In the guardhouse after the reconstruction will be located: Visitor Information Center and administration for management of the complex and the surrounding ecosystem.



Fig. 20. Guardhouse

4.4. The kitchen is situated in a separate building volume. It is equipped with a fireplace, ovens, stove, grill, and two 200-liter boilers united around the centrally located chimneys. The kitchen facilities are unique and should be preserved in their original form.

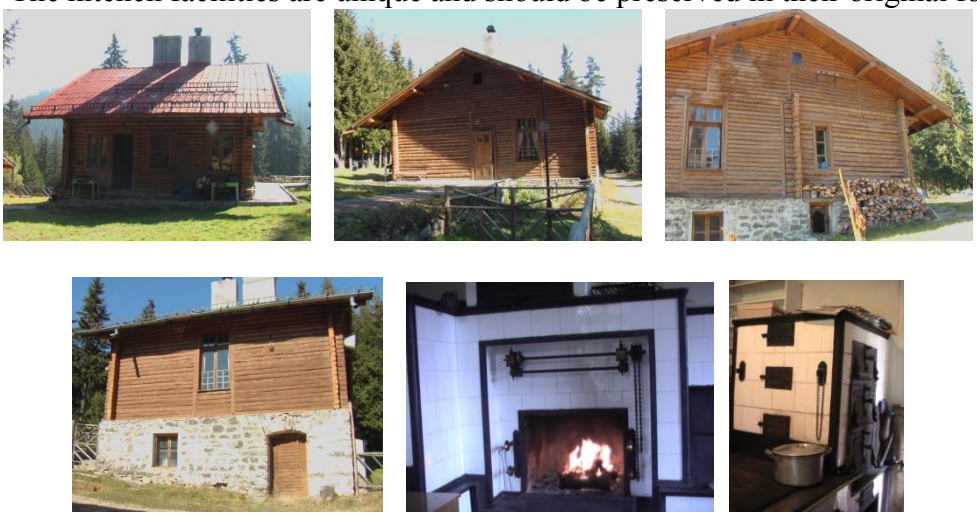


Fig. 21. Kitchen – exterior and interior

After the reconstruction, the existing original kitchen will be restored with the addition of places for the consumption of ecological local products. The design solution envisages building a glass pavilion in the architectural style of Secession. The Pavilion will serve as a hall for consumption - an integral part of the existing kitchen.

4.5. In the place of the destroyed palace stables it is appropriate to build a building with bedrooms for guests of the tourist complex. The architectural style of the building must correspond to the general ensemble. It is recommended that the building be built from a wooden "block" structure. The perimeter corridor is possible to be executed in a "Secession" style glazed metal structure.

4.6. The park environment will be enriched with new viewpoints; picnic area; a water mirror in front of the guest house. The existing dendrological richness will be complemented by new tree species typical of the Rila Mountains.

5. Conclusion

The new functions envisaged in the project will make the Summer hunting palace "Sitnyakovo" a living, socio-cultural, historical and ecological research center for the Rila Mountain in Bulgaria.



Fig. 22. View from Google Earth

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MILOS SYKORA BRIDGE – PAST AND PRESENT

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***Abstract**: The history of the bridges across the Ostravice River connecting the Silesian and Moravian parts of Ostrava has a long history. From 1847 to 1851 the Ostravice River was spanned by a chain bridge (the "Kettenbrücke"). From 1886 to 1912 there was a temporary structure called the Empire Bridge (the "Reichsbrücke"). After being destroyed by floodwaters, it was replaced by today's bridge in June 1913 produced by Vitkovice Iron Works. Bridge titled Milos Sykora Bridge is one of Ostrava's iconic structures, instantly recognizable by its distinctive arch. The bridge is named after the hero, who died in order to save the undermined bridge at the end of World War II.*
Milos Sykora Bridge is a riveted steel arch bridge with the lower roadway leading across the Ostravice River that combines part of the Moravian and Silesian Ostrava. The historical steel bridge has a steel truss arch with a span of 60 m and rise of 7 m, the total length of the bridge is 92 m and wide is 16 m. On the bridge there are two paths, each three meters wide. The bridge undergone a total reconstruction in 1979 and renovation in May till August 2002. The paper focuses the history and present of this historical structure.

***Key words**: Bridge, steel, historical structure, reconstruction, numerical modelling.*

1. Introduction

The article focuses on one of the dominants in city of Ostrava – Milos Sykora Bridge [1]. The historical steel bridge runs through the Ostravice River and connects the Moravian and Silesian parts of Ostrava. The design of the bridge took into account the effects of undermining and was formed as a three-armed continuous beam with embedded joints. A statically determined construction is perfectly compliant from a kinematic point of view, so no additional forces are introduced into the structure when the support moves. The main supporting system is designed as two riveted arched trusses. The description of the construction is complemented by a historical overview of the events from the circumstances of the construction itself, important facts related to the operation of the

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structure that affected the statics of the bridge, and information about the reconstructions that took place between 1970 and 2001 [2].

The bridge structure is modelled in commercial software RFEM [3] now to get an idea of the reliability of the load-bearing structure. Numerical model takes into account the original state in the time of realization and also in its present form after both reconstructions. The model of the bridge structure in its original state is loaded with the anticipated load at the time of the project. The load according to the current standards is applied to the numerical model of reconstructed state of the bridge. The article introduces works focused on reliability assessment of the load-bearing capacity and usability of a bridge structure, which is under analysis now.

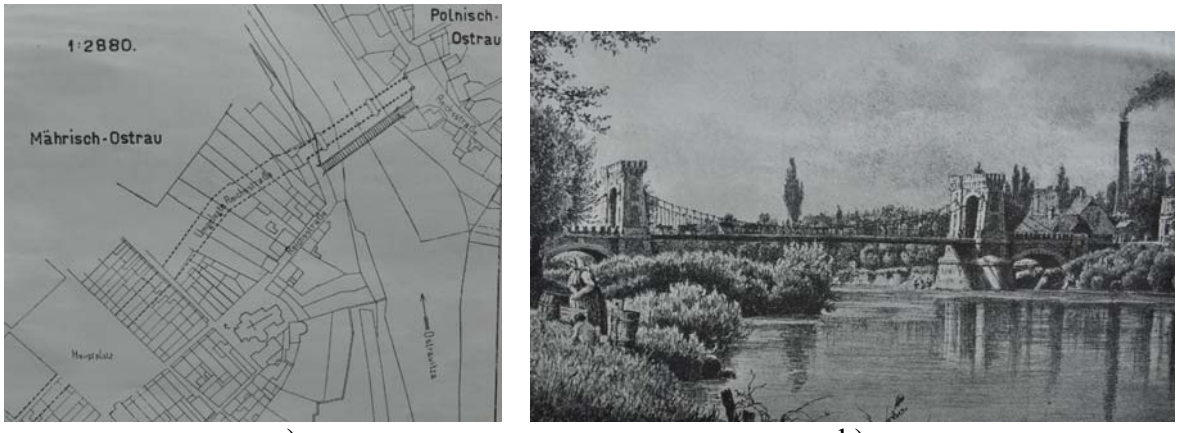
2. Bridging the Ostravice River

Villages and towns have always been built around the rivers. As a result of their development, both banks of watercourses were inhabited, and the need for interconnection of urban areas separated by the river arose. The first mention of the connection between the Moravian and Silesian parts of Ostrava dates back to 1450, when a wooden bridge was built on the site of the current Milos Sykora Bridge [4]. The wooden construction of the bridge was unable to withstand the floods, so a new wooden bridge was built every 2-3 years.

In the 18th century, a new material for the construction of the bridge - steel - was brought to the fore. The history of the origins of steel bridges is covered by the publication [1]. The first steel bridge in the world was the arched bridge in England built in 1779. This bridge is also a symbol of the industrial revolution. It carries the name Iron Bridge and runs across the Severn [5]. A few years later it became a popular different type of bridge construction - chain bridge. The first chain bridge was built in North America in 1801. The bridge was designed by Judge and Investor James Finley and the name was according to Jacob's Creek, which it bridges in Pennsylvania [6]. This type of bridge construction came into the Europe in 1819. The pioneer was again England, where was a boom in the building of chain bridges. The first chain bridge was built in Scotland by naval officer Samuel Brown. It bridges the Tweed River and is known as Union Bridge [7].

2.1. Chain Bridge (the "Kettenbrücke")

In the 1940s was the wood, the only material used to bridge the Ostravice River, replaced by steel. The location and view of the newly built bridge is shown in Fig. 1.



a) b)

Fig. 1. Chain Bridge, taken from [1]: a) position; b) historical drawing

The construction of the bridge was commissioned by Ing. Josef Seifert of the Land Building Directorate, who chose for his time a modern construction - a chain bridge. The first chain bridge in Czech Lands was built by Bedrich Schnirch from Patek over the Ohre River in 1823-1824. It was a road chain bridge in Straznice in Moravia [8]. The supporting structure of the newly built bridge was hung on chains led through portal pylons. Chains were anchored to the coastal blocks. The width of the bridge was 56.9 m. The bridge weight was transmitted by vertical iron rods to four chains, which were composed of 24 elements of flat iron. The bridgehead consisted of crossbars made of larch wood with dimensions of 420 x 316 mm. Construction began in 1847 and lasted for four years. During the load test, 113,000 kg of load was placed on the bridge, demonstrating the reliability of the load-bearing structure. The opening ceremony took place on 24 June 1851. A reference to the Chain Bridge can be found in publications [1, 4, 8].

Other chain bridges have begun to emerge as a result of imperfect chain links, insufficient rigid structures under wind load, or chain clearance due to tilting or shifting of the anchor blocks. All these failures, the designer Ing. Josef Seifert tried to avoid. Simple and well-arranged joints of the supporting parts were used on the construction, the bridge was reinforced with pairs of screwed beams. The design of the anchor blocks was also done with caution. The anchor blocks were connected with pillars above the pylons by massive vaults. This measure ensures a stable position of the chains irrespective of the state of the earth around the anchor blocks. During the realization of the bridge were also checked the individual elements from which the structure was composed. Each iron element was subjected to a pull test that had to prove that the element's load was higher than that calculated in the static calculation. Dynamic effects were also tested in the form of hammer strikes perpendicular to the length of the element.



Fig. 2. The chain bridge collapsed, taken from [1]

In 1880, the Chain Bridge resisted an extraordinary load. Ostrava was affected by large floods. After the crisis has stabilized, the bridge has been inspected and declared operational. Six years later the bridge was exposed to another unexpected load. Forty soldiers on the horses, who along with marching soldiers, waved the construction of the bridge, causing it to collapse. During this disaster, pedestrians and two wagons were also on the bridge, one of which was fully loaded with coal. The accident required 4 dead, 15

heavy and 16 slightly injured. The year before the disaster arose concerns about inadmissible vibration of the load-bearing structure. After the check, no deficiency was discovered and therefore traffic was not limited. In the investigation of the accident, however, it was found that the steel chains were considerably rusted. In some places the surface of the chain was only a fifth of the original area. In the trial, the district district engineer Frantisek Hainz of Tesin, who was responsible for the maintenance and regular sightseeing of the bridge, was the main accused. Several expert opinions were heard during the trial, agreeing that the blasting of the bridge was just an impulse that led to the collapse of the structure. The main cause of the disaster was to be found in improperly conducted inspections and maintenance. The supporting structure was, however, corroded in places that could not be seen. Insufficient control of the load-bearing parts of the bridge was caused by a faulty construction. Defendant Franz Hainz was acquitted of guilt. The crash of the bridge is captured in a contemporary photograph in Fig. 2.

The collapse of the Chain Bridge in Ostrava aroused distrust in this type of bridge construction in the Czech Lands. Despite the indisputable advantages of low cost of construction of the bridge and its aesthetic appearance, the using of chain bridges began to reduce.

2.2. Empire Bridge (the "Reichsbrücke")

Shortly after the Chain Bridge collapsed, the construction of the new bridge, which was called the Imperial, began. The name was based on the Reich Road, which led from Opava to Tesin. A steel was selected again to build the bridge. The material was supplied by Vítkovice ironworks. The load-bearing structure was a truss riveted construction. The total width of the bridge was designed at 9 meters, the roadway was 6 meters wide and the width of the two pavements was 1.5 m.

Industrial production in Austro-Hungarian Empire increased, as did the population. In 1890, 64,000 people lived in Ostrava, and over the decade it increased to 120,000 inhabitants. The Empire Bridge did not meet the needs of the emerging city, so it was necessary to replace it with a new bridge, which is now known as the Milos Sykora Bridge.

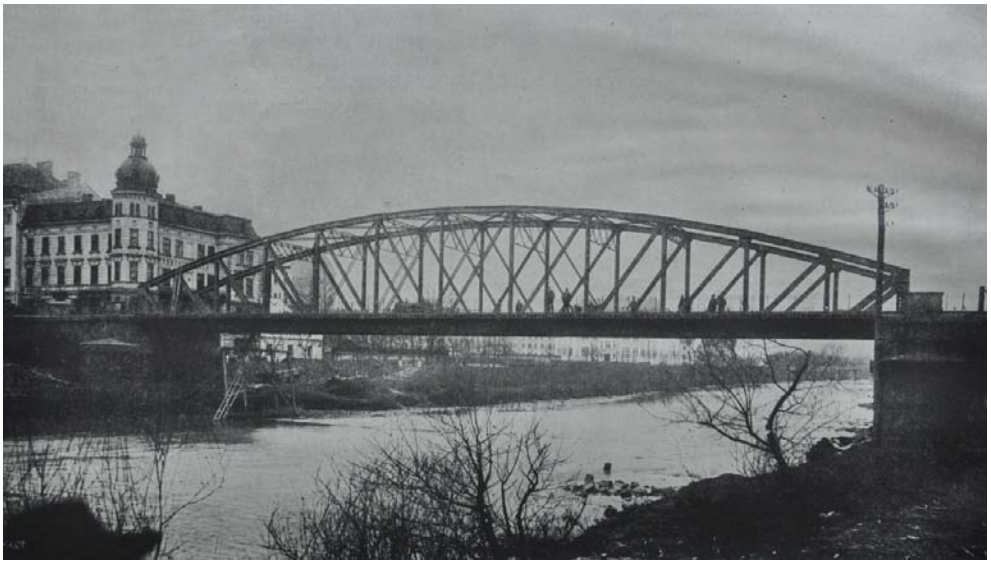


Fig. 3. The Empire Bridge, taken from [1]

2.3. Milos Sykora Bridge

The Milos Sykora Bridge was realized in the years 1912-1914 according to the project of Ing. Gustav Hermann (for details see [1]). The supporting structure consists

riveted steel lattice girders with a suspended deck. When designing a static system, it was necessary to consider the effects of mining activities. For this reason, a design of a statically determinate structure has been proposed, which guarantees that the decrease of the support does not lead to the creation of additional internal forces. The main static system of the support structure is a three-armed continuous beam with embedded joints. Road width has increased compared to the Empire Bridge to 9.3 meters, as well as widened sidewalks and whose width is now 2.5 meters. The span of the bridge is 91.6 meters and its height is 7.1 m. Rise of arch makes 9.6 m, the upper bracing provides stability along with two facing arcs. Due to the fact that the building is located in the city center, emphasis has also been put on aesthetics in the design of the bridge. The historical photograph of Milos Sýkora Bridge and its current appearance are shown in Figs. 4 and 5.



Fig. 4. Milos Sykora Bridge - historical photo



Fig. 5. Milos Sykora Bridge - view of the current state

3. Realization of the bridge in the period 1912-1914

Prior to the construction of the bridge itself, it was necessary to demolish the existing Empire Bridge. The bridge was dismantled by hand, due to the protest of the owner of one of the surrounding buildings, which has not agreed to the demolition by explosion. An emergency bridge, which was 330 meters downstream, was built to maintain the operation. The emergency bridge provided a 6.5-meter wide pavement and a 2.5-meter wide walkway. The realization of the bridge is captured in the Figs. 6, 7 and 8. Due to the political situation, pressure on the opening time of the bridge has increased. It dropped even after the stress test and on 9 August 1914 the bridge was made accessible to public transport. Shortly after the opening of the bridge, he was burdened with the crossing of many soldiers and the transport of military equipment, which proved his high capacity.

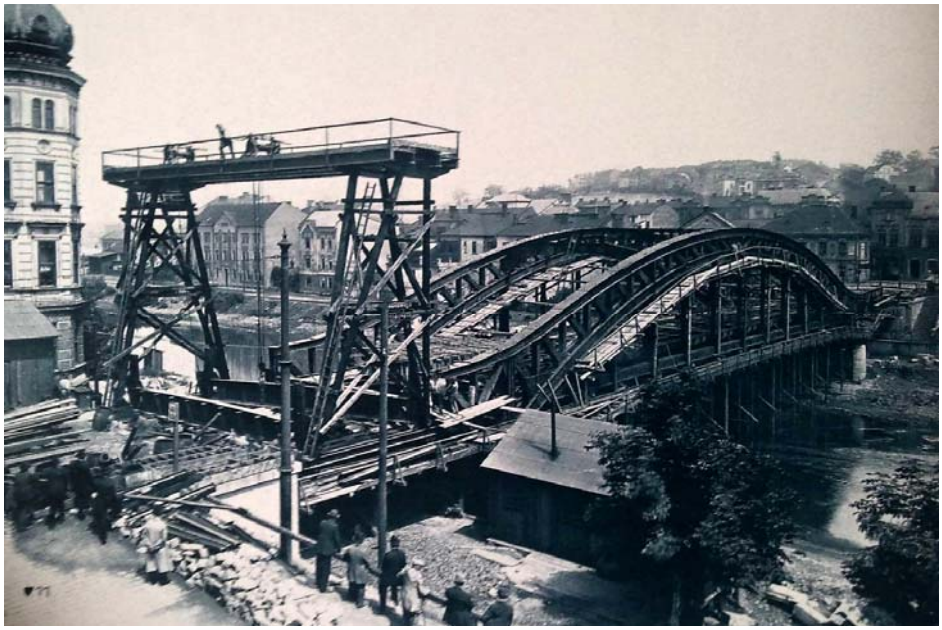
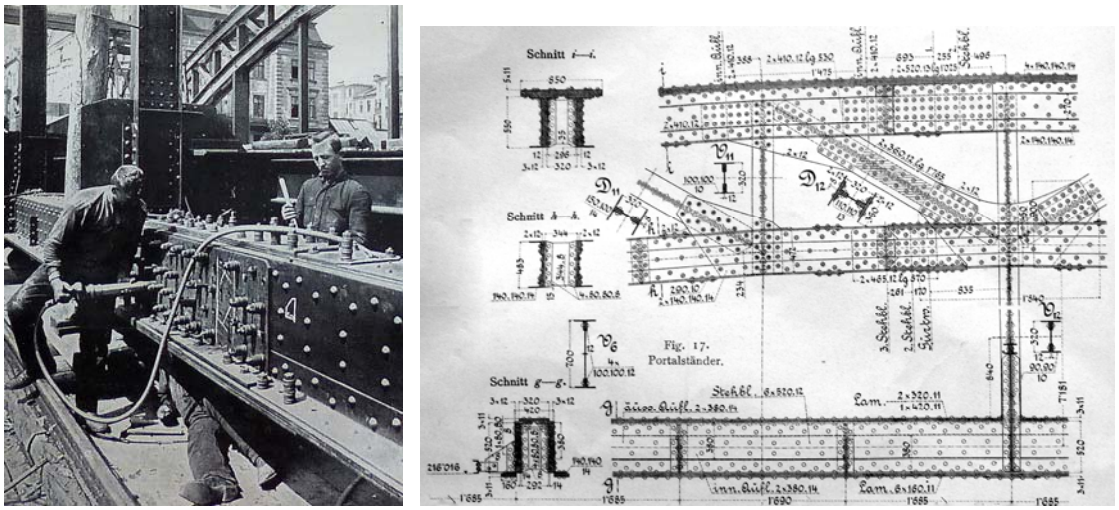


Fig. 6. Construction of Milos Sykora Bridge



Fig. 7. Construction of Milos Sykora Bridge, taken from [1]



a) b)

Fig. 8. Construction: a) air hammer riveting; b) details of production documentation

4. Operation on the construction

A heavy load resisted the bridge on April 30, 1945, when the Russian army liberated Ostrava. The event is described, for example, in [9, 10]. Russian tanks were heading from the center of Ostrava across the bridge towards Michalkovice. However, the Nazis placed bombs under the bridge. The young turner Milos Sykora, together with his friend Olsak, volunteered for their removal. Milos Sykora succeeded in cutting the wires into the bombs. The signal pistol fired a sign that the road was free but then he was hit. Tank could safely cross the bridge, but Milos Sykora was found dead. In his memory, the bridge was named as the Milos Sykora Bridge. During the war the Nazis destroyed almost all the bridges that connected the shores of Ostravice. The Milos Sykora Bridge was not destroyed, but the war period had a negative impact on him because of insufficient maintenance.

The construction of the new bridge was considered in the 1950s and 1960s. However, the proposal did not pass, thanks to which the Milos Sykora Bridge celebrated its centenary a few years ago and became one of the dominant in Ostrava city [11]. Based on the proposal, the Ministry of Culture of the Czech Republic declared it a cultural monument in 2009.

5. Reconstruction in the years 1970 and 2001

The Milos Sykora Bridge underwent two reconstructions during his existence. Both reconstructions are summarized in [2].

The first repairs on the bridge took place in 1970. The reconstruction consisted of replacing part of the bridge structure. On the existing longitudinal members was welded a bridge plate, on which was placed a reinforced concrete slab. An orthotropic bridge was created. Further adjustments were made to the storage pillars. Their bearing capacity was provided by steel sleeves. The lower part of the structure, which was galvanized with metallization, was also modified.

The second reconstruction took place in 2001. During the reconstruction, the road layers were removed. The steel construction was blasted and treated with 250 micron epoxy corrosion coating. A non-compliant reinforced concrete slab was replaced on the sidewalks with a new in 80-115 mm thick. The wall was also modified, partially drained and newly refurbished. Expansion bolts were replaced with new ones and repaired by the railing.

On the bridge structure were laid new road layers in the structure:

- modifying layer ABS I, 40 mm,
- the spray for coupling from the modified emulsion 0.25 kg/m^2,
- modifying layer ABJ II, 30 mm,
- insulation 10 mm,
- epoxy sealing layer.

The insulating and abrasion layer of the sidewalks in which CONCRETIN TEP was used at a thickness of 3 mm was also renewed. During the reconstruction, traffic was limited on the bridge - traffic was always in one lane. The reconstruction took place in the summer months and was completed in autumn 2002.

6. Reliability assessment using numerical modeling

The numerical bridge model is processed as a spatial structure in commercial RFEM software [3]. RFEM uses the finite element method for static calculations. The AutoCAD software was used to effectively defined geometry of the model, which was imported into the RFEM program.

When designing the cross-sections of the supporting elements, it was ingeniously based on the distribution of internal forces on the structure. The individual elements of the load-bearing structure are made up of several cross-sections so that the bridge is designed as efficiently and economically as possible with regard to steel consumption. Cross-sections are made of steel sheets and rolled profiles. Since these are atypical profiles, it was necessary to process all cross sections in the calculation module SHAPE-THIN, which is part of the RFEM system, for the creation of the mathematical model of the structure and computing all cross-sectional parameters. The supporting structure of the bridge was made from the steel of grade 37.

The main part of the bridge structure forming two riveted lattice girders and suspension deck. The shape of the lattice arches is shown in Fig. 9. The cross-section of the individual parts of the truss structure can be read from the diagram in Fig. 10. It is possible to see here in which parts of the arches the profiles have to be enlarged or on the contrary, it was enough to use cross sections less bearable.

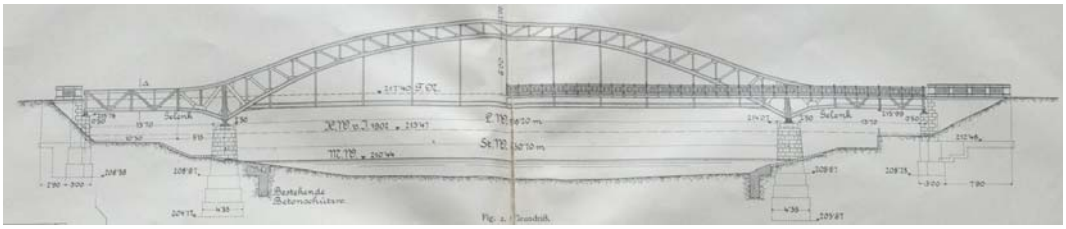


Fig. 9. Longitudinal section and view of Milos Sykora Bridge, taken from [1]

The analysis of the construction of the bridge in its original state is based on historical documentation, which was elaborated in the publication [1] published in 1916 in Austria. The numerical model created in RFEM program is shown in Fig. 11.

Modeling of the construction in the current state is based on the original values, which are modified according to the information about the made reconstructions [2]. During both reconstructions there was no changes of the main supporting elements of the bridge. In 1970, the bridge desk was modified. The 12 mm thick sheet was welded onto the existing girders, to which a 200 millimeter layer of reinforced concrete was laid. An orthotropic bridge was created.

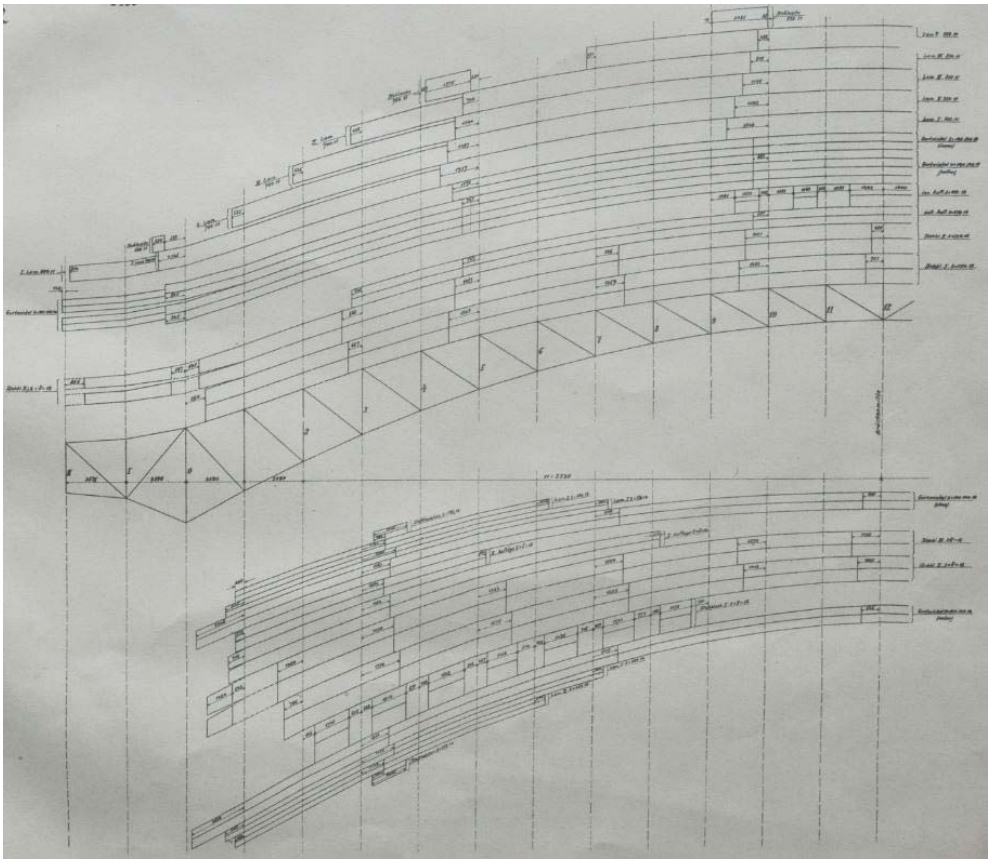


Fig. 10. Diagram of design of lattice arches, taken from [1]

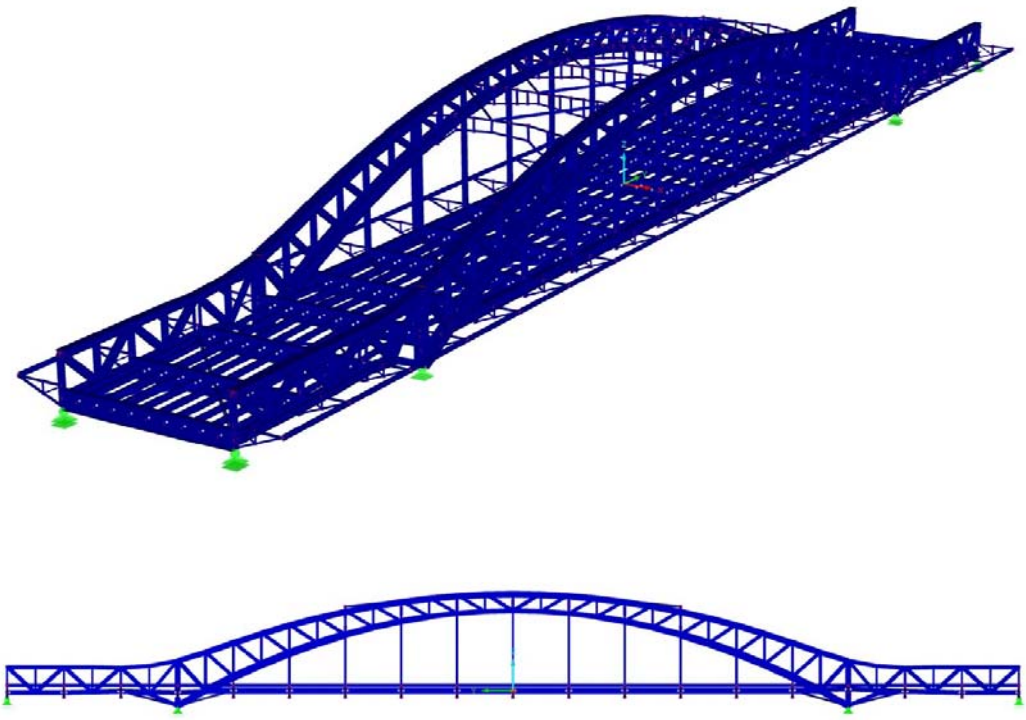


Fig. 11. Numerical model of Milos Sykora Bridge created in RFEM [3]

7. Conclusion

The article describes in detail the history of one of the most important bridges in the city of Ostrava. The article deals with all the important factors that influenced the emergence of Milos Sykora Bridge and describes significant events during its operation.

The purpose of the research of this technical monument is to create a numerical model in the RFEM program, by which using it is possible to determine the reliability of the steel bridge carrier system at the time of implementation and after both reconstructions. The information thus obtained is undoubtedly important for the further study of the vulnerability of this iconic structure, and it is possible to use it to hazard prediction and preventive measures.

Detailed analysis of the achieved results and an elaborated study of the reliability of the model states is the subject of research work in the next few month and will be published at next annual of the conference.

Acknowledgement

This contribution has been completed thanks to the financial support provided to VSB-Technical University of Ostrava by the Czech Ministry of Education, Youth and Sports from the budget for conceptual development of science, research and innovations for the 2017 year.

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WIND ACCELERATION CAUSED BY MORPHOLOGICAL EXTREMES IN STOCKHOLM'S LOUDDEN DOCKS

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***Abstract**: Loudden Docks oil terminal was built in 1926 in the Stockholm's district Lidingöbro. Stockholm Royal Seaport project is a sustainable urban development idea that proposes a transformation of Stockholm ports along the east coast. The morphology of Loudden Docks is quite extreme. More than 100 cylinder-shaped concrete silos are distributed very densely on the site that is approximately 680 meters long and 120 meters wide. This creates a specific wind situation of the place. The wind flow is accelerated between silo clusters and is also characterized by turbulent wakes. The silos deflect the wind in both, horizontal and vertical direction. This study proposes a transformation of the old oil terminal into "Windpark" and suggests options not only for directing the wind and deflecting it but also for utilizing it. An architectural shape is designed between and around the silos. The behavior of the shape in the wind flow, as well as wind loads, is tested in CFD software and the results are analyzed.*

***Key words**: industrial park, site-specific design, parametric designing, wind, CFD*

1. Introduction

The industrialization of the cities has radically influenced the preferred type and style of architecture. The industry and traffic were prioritized; the riversides were transformed and filled with cranes, storage tanks, containers, and factories. The consequences of these building strategies were not taken into account and thus the character of the natural environment was suppressed. In recent years, many cities reconsider their attitude and want to rediscover the hidden qualities that a city has by reusing old industrial places. Currently, there are many development projects dealing with the ecological transformation of unused industrial zones. Only a minority, however, focuses also on the specific conditions of the site that affect the final design and its future performance in unpredictable and ever-changing environmental and weather situations. A conversion, which respects the original character of the place and at the same time, reacts to the wind and sun situation of the site is proposed in this paper.

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2. Loudden Docks oil terminal

The Loudden Docks, in the district Lidingöbro, in the eastern part of Stockholm was the biggest oil terminal in the Stockholm County with the area of 250 000 m^2. The oil terminal was closed in 2011, after the decision made in 1999 and is currently unused. The main reason for closing the docks was to exclude the oil tank trucks from the city traffic [9]. The Stockholm Royal Seaport project is an urban initiative that plans a sustainable transformation of the eastern part of Stockholm's harbors. Within the framework of the regeneration project, the demolition of the old storage silos in Loudden Docks and a construction of new residential blocks are proposed [7]. Observing the unique urban character of the Loudden Docks, however, demands an alternative solution that is presented in this case study.

2.1. Ecological conversion of industrial parks

The following examples of transformation of a former industrial zone show a successful change of an ecologically and aesthetically exhausted land. "Emscher Park", located in the Ruhr region, was used for mining and industrial production for almost 150 years. After the decline of industries in the region, the debates about the renewal of the region started in the 80s with the project named IBA Emscher Park which proposed the change of the brownfield by opening the park to the public and inventing the artistic reasons for visiting it [5]. "Lower area Vítkovice" is a technologically unique industrial area that was opened in 1828 and used until 1998 for coal mining and electricity, coke and iron production. There was a polemic about destroying the complex after the primary exploitation ceased. In the end, the industrial architecture was preserved for the future generations and is now listed as a National Cultural Monument and a part of it is in the list of European Cultural Heritage. The revitalization project suggests a gradual transformation of the site. The northern part is opened to the public and except offering a concert hall in gas container and exhibitions in one of the blast furnaces; it hosts on a yearly basis a big music festival called "Colours of Ostrava" [6]. Another example of re-usage of an old, industrial architecture is the Gasometers project in Vienna. The four huge cylindrical structures served as gas tanks and in 1995 it was decided that the revitalization design will be proposed by four world-famous architects. The old gas tanks were changed into housing, cultural center, shopping center and offices. Three new architectural shapes were built in addition to the old gas tanks [4].

The following case study similarly proposes a possible transformation of an industrial landscape, and, furthermore, the main idea for the transformation is based on the analyses of the specific weather and environmental situation of the site.

2.2. Analysis of the case study site

More than 100 reinforced-concrete silos and 6 underground storage cisterns are built as one dense urban structure. The cylindrical silos are 13 to 34 meters in diameter and around 20 meters in height. They used to serve for storing petroleum, diesel, heating oil and naphtha. The north-east side of the area is enclosed by the fjord, in the south, there is a forest and in the north-west, the shipping containers are stored (Fig. 1).



Fig. 1. Current situation

The silos block the connection to the waterfront and, because of their height and the distribution on the site; they shade each other and the area close to the water throughout the year. Only the south-west side is exposed to the sun rays. Figure 2 shows the path of the shadow on 21st December and 21st June, which are the days of summer and winter solstice. The shadows are displayed in the color scheme from pink to violet which is adequate to the shadows from the morning until the evening.

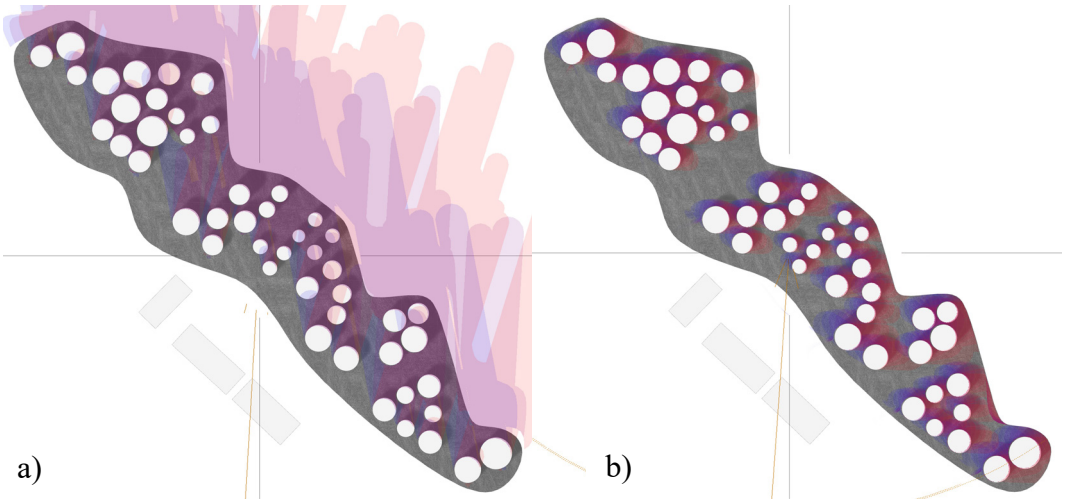


Fig. 2. Sun analysis: a) 21st December; b) 21st June

The site's morphology affects not only the sun situation but also the wind flow between the individual silos and silo clusters. The prevailing westerly winds that, in average, reach the speed 6 m/s and the stronger southerly winds with the average wind speed 9 m/s are shown on the wind rose. The weather data file for Stockholm was obtained from the Energy plus website [8] and using the Autodesk Ecotect software converted to the graphical form (Fig. 3). The wind flow was analyzed in the Autodesk Flow Design CFD (Computational Fluid Dynamics) software. The wind tunnel around the tested object(s) is created automatically. The materials or the roughness cannot be set in the software. A big advantage, however, is the fact that the tested geometry does not have to be waterproof.

The software is used mainly for a quick evaluation of the wind behavior in the conceptual stage of design [1]. The acceleration, the turbulence, and the wind flow pattern were observed for the westerly and southerly winds (Fig. 4a,b). The wind flow is displayed in 3D "flow lines" mode and the horizontal and vertical wind deflection can be observed in Figure 4.

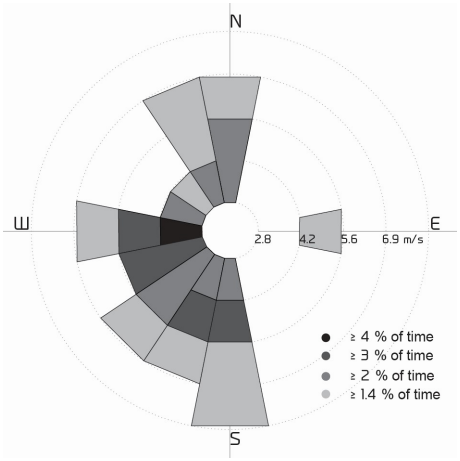


Fig. 3. Wind rose: frequency, speed and the directions of the wind

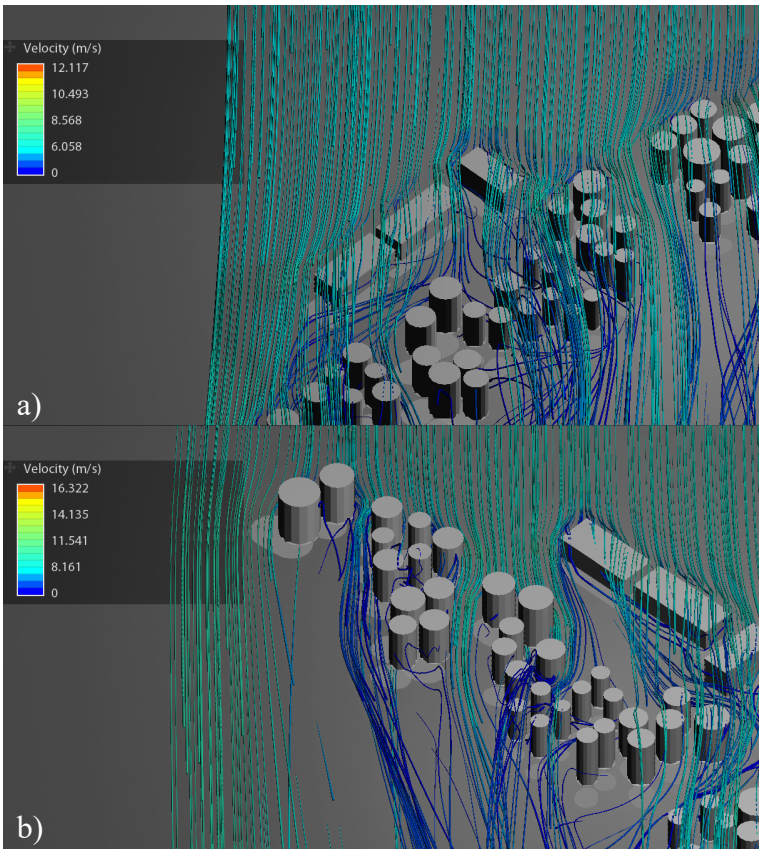


Fig. 4. Wind analysis: a) westerly winds; b) southerly winds

3. Proposal for the transformation of the Loudden Docks

The ecologically and aesthetically changed environment of Loudden Docks clearly affects its microclimate. This case study is inspired by the elements that are important for this location - water and wind. The project is named "Windpark" and proposes to use the

element of water to create constructed wetlands in the empty silos and the element of wind to be present in various forms to support the recreational character of the transformed place. The rainwater is collected and stored in the silos and then led through the two stages of plant root water purification. The clean water is directed into silos in the southern side of the "Windpark". These silos are reduced in height and serve as swimming pools in the summer and ice rinks in the winter. The relationship between the wind and architecture is examined. By reducing the height of the silos and designing a new architecture, the direction and the speed of the wind flow are affected. Figure 5 depicts the proposed urban solution of the Loudden Docks.



Fig. 5. Urban solution and proposed functions

3.1. Wind barrier design

The silos intended for collecting and storing rainwater are left untouched, the silos for the first and second stage of plant root water purification and the silos with clean water are reduced in the height. Three silos are chosen: two swimming pools/ice rinks and one pool for the water purification (Fig.5). An architectural shape is considered as a wind barrier and is proposed around the chosen silos. It was designed using 3D modeling software Rhinoceros and the plug-in Grasshopper. The goal was to create a wind barrier that would deflect the wind from the area of 3 silos and to achieve a wind-protected recreational zone. Before the design itself, an angle of the future barrier was analyzed in the Flow Design software. A simple thin barrier, a rectangle in the cross-section was oriented perpendicularly and in an angle of 30º from the vertical plane (Fig. 6). The wind flow pattern and wind speed were observed in the elevation views. The initial wind speed was set to 9 m/s to observe the average wind speed of southerly winds. The resolution for the simulation was set to 140 units to obtain the voxel (one 3-dimensional particle for calculations) size less than 0.1 meters. The results are displayed in the 3D "plane" mode. The plane was placed in the middle of the barrier's length. The exports from the software were made right after the simulation went to the stable state. Maximum wind speed in the first case with the straight barrier was 15.112 m/s and is displayed in red color. Maximum wind speed for the inclined wind barrier was 12.884 m/s and is again displayed in red. The

darkest blue color represents the wind speed equal to zero. The barrier in an angle was chosen for following reasons: a) it can ensure calmer wind flow behind the barrier and b) a barrier of a smaller height has an ability to deflect the wind vertically to the larger distance behind the barrier. In Moya's project of a windbreak design in Melbourne, one of the designed shapes intended for guiding the wind above a building entrance performed better with an additional shape inclined towards the wind [3].

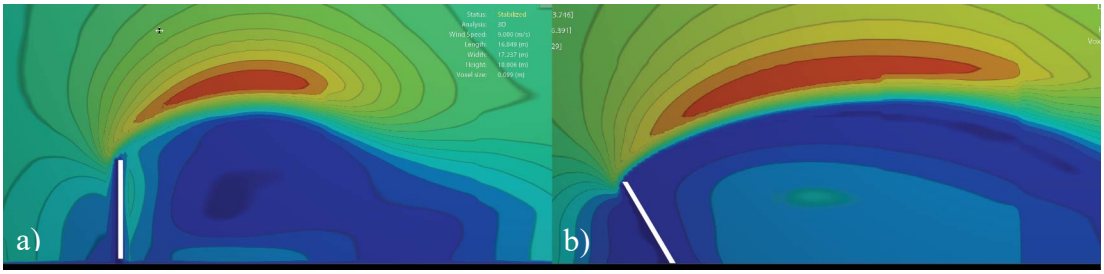


Fig. 6. Comparison of the wind flow: a) straight barrier; b) barrier in an angle

3.2. Parametric shape

A single curve was drawn in Rhinoceros that was a basis for the 3D shape. Thanks to the parametric definition in Grasshopper, the resulting architectural shape can be modified by changing the shape of the curve. Based on the distance of the curve from the silo, the lift angle is calculated and the final shape is formed [2]. Three different curves were tested and the behavior of the resulting shapes was observed in the previous research. One curve that performed the best was chosen (Fig. 7) and the final shape was designed (Fig. 8).

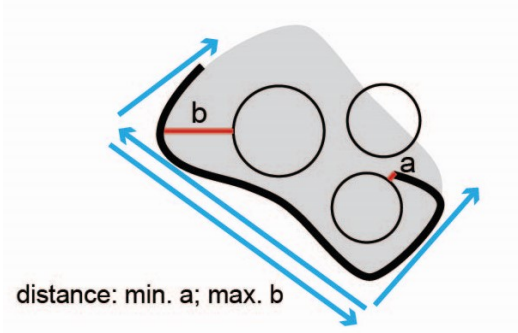


Fig. 7. Top view of the proposed shape: curve drawn in Rhinoceros (expected wind-protection zone in grey color, desired wind directions in blue color)

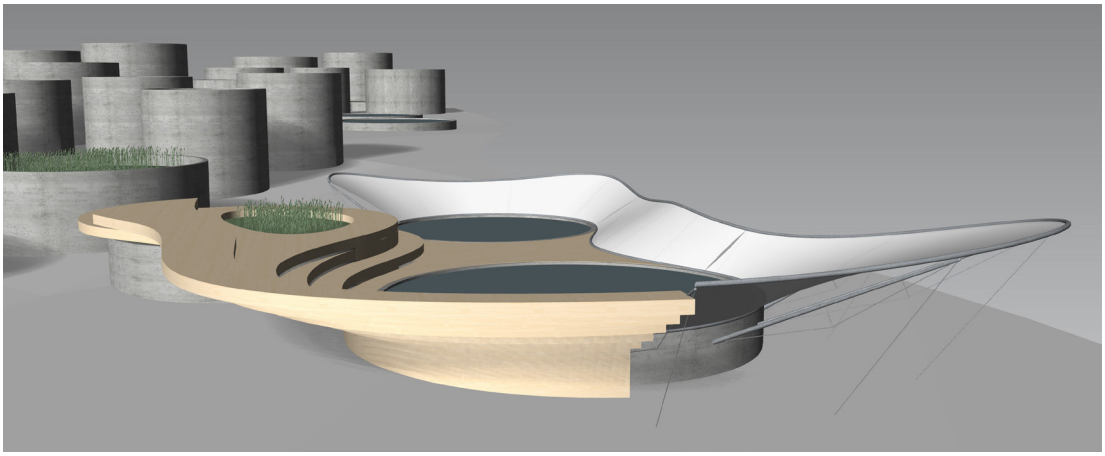


Fig. 8. Visualization of the proposed architectural shape

3.3. Tensile membrane structure as a wind barrier

For the material of the proposed architectural shape, a tensile membrane was chosen. A lightweight support structure was considered. The wind barrier is complemented with the wooden auditorium. The tensile membrane structure was tested in the Flow Design software for the southerly (9 m/s) and westerly (6 m/s) winds. It was observed, how the new situation and the new architectural shape influence the wind. The tests were made only with the geometry of two silos and the tensile membrane to get more precise simulation results. The third chosen silo is hidden under the wooden auditorium so its geometry was ignored. In the simulation, the resolution was set to 104 units to obtain the voxel size approximately 1 meter. In Flow Design, it is not possible to set the voxel size; it can be only influenced by the change in the resolution. The relation of resolution and voxel size is dependent on the size of the tested model. Figure 9 shows the wind flow around the designed tensile membrane and depicts the state of calculation right after it went to the stabilized state. It also shows the wind pressure on the surface.

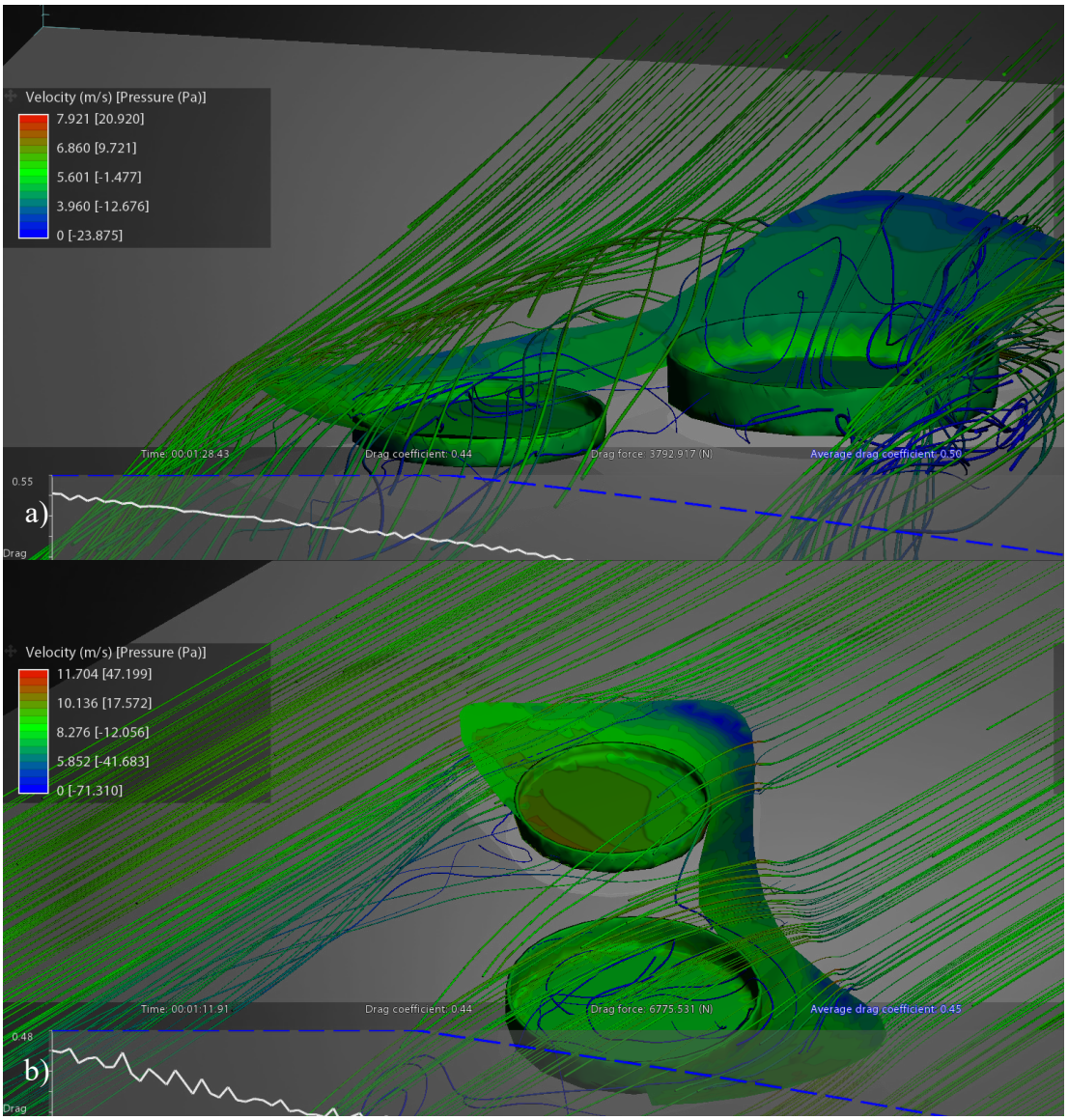


Fig. 9. Wind analysis of the proposed shape: a) westerly winds; b) southerly winds

4. Conclusion

In this paper, an ecological transformation of an unused industrial site was proposed. The former oil terminal is changed into the "Windpark", where the old silos are now used

as wetlands for the water purification and a recreational and cultural use of the place is suggested. The wind behavior on the site is tested and evaluated and an architectural shape is proposed around three chosen silos to serve as a wind protection for the leisure activities. It is verified that the proposed shape should be inclined towards the wind flow to result in a calmer wind flow before and behind the barrier. Turbulence will occur few meters above barrier but should not disturb the planned leisure activities behind the wind protection. The wind flow is also deflected to a larger distance behind the barrier so the planned auditorium around the swimming pools/ice rinks should be also protected from the strongest winds. Finally, a tensile membrane was proposed around three selected silos and the results of the wind test in the prevailing westerly and strong southerly winds show the following:

- In the westerly winds with an average wind speed determined as 6 m/s, the wind flow is deflected vertically and horizontally by the wind barrier. There are slow turbulences above the pools with the wind speed close to zero. The wind pressure on the proposed tensile membrane is between -23.9 and 20.9 Pa. The calculated average drag coefficient that represents the resistance of the shape in the wind flow is 0.50.

- In the southerly winds with an average wind speed determined as 9 m/s, the wind flow is again deflected by the newly added wind barrier. The slow wind turbulences occur above and behind the swimming pools/ice rinks. The wind pressure on the proposed tensile membrane is much higher, in this case, it is between -71.3 and 47.2 Pa. The calculated average drag coefficient of the shape is a little lower than in the case of westerly winds, it is 0.45. The designed wind barrier has better aerodynamic properties in the southerly wind flow.

Acknowledgement

The paper was realized with the financial support of the Grant Agency of the Slovak Republic (VEGA). The project registration numbers is VEGA 1/0477/15.

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LOG CABINS AND MOSQUES WITH WOODEN MINARETS IN BOSNIA AND HERZEGOVINA

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***Abstract**: Bosnia and Herzegovina is rich in forests with different deciduous and coniferous trees which are suitable for construction, and timber as a construction material found its massive usage in the oldest periods of time. Sacral buildings, mosques with wooden minarets and chapel churches- Brvnara (Log cabin), Orthodox and Catholic, represent cultural heritage structures made of wood in Bosnia and Herzegovina. Their common denominator is residential architecture which was the basis for their construction techniques, and the diversity is reflected in their specific features. These structures were for a long time neglected and placed on the margins to be forgotten. The paper presents the basic data and gives illustrative examples of sacral wooden objects (log cabins and mosque with wooden minaret) in Bosnia and Herzegovina. As well some criticism is expressed regarding inadequate and unnecessary repairs which lead to the loss of historic material and consequently the loss of historic character and authentic value of the heritage. So, it can be stated that sometimes peoples' ignorance leads to the loss of authenticity of cultural heritage structures.*

***Key words**: Brvnara (Log cabin), Mosques, wooden sacral objects, connections, maintenance, ICOMOS*

1. Introduction

Bosnia and Herzegovina is rich in forests with different deciduous and coniferous trees which are suitable for construction. Timber as a material had a massive use and application in the building construction in the oldest periods of time. Two types of sacral objects were built from timber - mosques with wooden minarets and Brvnara (Log cabin).

The significance of these structures has been recognized only in the last decades in Europe and even later in Bosnia and Herzegovina. This was not the case several decades ago, when these structures where either reconstructed in an inappropriate manner (which is still happening in some occasions), or completely destroyed, or as a result of non-maintenance, have significantly deteriorated. Unnecessary repairs very often lead to the loss of historic material and consequently the loss of historic character and loss of authentic value of the heritage which is not in line with [1].

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2. Mosques with wooden minarets in Bosnia and Herzegovina

It is the position of Bosnia and Herzegovina, located at the peripheral region of the Ottoman Empire that played an important role and had a great influence on the architecture of mosques. Additionally the abundance of wood and stone, as well as the land configuration and the building traditions had a great impact [2]. The mosques in Bosnia and Herzegovina can be divided into two basic groups. The first group represents mosques with four-sided roofs (broached roofs) with wooden or stone minarets. While the second group are domed mosques. The first group of mosques have not been investigated deeply and there is very small amount of research and data regarding these sacral objects. One can argue that this was connected to their size, while on the other hand the domed mosques were deeply investigated [3], [4]. It should be emphasized that the domed mosques in Bosnia and Herzegovina are numberless in comparison to the mosques with wooden minarets.

The mosques of the first type, unfortunately, were rather unnoticed and in a way stayed in the shadow of monumental mosques, and were referred to as "timber" or "rural" mosques. Yet, these types of structures were numerous, and placed not only in the small "rural" places, but in cities like Sarajevo, Tuzla, Banja Luka etc. and they indeed deserve attention as cultural heritage monuments. Until 1990 in Bosnia there were 992 mosques, and from that 770 (77,6%) were with wooden minarets, 186 (18,7%) with stone minarets and 36 (3,6%) without minarets, as a characteristic architectural feature. On the other hand, in Herzegovina there were in total 128 mosques, and from that, only 16 (12,5%) with wooden minaret, 73 (57%) with stone and 39 (30%) without minaret [2], [5]. When referring to the mosques with wooden minarets there are two sub-groups. The first type are mosques with a single-space room for prayer and the second type are mosques with the pillars as a specific construction element inside the mosque. In the following text Sarajevo Golobrdica Mosque a representative of the first sub-group will be elaborated in more detail.

2.1. Golobrdica Mosque in Sarajevo

Golobradica Mosque is located in the municipality Stari Grad in Sarajevo. The practice in the XV century was first to build a sacral structure and then to build a settlement around it. The *mahala* (community) where this mosque is located is mentioned for the first time in 1528, which means that the mosque had to be built before. It is one space mosque having inner dimensions of 9,80 x 7,40 m. Fig. 1 shows the Golobrdica mosque as it is today and the minaret.



Fig. 1. Golobrdica Mosque in Sarajevo – today (author's photo)

In 1982, a self-initiated intervention was done by the local people going to the mosque. They have removed the old minaret and placed it on a prepared foundation on the northern outer wall of the mosque, thus disrupting its authentic appearance of the mosque. This intervention jeopardized one of the rarely preserved specific minarets of the city of Sarajevo, a minaret that was placed inside the roof structure and supported on the *müezzin mahfili* [2]. One has to say that inadequate education of people can lead to loss of cultural authenticity.

Reconstruction of the mosque was done in 2009 [6]. And the main task was to restore the minaret to the original position, so that the mosque would return its authentic appearance as per Venice Charter [7]. The specific feature of this minaret is that it is supported by the *müezzin mahfili*. The entrance to the minaret is enabled by the double doors as well made in wood (Fig. 2).

An authentic position of the minaret was established (Fig. 3) once part of the roof ceiling was dismantled and the visible remains were identified were the original minaret was located [6]. A new 7 meters high minaret was placed on the authentic location inside the mosque, on the *müezzin mahfili* (Fig 1, Fig 3). Mosques with wooden minarets in Sarajevo have a slightly sloping roof, which was the most suitable roof cover. The roof is covered by tiles.



Fig. 2. Golobrdica Mosque in Sarajevo- minaret supported by the müezzin mahfili (author's photo)



Fig. 3. Golobrdica Mosque in Sarajevo- placement of the new minaret [6]

3. Log cabin churches in Bosnia and Herzegovina

The characteristic sacral objects of Christianity either orthodox or catholic are log cabin churches. The origin of log cabin buildings in Bosnia and Herzegovina, being used as homes, or places of worship, is associated with primordial homeland of Slavs, the Carpathian regions famous for their dense forests. Slavs probably brought the idea of log cabin with them to the lands that they inhabited and which were also rich in forests, log cabins are found in Russia and Poland, and entire Balkans [2]. Of course, the appearance of these log cabins is very different, due to socio-historical and cultural relations predominant in certain areas [8].

In Bosnia and Herzegovina today, there are about 30 churches of Orthodox provenance [9], while only one Catholic Church of this type is mentioned, Saint Joseph in Pale [10].

3.1. Orthodox Log cabin in Romanovac

The wooden church dedicated to St. Nicola in Romanovci (Fig. 4) was declared a national monument of Bosnia and Herzegovina by the Decision of the Commission to Preserve National Monuments on in the year of 2004 [11].



Fig. 4. Orthodox Log cabin church in Romanovci (author's photo)

On the western side of the church at the entrance to the location of the sacral structure, a new bell tower is located as seen in Fig. 5. The new bell tower is completely different [12] in respect to the original one, so the authenticity of this structure is rather questionable. The covering of the church is made of "klis" fir wood having small dimensions and chamfered, as can be seen in Fig. 4.



Fig.5. Log cabin church in Romanovci (author's photo)

Data regarding the time of construction of the Log cabin church in Romanovci is not available and it is mainly related to the folk tales and traditions. The church's layout is a rectangle measuring 7.42 meters in length and 4.16 meters wide.

The walls of the building are made of oak planks. The "reinforcement" of the walls is done by struts arranged only on the inside, as is clearly seen in Fig. 6.



Fig.6. Strengthening of the walls (author's photos)

Fig. 7 shows the wooden roof structure which is visible from inside of the church as there is no ceiling [13].



Fig. 7. Roof wooden structure (author's photo)

The bearing elements of the structure are made of horizontally placed beams having sections 15/15, 17/16 and 13/12 cm. This church has two openings, the main entrance on the west side, and the second opening on the south side. In 2000 conservation works took place and they consisted of: repairs of the roof structure, cladding of the church was done with new roofing material – "klis" (Fig.4) [2], and preparatory works were done for the placement of stone slabs on the prepared surface of sand, gravel and packed earth as well as preparations for making channels that will be used for rain drainage and water that falls from the church's roof [11]. The round form of the roof is enabled by the usage of numerous rafters (Fig. 4, Fig. 7) which is not the case in other similar structures. The

structure consists of a ground and first floor but no ceiling. The inside of the church has many paintings (Fig. 8).

It has two openings. The main entrance from the west, of a rectangular shape with architrave doorstep, dimensions 1.46 x 1 m (Fig. 9). It is closed by double winged oak door 0.55 m thick (Fig. 9). The other opening is of dimensions 66 x 50 cm located on the southeast corner (see Fig. 4), which by the priest's telling was the main entrance to the church at that time (Fig. 8).



Fig. 8. Altar of the church, southeast corner, previous main entrance (author's photo)



Fig. 9. Entrance to the church and architrave doorstep (author's photo)

In order to enable the entrance of the light [12] several narrow cuts are made on the planks as shown in Fig. 10.



Fig. 10. Openings in the walls for light (author's photo)

3.2 Catholic Log cabin church in Pale

The only Catholic church of the cabin log type is the Roman Catholic Church of St. Joseph in Pale (Fig. 11). The church was built in 1911 by Austrian engineers for their religious needs. It is made exclusively from wood, so it is the only wooden church in the Vrhbosanska Archdiocese, and by the decision of the Commission to Preserve National Monuments declared a national monument of Bosnia and Herzegovina in 2004 [14].

The style of the church is Tirolian, with a combination of orthodox and catholic elements. The priest indicated that this is one of the churches which was not destroyed during any of the wars that happened in this region, which is quite remarkable.

The outer dimensions of the structure are 17 x 8 m, with an apse, or 12 x 8 m, without an apse. The height of the walls of the church, measured from the floor to the beginning of the vault, is about 4 m and the height to the bottom of the vault is about 7 m [2].



Fig. 11. St. Joseph Church in Pale (author's photo)

The logs were placed on a 40 cm high platform made of irregularly carved stone blocks placed horizontally (Fig. 11).

Above the entrance to the church is a belfry with baroque elements. The two-level bell, with a square base of the first and octagonal bases of the second level, is completed with a bulbous dome. This is something that is characteristic for this church and as stated by the priest only people with good knowledge can recognize this as a Catholic church.

The frontal facade of the church is remarkable (Fig. 12). The entrance wall is divided into three fields in respect to the treatment and the used material in the horizontal direction. The first field corresponds to the height of the indirect columns, that is, the height is about 3 m. It was built like all other walls of the church, from the logs with thin wood plagues in between (Fig. 12). The next two fields, separated by a horizontal beam, are made of planks mounted perpendicular to the logs [2], [14].



Fig. 12. Frontal facade of the church (author's photo)

The boards were used as load bearing elements and this is one of the elements that are unique for this church in respect to others. There are several windows-opening in the church making the inside of the church spacious and lightly. This is a mutual feature with the mosques and oppose to the Orthodox churches which have a lack of light. All constructive and decorative wooden elements within the church - columns, trusses, fences, cornice and window frames are richly profiled. Fig. 13 shows the inside of the church and the ties which are used to connect and stiffen the roof structure.



Fig. 13. Frontal facade of the church (author's photo)

By the priest's tails all the wooden elements are original, dating back from 1911 (Fig.14).



Fig. 14. Picture of the church taken in 1913 (author's photo)

The cruch was renovated inside however the wooden construction of the church remained the same. Only impregnation of the wood was done as a standard procedure for wood maintenance.

4. Conclusion

Wooden sacral objects in Bosnia and Herzegovina are interconnected by architectural concepts, and differ in segments which are characteristic for each religion. However, even though religions differ some elements are mutual for all of them. In a sense we can say that they are interconnected and intertwined. In the last decades the significance and importance of these structures has been recognized and they have move from the shadow to the bright light.

For all types of religious structures, the building material is used in the same manner as for local dwellings. It is seen that the sacral buildings differ regionally in the same way as the residential architecture. Permanent maintenance and repair of these types of structures is required. Unfortunately, awareness regarding the value of these types of structures is not satisfactory. This is evident by the fact that a small number of wooden sacral objects has been preserved.

Additionally, some interventions that have been done before did not follow Venice-Charter and rules of ICOMOS. This is being corrected nowadays. However, maintenance of these cultural heritage structures is not on a satisfactory level. Awareness about these structures has to be increased. Protection of all the intangible and tangible assets with the goal to preserve the cultural-historic heritage needs to be done in the near future otherwise a big historical cultural heritage will be lost forever.

Acknowledgement

We would like to thank religious representatives of all confessions who enabled us to enter the sacral objects and gave us valuable additional information about these remarkable structures.

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PARK "KAZARMITE" - A SUSTAINABLE CONCEPT FOR THE DEVELOPMENT OF THE GREEN SYSTEM IN THE SEVLIEVO CITY

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Abstract: *Sustainable development is a concept that concerns the global public following the Brundtland report (1987). Nowadays this concept is further developed into new dimensions. More and more green areas and parks are being created to recreate and improve the urban infrastructure green system. This trend originated in Germany, where after the Second World War massive landscaping of cities and the affected by the bombing of the forest massifs began. In Bulgaria, following the abolition of mandatory military service in 2007, an innovative solution is needed for the former barracks, namely to be turned into green park systems. The towns of Sevlievo, Stara Zagora, Kardzhali and Blagoevgrad are the first in our country to make the former barracks in an active park system.*

Key words: *Park, green system, sustainable development, innovative management, demilitarization*

1. Introduction

Sevlievo is a city in Central Northern Bulgaria – **fig. 1**. It is located in the Gabrovo region and is the administrative center of the Municipality of Sevlievo (http://www.sevlievo.bg/). The city is situated in the Sevlievo valley of the same name, surrounded by the Sevlievo heights to the north and to the south by the Chernovrash ridge of the Middle Forebalkan. The city lies on the left bank of the Rossitsa River, shortly after the river Vidima (left tributary) flows into it. Rossitsa River is a left tributary of the Yantra River. Its length is 164 km, which is the 13th place among the rivers of Bulgaria.

The Rosica River is Yantra's largest influx (http://alba-books.com/alba-books/index.php?route=product/product&product_id=7581). The average altitude of the settlement is 200 m. The climate is moderate-continental, with great amplitude between the lowest and highest annual temperatures (Velev, 2002). Sevlievo is surrounded by a variety of landscapes - ranging from vast fields to rounded hills and pointed hills with steep slopes and picturesque gorges cut by the rocky winds of Rossitsa. There is a rich biodiversity in the area, with varied flora – deciduous forests (mostly oak), bushy vegetation and pastures, and a diverse animal world. These features give the local landscape a special color and attractiveness (Bondev, 1991). They are also a prerequisite for the development of different

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types of tourism. Sevlievo is a sustainable city (Rusev, 2005; Georgieva, 2015). Nowadays this concept is further developed into new dimensions. In Bulgaria the transition to sustainable development has been terribly difficult for many cities (Rusev, 2002).



Fig. 1. The Center of Sevlievo city

At present, sustainable cities are a few in number compared to other countries. The fact is that Sevlievo and Pleven are the sustainable cities in Central Northern Bulgaria. (https://dariknews.bg/regioni/pleven/pleven-poluchi-priza-evro-grad-v-klasaciiata-na-darik-naj-dobriiat-grad-za-zhiveene-2025219). "Kazarmite" – **fig. 2** is the first park in Bulgaria created in 2012 on the territory of the old abandoned barracks. This is an innovative management in 21 century, (https://dariknews.bg/regioni/gabrovo/v-sevlievo-otkriha-edinstvenite-kazarmi-stanali-park-963744) originated from Germany. We should say that Germany is a sustainable country, that is an innovator in many ways. After the World War 2 it's forests and cities are destroyed by bombing and improper operations.





a) **b)**

Fig. 2 Park "Kazarmite": a) City views; b) Mountain views

The famous Sevlievo's barracks **(fig. 3)**, built on 130 acres in 1892 and used to 2000, are now a place for rest. These barracks have preserved the history and heroism of the 9th Artillery Regiment's soldiers and officers **(fig. 4)**, and a hundreds of conscripts that have been trained in Sevlievo's garrison for 111 years (http://www.sevlievo.bg/).



Fig. 3. The famous Sevlievo's barracks



Fig. 4. Sevlievo's garrison

The Ministry of Defense had donated the barracks in 2004 to the municipality. Three years later, working design began, and after more three years, the project proposal was submitted for funding. Up to now, 25,000 m^2 of alleys and car parks, 1 decar of children's playgrounds, 15 new playgrounds, over 1 decar of water attractions, 109 flower gardens, 170 benches have been built. About 10 decares are grassed. There are 300 lighting fixtures, 250 bins. The idea is this park to combines the memory of the barracks and to be an attractive place for recreation and entertainment. All this led to the development of the green system in Sevlievo, that basically consists of three parks – "Chernichkite" (the biggest one), "Kazarmite" and "Aprilsko vastanie" (the smallest one). Every year the innovative green

space – park "Kazarmite" is a place for the national festival "Seme bulgarsko" – **fig. 5.** (http://www.elmazovi.com/semebalgarsko/). This festival is a result of the innovative and sustainable management of the Sevlievo Municipality (http://www.sevlievo.bg/).



Fig. 5. A poster for the 4-th National Bulgarian festival "Seme bulgarsko" in park "Kazarmite"

2. Discussion

As we mentioned above, Sevlievo is the first Bulgarian city, which turned it's abandoned barracks into a park. Now also 3 other cities (Blagoevgrad, Kardzhali and Stara Zagora) after Sevlievo had turned it's abandoned barracks into a park. In 2012 Kardzhali city have opened a new city park called "Arpezos - Sever" – **fig. 6**, built on a former barracks area of 64 decares. The new city park is featured with a solar fixtures, static and mobile surveillance cameras. It is funded under Operation Program "Regional Development". Thanks to this green place, Kardzhali municipality ranks among the leaders of the green cities in the country and in Europe (https://tourism.kardjali.bg/?pid=1,49).



Fig. 6. Park "Arpezos – Sever" in Kardzhali city

https://tourism.kardjali.bg/?pid=1,49

The newest (built 2015th) and modern park in Stara Zagora – "Former Artillery barracks" is located in the western part of the city – **fig. 7**. It is built on 70 acres, replacing the former artillery barracks. The grassy park areas are about 35 000 square meters. Many

young trees, shrubs and seasonal flowers are planted, two rock gardens are separated. A central fountain was built in the park - the largest in Stara Zagora. It is covered with fiberglass tiles, and in the whole circle are located the water effect sprays. Separate fountains (in the west and south) and a "Japanese lake" with a garden are made separately. In the park there are also: playgrounds with an area of more than 1000 square meters (meeting the most up-to-date safety requirements), four small soccer fields, basketball, volleyball, badminton and tennis, open stage various events, a mini zoo with different bird species. Everywhere the environment is accessible for disabled people, where invalid ramps are required. In the southeastern part of the park there is a car park for 200 cars (http://poseti.guide-bulgaria.com/a/1340/park_former_artillery_barracks.htm).



Fig. 7. Park "Former Artillery barracks" in Stara Zagora city

In 2017 was founded in Blagoevgrad city a park, called "Makedonia", which was again a former barracks territory, had turned into a green space for rest and entertainment (http://www.blgmun.com/). It is interesting that Sevlievo is not a district city like Blagoevgrad, Kardzhali and Stara Zagora, but was the first in this sustainable green management of abandoned barracks, turned into a green park space.

3. Conclusion

To date, park "Kazarmite" is a green place for recreation, sports and entertainment – **fig.8.** What has been achieved so far is (http://www.bgregio.eu):

- The lighting of the park, as well as the new sidewalks, allow easier access for disabled people.
- There is a network of alleys with park furnishings.
- Children's playgrounds, picnic areas, fountain and water pedal are built.
- Fountains, flower beds, garden benches, waste bins are installed.

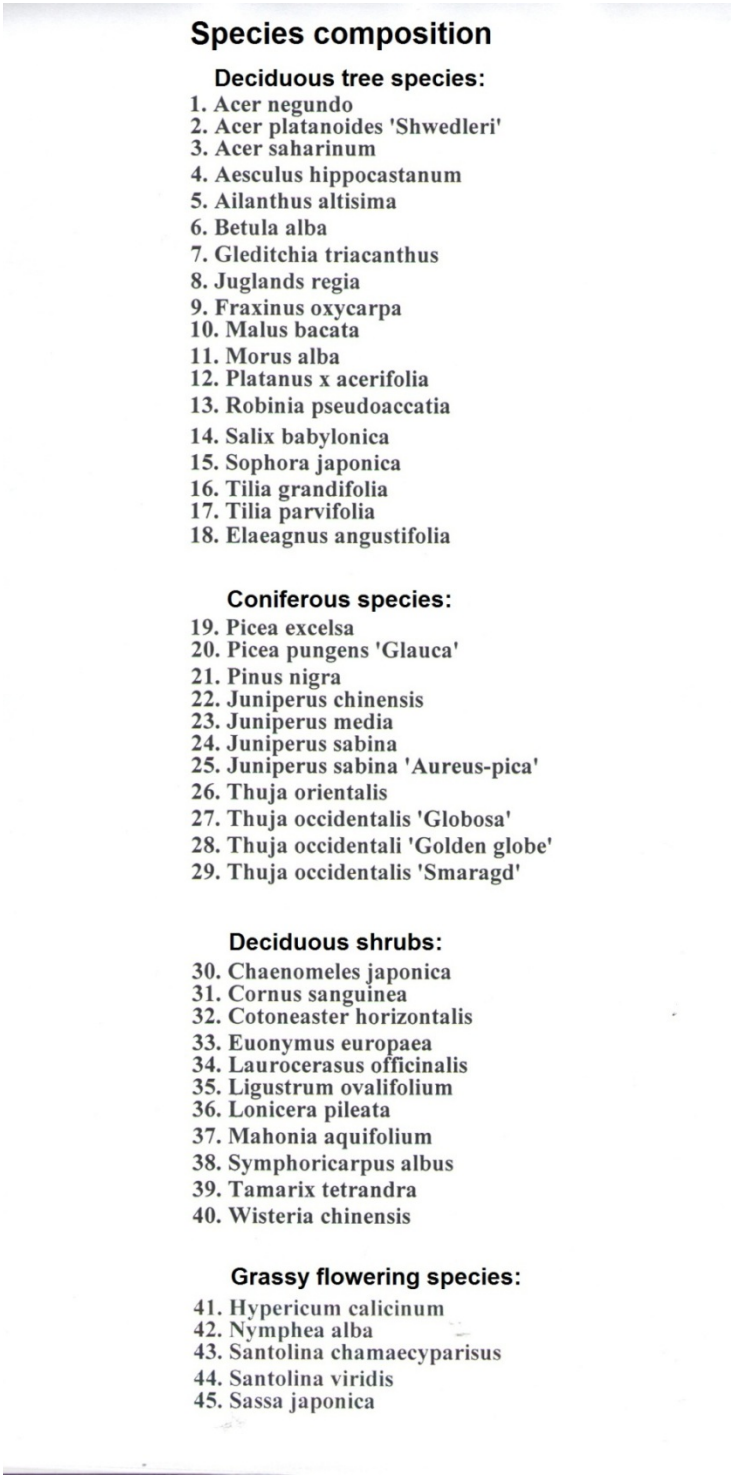
The existing vegetation is preserved, with further afforestation with deciduous, coniferous tree species, shrubs and new grassing – **fig. 9.**



Fig. 8. View of the Plant Diversity of Park "Kazarmite"

In the historical development of Sevlievo as a city, there is a big contribution to the Friday market. He was widely known, and traders from different parts of Bulgaria came to him. Since 1922, the traditional "Petkovden Autumn Fair" has begun, which has grown into the Sevlievo Festivals, and since the end of the autumn, the traditional "Pumpkin Feast" has been held. Sevlievo is a city that jealously preserved it's cultural and historical heritage. All the architectural and memorial real cultural values from the Renaissance are periodically restored by different projects or by financial means from the municipal budget. Because of it's rich revival history, Sevlievo is a member of the National Association of Bulgarian Revival Cities (http://www.sevlievo.bg/bg/article/55/sevlievo_v_istoria.html).

All this basis is one of the reason that Sevlievo is a sustainable city with first innovative green park in Bulgaria, that was built on the abandoned barracks of the city.



Species composition

Deciduous tree species:

1. Acer negundo
2. Acer platanoides 'Shwedleri'
3. Acer saharinum
4. Aesculus hippocastanum
5. Ailanthus altisima
6. Betula alba
7. Gleditchia triacanthus
8. Juglands regia
9. Fraxinus oxycarpa
10. Malus bacata
11. Morus alba
12. Platanus x acerifolia
13. Robinia pseudoaccatia
14. Salix babylonica
15. Sophora japonica
16. Tilia grandifolia
17. Tilia parvifolia
18. Elaeagnus angustifolia

Coniferous species:

19. Picea excelsa
20. Picea pungens 'Glauca'
21. Pinus nigra
22. Juniperus chinensis
23. Juniperus media
24. Juniperus sabina
25. Juniperus sabina 'Aureus-pica'
26. Thuja orientalis
27. Thuja occidentalis 'Globosa'
28. Thuja occidentali 'Golden globe'
29. Thuja occidentalis 'Smaragd'

Deciduous shrubs:

30. Chaenomeles japonica
31. Cornus sanguinea
32. Cotoneaster horizontalis
33. Euonymus europaea
34. Laurocerasus officinalis
35. Ligustrum ovalifolium
36. Lonicera pileata
37. Mahonia aquifolium
38. Symphoricarpus albus
39. Tamarix tetrandra
40. Wisteria chinensis

Grassy flowering species:

41. Hypericum calicinum
42. Nymphea alba
43. Santolina chamaecyparisus
44. Santolina viridis
45. Sassa japonica

Fig. 9. Afforested plant species in park "Kazarmite"

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НАПРАВЛЕНИЕ 3

Исторически строителни материали и тяхното диагностициране; Нови материали за опазване на културно-историческото наследство; Исторически структури и техния мониторинг; Нови технологии за опазване на културно-историческото наследство.

TOPIC 3

Historic materials and their diagnostics; New materials for safeguarding culture heritage; Historic structures and their monitoring; New technologies for safeguarding culture heritage.

A NUMERICAL APPROACH FOR THE SEISMIC UPGRADING BY TIES OF HISTORIC REINFORCED CONCRETE BUILDINGS UNDER SHEAR EFFECTS

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Abstract:
A numerical approach is presented for the seismic upgrading of existing Cultural Heritage industrial reinforced concrete (RC) structures by using cable elements (tension-ties). Special attention is given to shear-effects, common to old RC structures, which have been designed and constructed before the use of current aseismic codes. A double discretization, in space by the Finite Element Method and in time by a direct incremental approach, is used. The unilateral behaviour of the cable-elements, which can undertake tension stresses only, is strictly taken into account and results to inequality constitutive conditions. Finally, using damage indices, the optimal strengthening version of cable-systems is chosen for the case of multi-storey RC frames under seismic sequences of multiple earthquakes.

Key words: *Seismic Upgrading of Historic RC Structures, Strengthening by Ties, Flexure-Shear failure effects, Multiple Earthquakes.*

1. Introduction

Old existing industrial reinforced concrete (RC) structures have been usually designed and constructed without taking into account the requirements imposed by the modern seismic codes [1-4]. So, the flexural failure of their RC structural members can be strongly influenced by shear effects, as building damages due to recent earthquakes have shown.

Many of the above RC structures in Greece are considered as elements of the recent built Cultural Heritage [5]. Such structures are subjected to non-usual extremal actions (seismic, environmental etc.), which can cause significant strength degradation and damages. To overcome such strength degradation effects, various repairing and

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strengthening procedures can be used for the seismic upgrading of existing RC buildings [1-5]. Among the seismic upgrading methods, cable-like members (ties) can be used as a first strengthening and repairing procedure [6-8].

It must be emphasized that the cable-members can undertake tension, but buckle and become slack and structurally ineffective when subjected to a sufficient compressive force. So, in the mathematical problem formulation, the constitutive relations for cable-members include also inequality conditions, and the problem becomes a high nonlinear one. For the strict mathematical treatment of the problem, the concept of variational and/or hemivariational inequalities can be used and has been successfully applied [10,11]. As concerns the numerical treatment, non-convex optimization algorithms are generally required [6-8, 10-11].

In the present study, the earthquake analysis of existing historic reinforced concrete (RC) building frames, which have been environmentally degraded and after their seismic assessment have to be strengthened by cable elements, is numerically investigated. The unilateral behaviour of the cable-elements and the non-linear behavior of the RC elements, as well as the shear effects, are taken into account. The finite element method is used for space discretization in combination with a time discretization scheme, and damage indices are computed in order the optimal ties-system to be selected.

2. Method of analysis

2.1. Problem formulation and numerical treatment

The RC frame structural system is discretized in space by using finite elements [15,16,21]. The usual frame elements are used for the reinforced concrete frame. On the other hand, for the cable strengthening system, pin-jointed bar elements are used. The behaviour of both, the cable elements and the non-linear RC elements, includes loosening, elastoplastic or/and elastoplastic-softening-fracturing and unloading - reloading effects as depicted in Fig. 1.

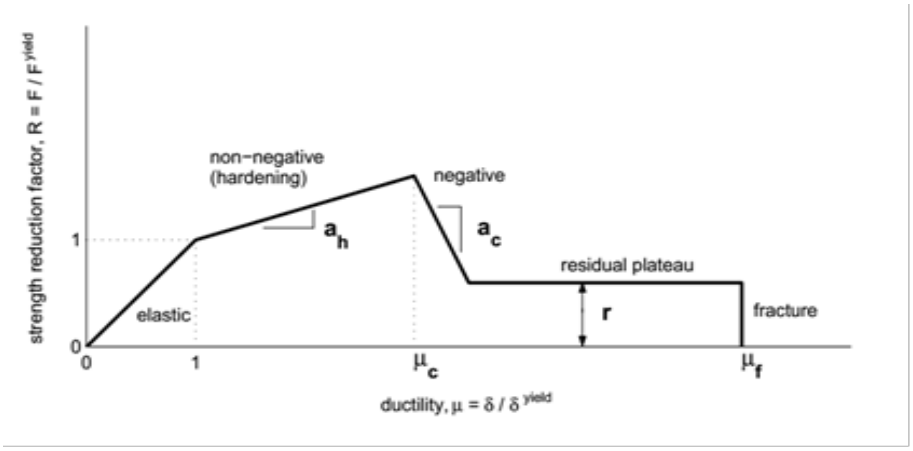


Fig. 1. The constitutive piecewise linearized diagramme (backbone) concerning generalized force quantity versus generalized displacement quantity [9].

All these constitutive characteristics can be expressed mathematically by non-convex relations of the general form:

$$s_i(d_i) \in \hat{\partial} S_i(d_i) \ . \qquad (1)$$

Here s_i and d_i are generalized stress and deformation quantities, respectively, $\hat{\partial}$ is the generalized gradient and S_i is the superpotential function, see Panagiotopoulos [11].

Next, when the cable-elements are taken into account, the dynamic equilibrium for the structural system is written in incremental matrix notation:

$$\boldsymbol{M\,\Delta\ddot{u}+C\Delta\dot{u}+(K_T+G)\Delta u = f - M\,\Delta\ddot{u}_g + T\,\Delta s} \qquad (2)$$

Here $\boldsymbol{u}$ and $\boldsymbol{f}$ are the displacement and the loading forces (time dependent) vectors, respectively. The damping and tangent-stiffness terms, $\boldsymbol{C}$ and $\boldsymbol{K_T(u)}$, respectively, concern the general non-linear case. By $\boldsymbol{s}$ is denoted the cable elements stress vector and $\boldsymbol{T}$ is a transformation matrix. $\boldsymbol{G}$ is the geometric stiffness matrix, by which P-Delta effects are taken into account. Dots over symbols denote derivatives with respect to time.

The system of the above relations (1)-(2), combined with the initial conditions, provide the problem formulation, where, for given $\boldsymbol{f}$ and/or $\ddot{\boldsymbol{u}}_{\boldsymbol{g}}$, the vectors $\boldsymbol{u}$ and $\boldsymbol{s}$ have to be computed. Relevant computational approaches are described in [12,13]. After piecewise linearization of the above constitutive relations (1), in each time-step Δt a relevant non-convex linear complementarity problem of the following matrix form is eventually solved :

$$\mathbf{v} \geq \mathbf{0}\,, \qquad \mathbf{A\,v + a} \leq \mathbf{0}\,, \qquad \mathbf{v}^{\mathbf{T}}\,.\,(\mathbf{A\,v + a}) = 0. \qquad (3)$$

Here **v** is the vector of unknown unilateral quantities at the time–moment t, $\mathbf{v}^{\mathbf{T}}$ is the transpose of **v**, **a** is a known vector dependent on excitation and results from previous time moments (t- Δt), and **A** is a transformation matrix.

Concerning the incremental approach of eq. (2), the structural analysis software Ruaumoko [16] is used herein. This software uses the finite element method and permits an extensive parametric study on the inelastic response of structures [12].

2.2. Effects due to shear and to concrete cracking

During a seismic excitation, the RC structural elements can appear flexural, shear or combined flexural-shear failure mode. Considering shear effects, the RC structural elements are classified [1] according to their shear span-to-depth ratio:

$$a_S = \frac{L_S}{d} = \frac{M}{Vd}\,, \qquad (4)$$

where $L_s = M/V$ is the *shear length* corresponding to the considered member end, M the bending moment, V the shear force and d the section depth (height). In the case of RC frame columns or beams, the shear length L_s is equal to one-half of the clear length of the examined structural element according to EC8 [3,4].

Due to shear effects, the RC structural elements appear one of the three following behaviours [1] according to their shear span-to-depth ratio:

1. For $a_s \geq 7.0$, bending prevails, and in this case bending failure occurs before any shear failure, no matter if shear reinforcement exists or not.
2. For $2.0 \leq a_s \leq 7.0$, the failure mode depends on the shear reinforcement of the web.
3. For $a_s \leq 2.0$, the case of short R/C columns is considered, where a special design procedure must be followed so that an explosive cleavage failure of the short column is avoided.

As had been remarked for bridge structural systems [14], in general the shear effects can reduce the available plastic curvature ϕ_p of critical sections. Due to shear effects in plastic hinges, the flexural ductile behaviour can be modified to a flexural-brittle one. Thus a shear failure, and a so-caused reduction of the curvature ductility $\mu_\varphi = \varphi_u/\varphi_y$, can be eventually appeared or not. This must be checked continuously during the seismic excitation, in order the real response of the historic structure to be correctly estimated.

Under a seismic excitation, the RC structural elements have a linear-elastic behavior until one or two of their critical end-regions, after cracking, enter to the yielding state and plastic hinges are appeared. These effects of cracking on columns and beams are estimated by applying the guidelines of Eurocode 8, part 3 [2,3] and Greek Retrofitting Code [4]. So, the effective flexural stiffness E_cI_{eff} is given by the EC8-formula [3-5]:

$$E_{\mathrm{c}} I_{\mathrm{eff}} = \frac{M_{\mathrm{y}} \cdot L_{\mathrm{s}}}{3 \cdot \theta_{\mathrm{y}}}. \tag{5}$$

Here M_y and $\theta_y$are the flexural moment and the chord rotation at yield, respectively, which are calculated by EC8 formulas given in [3,4].

On the other hand, the available cyclic shear strengthV_R (in MN), corresponding to the considered member end, is decreased with the incremental demand plastic chord rotation $\theta_{p,d}$ according to the following semi-empirical experimental expression given by Eq.(A.12) of Eurocode EC8, see [3,4, 17,18]:

$$V_{\mathrm{R}} = \frac{1}{\gamma_{\mathrm{el}}} \cdot \left[\frac{h - x}{2L_{\mathrm{s}}} \lambda_1 + (1 - 0.05\lambda_2) \cdot \left[0.16\lambda_3 (1 - 0.16\lambda_4) A_{\mathrm{c}} \sqrt{f_{\mathrm{cm}}/\mathrm{CF}} + V_{\mathrm{w}} \right] \right]. \tag{6}$$

Here, γ_{el} is a safety factor that is taken equal to 1.15 for primary seismic structural elements (due to scattering of the experimental values) and is taken 1.00 for secondary seismic members. x is the compression zone depth (in meters) that is known by the sectional analysis, CF is the Confidence Factor according to Table 3.1 of EC8 [3,17]; V_W is the contribution of the transverse reinforcement to shear strength, taken as being equal to $V_{\mathrm{w}} = \rho_{\mathrm{w}} \cdot b_{\mathrm{w}} \cdot z \cdot f_{\mathrm{yw,m}}/\mathrm{CF}$ for cross-section with rectangular web of width b_W; ρ_W is the transverse reinforcement ratio that is given by $\rho_{\mathrm{w}} = A_{\mathrm{sw}} \cdot \ell_{\mathrm{w}} / h_{\mathrm{c}} \cdot b_{\mathrm{c}} \cdot s_{\mathrm{h}}$, where ℓ_{w} is the total length of the stirrups, A_{sw} is the steel section area of the stirrup, h_C and b_C the dimensions of the confined core of the section and $s_h$is the centerline spacing of stirrups.

The other parameters in eq. (6) are: $\lambda_1 = \min N, \; 0.45 f_{\mathrm{cm}} A_{\mathrm{c}}$; $\lambda_2 = \min(5, \mu_{\Delta}^{\mathrm{p}})$; $\lambda_3 = \max(0.5, 100\rho_{\mathrm{tot}})$; $\lambda_4 = \min(5, a_{\mathrm{s}})$, where N is the axial force in [MN], that is positive for compression, while when the axial force is tensional then it is taken zero; A_C=$b_W d$ for rectangular sections with b_W as width of compression zone and d is the depth of the tension reinforcement in meters; f_{cm} is the concrete compressive strength (mean value) in [MPa]; $\mu_{\Delta}^{\mathrm{p}} = \theta_{\mathrm{p}}/\theta_{\mathrm{y}}$; ρ_{tot} is the total longitudinal reinforcement ratio (tensional, compression and intermediate; and a_s is the contemporary shear ratio.

In order to define the final elastic-plastic diagram of Moment-Chord Rotation (M-θ) of the critical section at the considered member end, it must be checked which type of failure precedes, the flexure-failure or the shear-failure [17]. Thus, using the known value of the shear strength V_R from eq.(6), the moment $M_{u,v}$ at the critical section due to V_R is calculated:

$$M_{\mathrm{u,v}} = L_{\mathrm{s}} \cdot V_{\mathrm{R}}. \tag{7}$$

When $M_{u,v}$>M_y, i.e. when$M_{u,v}$ is greater than the flexural yielding momentM_y, then the flexural failure precedes the shear one. In that case, the final elastic-plastic diagram of Moment-Chord Rotation*(M-θ)* is given by Figure 2(a).

On the contrary, when $M_{u,v}$<M_y, i.e. when$M_{u,v}$ is smaller than the flexural yielding momentM_y, then the shear failure precedes the flexural one. In this case, the final elastic-plastic diagram of Moment-Chord Rotation *(M-θ)* diagram of the considered member end is given as the curve OABCD of Figure 2(b) according to sect.7.2.4.2 of KANEPE [4].

After the above check, the final (corrected) values of M_y and θ_y are used in eq. (5).

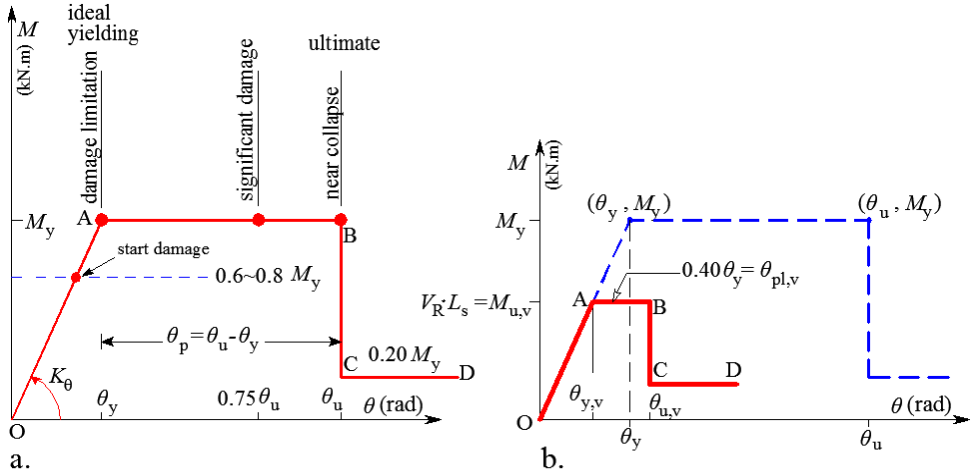


Fig. 2. Moment-Chord Rotation Diagram (a) for ductile failure and (b) for shear failure (from Makarios [17]).

2.3. Choice of the optimal system of ties by using damage indices

The decision about a possible strengthening for an existing RC structural system can be taken after an assessment realization [1-3].

Here the assessment is based on a relevant evaluation of suitable damage indices [12]. After Park/Ang [19], the *local* damage index DI_L is computed by the following relation:

$$DI_L = \frac{\mu_m}{\mu_u} + \frac{\beta}{F_y d_u} E_T \tag{8}$$

where: μ_m is the maximum ductility attained during the load history, μ_u the ultimate ductility capacity of the section or element, β a strength degrading parameter, F_y the yield generalized force of the section or element, E_T the dissipated hysteretic energy, and d_u the ultimate generalized deformation.

The Park/Ang *global* damage index is given by the following relation:

$$DI_G = \frac{\sum_{i=1}^{n} DI_{Li} E_i}{\sum_{i=1}^{n} E_i} \tag{9}$$

where DI_{Li} is the local damage index and E_i the energy dissipated at location i , and n the number of locations at which the local damage is computed.

3. Numerical example

3.1. Description of the considered RC structural system.

The RC frame of Fig. 3, denoted as System F0, is element of an old industrial Cultural Heritage RC building and will be strengthened by a ties-system. The frame is of concrete class C 16/20, it was designed according to old Greek building codes (before 1985), and so is characterized as a flexure-shear critical one [18,20]. The beams have a total vertical distributed load 30 KN/m (each beam).

The frame F0 is to be strengthened by ties after its seismic assessment by damage indices. The cable-bracing system F6, shown in Fig. 4, is proposed as optimal after a

parametric investigation by damage indices and according to service conditions. The cable elements have a cross-sectional area $F_c = 18$ cm^2 and they are of steel class S220 with yield strain $\varepsilon_y = 0.11$ %, fracture strain $\varepsilon_f = 2$ % and elasticity modulus $E_c = 200$ GPa. The cable constitutive law, concerning the unilateral (slackness), hysteretic, fracturing, unloading-reloading etc. behavior, is depicted in Fig. 5.

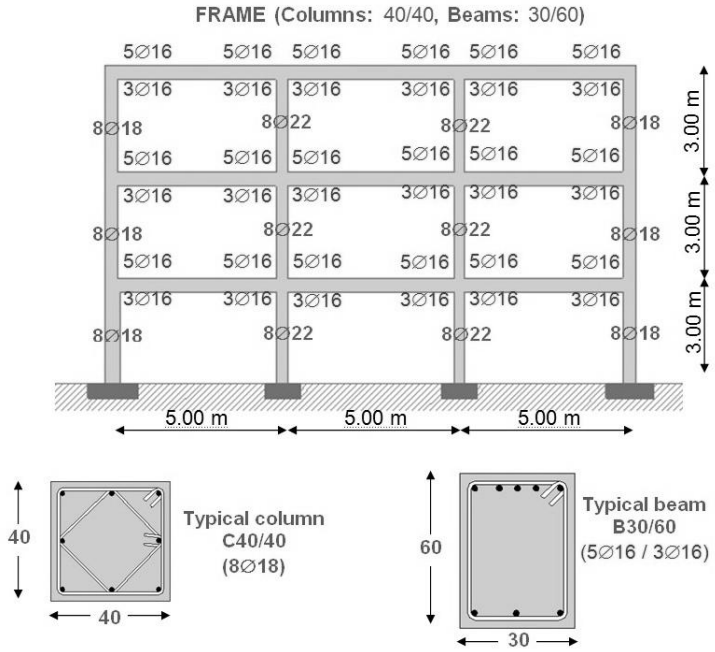


Fig. 3. System F0: the old industrial RC frame without cable-strengthening.

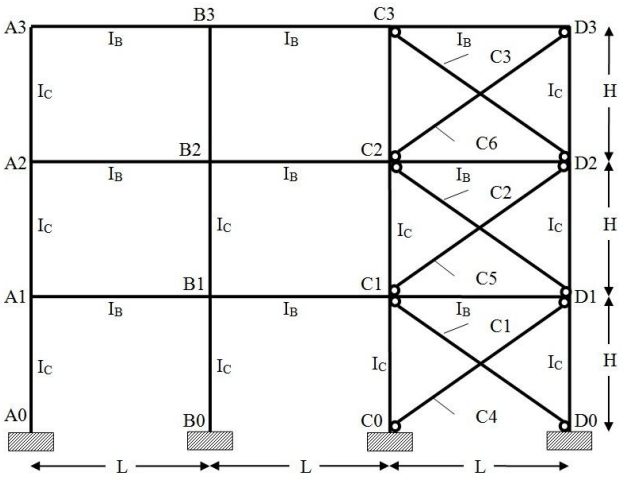


Fig. 4. System F6: the old industrial RC frame with X-braces cable-strengthened.

The systems F0 and F6 of Fig. 3 and Fig. 4, respectively, are considered to be subjected to a multiple ground seismic excitation, presented and discussed in [21]. The complete list of these earthquakes was downloaded from the strong motion database of the Pacific Earthquake Engineering Research (PEER) Center and appears in Table 1

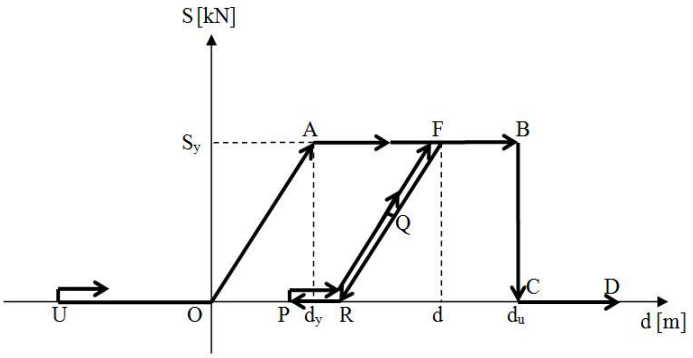


Fig. 5. The piecewise linearized constitutive law of the unilateral behavior of the cable-elements.

Table 1. Sequential earthquakes data

No	Seismic sequence	Station	Comp	Date (Time)	Magnitude (M_L)	Recorded PGA(g)	Norm. PGA)
1	Mammoth Lakes	54099 Convict Creek	N-S	1980/05/25 (16:34)	6.1	0.442	0.200
				1980/05/25 (16:49)	6.0	0.178	0.081
				1980/05/25 (19:44)	6.1	0.208	0.094
				1980/05/25 (20:35)	5.7	0.432	0.195
				1980/05/27 (14:51)	6.2	0.316	0.143
2	Chalfant Valley	54428 Zack Brothers Ranch	E-W	1986/07/20 (14:29)	5.9	0.285	0.128
				1986/07/21 (14:42)	6.3	0.447	0.200
3	Coalinga	46T04 CHP	N-S	1983/07/22 (02:39)	6.0	0.605	0.165
				1983/07/25 (22:31)	5.3	0.733	0.200
4	Imperial Valley	5055 Holtville P.O.	HPV 315	1979/10/15 (23:16)	6.6	0.221	0.200
				1979/10/15 (23:19)	5.2	0.211	0.191
5	Whittier Narrows	24401 San Marino	N-S	1987/10/01 (14:42)	5.9	0.204	0.192
				1987/10/04 (10:59)	5.3	0.212	0.200

3.2. Representative results

Representative results of the numerical investigation are presented in next Table 1. In column (1), Event E_1 corresponds to Coalinga seismic event of 0.165 normalized PGA of Table 1 and Event E_2 to 0.200 normalized PGA. The sequence of events E_1 and E_2 is denoted as Event (E_1+ E_2). In column (2) the Global Damage Indices and in column (3) the Local Damage Index DI_L for the bending moment at the left fixed support of the frames are given. Finally, in the column (4), the maximum horizontal top displacement u_{top} (absolute value) is given. As the table values show, multiple earthquakes generally increase, in an accumulative way, the response quantities.

Table 2. Representative response quantities for the frames F0 and F6

FRAMES	EVENTS	DI_G	DI_L	u_{top} [cm]
(0)	(1)	(2)	(3)	(4)
F0	Event E_1	0.254	0.184	1.348
	Event E_2	0.484	0.548	1.588
	Event (E_1+ E_2)	0.648	0.687	1.6743
F6	Event E_1	0.118	0.037	0.845
	Event E_2	0.352	0.484	1.244
	Event (E_1+ E_2)	0.379	0.505	1.257

4. Concluding remarks

The herein presented numerical approach can be used for the effective investigation of the inelastic seismic behaviour of existing historic RC building systems strengthened by cable elements. Shear effects are taking into account. As the results of a numerical example have shown, the optimal strengthening version of cable-bracings can be decided by computing necessary damage indices.

Acknowledgment

The co-author K. Liolios was partly supported for this work by the Bulgarian Academy of Sciences through the "Young Scientists" Grant No. DFNP-97/04.05.2016.

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MONITORING OF APPLIED PROTECTIVE STRUCTURES AND THE EVALUATION OF THEIR INFLUENCE ON ARCHAEOLOGICAL SITES

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Abstract: *Protection of archaeological sites in the world and Europe is a continuous process, which is intended to prevent devastation of the monument, creating conditions for work on their conservation and a museum "in situ". Some archaeological sites in Serbia are monuments of extraordinary significance.*
In recent years, five archaeological sites were covered with seven shelters. The need for the construction of new protective structures still exists.
The application of different types of protective structures produced certain effects on the sites. This paper presents elements which are the basis for continuous monitoring, and an appropriate method for assessing the efficacy of existing protective structures and the evaluation of any altered conditions on the site itself (the effects of the presence of water in different physical states, instability of micro-climatic conditions: temperature, humidity and luminescence, as well as biological effects).
The aim of this paper is to form objective databases and the initiation of a "Shelter Management System" on the archeological sites in Serbia as well as indicate the necessary requirements for the application of the assessment method.

Key words: *Protective structures, Archaeological site, Methods of assessment, Appropriate database, Shelter Management System*

1. Introduction

Hundreds of archaeological sites in the world are covered with modern structures which offer protection for the sites from the influence of the sun, rain, wind and snow, and at the same time offer protection for the visitors as well. In our profession these structures are known as "shelters". The applied geometric forms, the construction systems and material used to manufacture these shelters differ from the traditional solutions (massive brick walls, wooden or metal roof structures...). It is often the case that there emerge various combinations of traditional and contemporary material, ranging to the contemporary high-tech achievements (steel girders, plywood, steel rope, textile membranes, polycarbonates...).

In addition, archaeological sites which are protected with shelters differ from one another and even date back from paleolithic to byzantine archaeological remains, ranging from earthen structures, to the remains of walls made of clay and rock, to mosaics which

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predominate (from the Ancient Greek, Roman and the Byzantine period). The trend of the presentation of mosaics developed intensely during the 1990s with the primary aim of presenting the all-encompassing architectural-engineering context and the authentic artistic aims of that time (Stewart et al. 2005). At the beginning of the new millennium, was intense work was done on the development of protective strategies of architectural sites and on the erection of shelters above them.

A shelter on an architectural site represents a structure which is meant to satisfy the basic, but also the complex requirements of the conservation of the architectural remains, their interpretation and presentation "in situ". Considering that we are dealing with mutually contradictory project conditions, the complexity of the set task is clear. Instructions on how to plan a shelter on archaeological sites with mosaics were prepared by our colleagues from the Conservation Department of the Israel Antiquities Authority in cooperation with the Getty Conservation Institute and English Heritage (Alef, 2013). The methodology which was developed during the course of the joint project includes "rapid assessment" of the state of the mosaics and the influence of the construction on the mosaics through continued monitoring. At the same time, a standard questionnaire was established – a survey questionnaire on the state of the site, based on which relevant data is gathered (Cetin, 2013).

In the study we used a survey questionnaire on the state of the site, where the elements presented in the responses were used in the method for evaluating the efficacy of the existing protective constructions in relation to the evaluation of the altered conditions on the site itself. The aim of the paper is to evaluate the implemented method for the evaluation of the efficacy of the shelter for a specific object.

2. Applied protective structures and the evaluation of their influence on archaeological sites

The basic relevant conditions which the protective structures are meant to satisfy are primarily their adhering to the characteristics of the site itself (the historical period of the archaeological remains, the characteristics of the material and the structure which should be protected, the optimum microclimatic conditions for working on the conservation and preservation of the archaeological remains), the geo-morphological characteristics in the vicinity of the site (the relief, the vegetation, proximity of any rivers/seas/lakes), the concept of the presentation of the sites to a wider audience (education, scientific research, tourists, marketing), the life span of the protective structure.

There are still no clearly defined guidelines for adopting a certain structural concept depending on the previously mentioned most important and influential factors. Architects have the freedom of choice in a constructive sense depending on the range of the protective structure, while in terms of the shelter material two opposite opportunities are presented which are inextricably linked with the adopted system of the supporting structure: the traditional and innovative.

These structures are meant to satisfy the strict requirements for the protection of archaeological sites from destructive meteorological influences for a period of several years. Direct contact of the uncovered archaeological remains with oxygen, radiation from the sun and the newly occurring – changing conditions regarding humidity can cause various kinds of damage. Meteorological causes which might lead to damage include: rain (mechanical impact of the raindrops and the chemical dissolution of the material of the archaeological remains on the), wind (the mechanical pressure and abrasive effect on the surface), rays of the sun (thermal expansion and the difference in the dilatation of the sun drenched remains and remains which are sheltered from the sun (paint and pigment damage), ultraviolet rays of the sun (damage to the paint and pigment, frequent cycles of evaporation/condensation (the crystallization of salts in the material and their impact on the

surface of the archaeological remains), frost (destruction through contraction and thawing, especially of surfaces which are moist and saturated in water). The typical pathology of the damage is linked to the presence of water from precipitation or condensation in which aggressive matter is dissolved, primarily the salts which chemically decompose material.

The conditions of the project task should also contain the criteria which these structures must satisfy in terms of ventilation which should prevent condensation in the period from October to May and decrease any heating due to solar radiation from May to October, as well as natural lighting which does not change the solar spectrum. In a structural sense these structures must be stable regarding all the known load and external influences, must weigh as little as possible, be easy to maintain, be constructed out of modern and improved material, offer the possibility of dismantling and reconstruction on other locations, and enable the adaptation of the functional distribution, with the necessary condition that the budget for their construction in complex conditions is as low as possible.

Once the project is designed, it is necessary to evaluate the influences of the protective structures on the environment and the remains on the site itself so as to avoid the any influences and damage from increased humidity, thermal dilatation and condensation, heating as a cause of radiation, freezing, as well as the from the photometric aspect, which is performed by an evaluation of the state of the site by completing a questionnaire. Currently, the instrumental monitoring of the most important factors which influence the architectural remains is performed only in the shelter on the site of Drenovac near Paraćin (a meteorological station with a data logger which monitors and records the temperature and humidity of the air on the site at several different pressure zones). An evaluation of the state essentially relies on the objectivity and ability to perceive and compare any macroscopically noted changes in relation to the previously noted state, which in the methods of evaluation cannot present absolutely valid data.

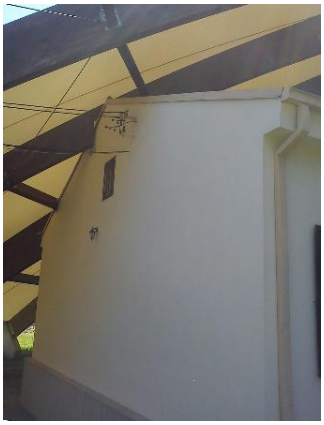
3. Protective structures on the site "MEDIJANA" in Niš

In Serbia there are many archaeological sites (189) which have been decreed immovable cultural property. Nineteen sites have been declared Cultural Heritage of Exceptional Importance, while only 25 have the status of cultural heritage of great importance (Vasic-Petrović et al. 2015). Closed protective structures of a more permanent character have only been built on five sites in the past ten years or so: Lepenski Vir near Majdanpek, Drenovac near Paraćin, Sirmium near Sremska Mitrovica, Viminacium near Stari Kostolac and Medijana near Niš (Kostić et al. 2015).

Based on the responses collected from the survey questionnaire, an identification card of protective structures has been compiled for the sites Medijana near Niš, Viminacium - Mausoleum, Viminacium - The North Gate, Viminacium - Thermae, Drenovac - Slatina Turkish fountain. Due to spatial constraints, we provide an overview of the Identity Card of the Site Medijana - Villa with peristyle (Table 1).

Table 1. Identification card of Mediana-Villa with a peristyle

ARCHAEOLOGICAL	Archaeological site Medijana near Niš, Serbia	 Foto 1. Inside look
	Characteristics of the site The ancient settlement erected in the vicinity of Niš mentioned for the first time in the 4th century has been decreed an immovable cultural goods of exceptional importance. It flourished in the time of the emperor Constantine the Great. The settlement is known for its residential objects, and includes thermae,	

<table>
<tr><td rowspan="5"></td><td colspan="2">as well as accompanying facilities including barracks, servants' quarters …</td></tr>
<tr><td>The identification of archaeological remains
The most important object on the location of Medijana is the Villa with a peristyle, which was mostly covered with a protective structure in 2014.</td><td rowspan="2"> 
Foto 2. External view</td></tr>
<tr><td>Period
The end of the 3rd and beginning of the 4th century A.D.</td></tr>
<tr><td colspan="2">Excavation and conservation work
The first registered data on the remains of the mosaic and the buildings date back from Felix Kanitz from the end of the 19th century, while the fist systematic excavations began in the 1930s.
The systematic excavations continued in 1959, 1979, 2011, and were completed in 2015.</td></tr>
<tr><td colspan="2" rowspan="3"> 
Foto 3. Overlap the two structures ("old" and "new")</td></tr>
<tr><td rowspan="3">PROTECTIVE STRUCTURE</td><td>Type
Shelter – Hybrid protective structure
Erected over the archaeological remains.</td></tr>
<tr><td>Architect / Construction company
Mile Veljković, A&E
/Institute for the protection of cultural monuments Niš
(architectural phase),
Dragan Zlatkov, MSc., A&E
/ProjektInženjering Niš
(construction phase).</td></tr>
<tr><td>Year erected
Begun in 2013, as yet incomplete</td><td>Range and covered surface
L=72,5m, P=10.000m^2</td></tr>
<tr><td colspan="3">Function and purpose
Protection from atmospheric influences (sun, rain, snow).
Work on the conservation of the mosaic, presentation to the public.</td></tr>
<tr><td rowspan="4">PROTECTIVE STRUCTURE</td><td colspan="3">Principles and project tasks
The project task is based on the idea of implementing very shallow arch constructions made of plywood with a cover made of textile membranes, with open sides to allow natural ventilation.</td></tr>
<tr><td colspan="3">Construction system and material
A shallow arch system with a range of 72,5m with arches each at a distance of e=5m, b/h=18/150cm
Plywood with segments bound with metal connectors at each third of the range.
A cover with a polyester membrane with PVC protective layers d=1mm.
The north and south dome-like parts are not completed, have not been covered. The construction is bound with semi-arches in the northern part by means of metal connectors along the first arch.</td></tr>
<tr><td colspan="3">Project requirements for the control of microclimatic parameters
No data.</td></tr>
<tr><td colspan="3">Monitoring internal / external microclimatic parameters
The pilot project of the Central Institute for Conservation from Belgrade from July</td></tr>
</table>

	2016 has been monitoring the temperature and humidity of the soil via a probe located immediately beneath the mosaic in the north-west part of the Villa.
	The problems of conservation and possible causes The National Institute for the Protection of Cultural Monuments does not allow the mosaics to be put on display until the necessary conditions for it have been met: the relocation of ventilation openings from which water often drips onto the mosaics in the peristyle of the Villa, covering the northern part of the cupola and its frontal northern façade, which allows the protection of the mosaics in the festival hall and side feast halls, and the prevention of the penetration of atmospheric conditions by constructing a side north-west wall. By inspecting the influences, detrimental effects were noted (of the wind, water dripping from the ventilation openings and condensation from the membrane and metal parts of the construction, as well as snow build-up and the penetration of snow and rain from the sides and from the front). The cited detrimental influences must be dealt with in order to prevent the devastation of the mosaics and frescos in order for them to be made available to the public. Until the completion of the structure, the mosaics will remain covered by sand.
	Modification of the protective structure to lessen the problem No modification is planned
Climatic conditions on the site Niš is located in an area with a continental climate (with all four seasons) with a tendency of growth of the average annual temperature expressed in extreme temperature differences, as well as considerable precipitation over shorter periods of time, which is a consequence of global climate change. Hydro meteorological statistics are based on the measurements made in the Fortress from 1951 to 2014 and will be presented for the year 2014: the average annual temperature was 13,0°C, which represents an extremely warm category. The lowest average temperature was noted in December, 2,8°C (cold), while the highest average monthly temperature was recorded in July 22,3°C (warm). The value of solar radiation was 91-100% of the normal radiation for 1900-2000 hours of sunlight every year. The average annual precipitation in Niš in 2014 was extremely high 950,2mm (lit/m^2), while the driest period was recorded in February 6,5 mm (extremely dry), and most of the rainfall residue was recorded in May (177,1mm – extremely rainy). The greatest recorded wind speed was 23,1 m/s while the greatest average wind speed for Niš was 19m/s. (Annual bulletin of the RHMZ).	

4. Necessary data for the evaluation of protective structures

In collaboration with archaeologists, architects, conservationists and engineers from other fields, civic engineers ensure the construction of protective structures which need to be functional, safe for use and sustainable. Sustainable protective structures are those which are at the same time archaeologically, ecologically and economically acceptable, but are also permanent. The usual expected life span of the protective structures is 50 years. However, the capital investments for archaeological sites often require a much longer life span. Considering that the current life span of protective structures is far below the required level, a change is needed in how to approach the design process, construction and maintenance of protective structures.

A planned and systematic approach to the process of protecting archaeological sites in the world is becoming increasingly more important. The older the infrastructure gets, the more resources are needed for it to be maintained in an acceptable manner, in order to avoid any undesirable consequences, which in the cases of treatment of heritage sites are not allowed. As a result, an adequate evaluation of the current state establishes the

necessary requirements for the evaluation of archaeological sites and their protective structures.

The empirical models and expert judgment represent the basis for most of the procedures and analyses in the decision-making phase, which for the methods of evaluation of protective structures cannot be considered absolutely valid data. For that reason the concept and functional design of a new method have been developed, which is used to quantify the evaluation of protective structures and the evaluation of their influence on the archaeological remains.

The method includes all the parameters which define archaeological sites with protective structures, as well as their status, and includes all the basic items in the database, as well as the factors of the increased spectrum of the contamination of the environment and the noted shortcomings.

The method classifies two different groups of elements which are being evaluated. The first group is made up of elements of the protective structure, the existence and efficacy of the drainage systems (the roof, surface and drainage of the sites), the efficacy of the insulation, the type and appropriateness of the ventilation system. The elements of these groups are quantified with an appropriate grade from a three-level scale, and if there are none or if they are superfluous they are given the grade 0. The state of the construction material of protective structures is evaluated on a five-level scale (in good shape, exposed, threatened, damaged or ruined as a result of age). In addition, in the first group we evaluate whether there is any superfluous humidity within the protective structure and what the sources of this increase in humidity are, as well as if the climate is stable in the protective structure.

What is evaluated is whether the protective structure is damaged from the influence of birds, insects (wasps, spiders, rodents (mice, rats, badgers), people, microbiological vegetation (algae, mold, lichen, moss) and vegetation (grass, and other woody plants).

We also evaluate whether there is any monitoring of the protective effects of the structure by monitoring the internal environment, external surroundings, giving an evaluation of the state of the archaeological remains and a comparison of the current state with the one on historical (earlier) photographs, as well as measuring humidity and the temperature of the air and soil. These influences are evaluated on a five-tiered scale (never, rarely, sometimes, very often and always). In addition, a five-level scale is used to evaluate the efficacy of the protective structure in the protection of archaeological remains and of mitigating the risks in the surrounding environment.

The other group of elements which are evaluated include the state of the archaeological remains located underneath the protective structure. What the archaeological remains underneath the protective closed structure are and what their frequency is can be quantified by the factor of frequency based on the percentage of the prevalence of mosaics, frescos, cement, stone, clay (blocks, bricks) and crumbling of the structures on the site. The state of the covered remains – ruins is evaluated on a five-level scale (in good condition, exposed, threatened, damaged and ruined). For mosaics, frescos and the other structures the phenomenon of deterioration and the severity of the aggravated state is evaluated on a five-level scale (very serious, serious, moderately serious, mild deterioration and very mild deterioration, and if there is none the awarded grade is 0).

5. The method of evaluation

Visual evaluations are used to identify the damage, and based on the determined state an inspection is then used to evaluate state which is entered into the formed database. The evaluation of the state describes the severity and extent of the damage. Each of the cited described evaluations was quantified by means of an appropriate numerical value. Based on that we can calculate the *Archeological Site Health Index*, which represents the basis for

the development of a *Shelter management system*, and thus the application of stochastic and other methods (Gligorijević 2016, Collines 1972) used to predict future states.

Inspections are performed every 5 years, even though in some cases evaluations may be needed over shorter periods of time. For newly constructed objects, the interval between the initial commission and the first inspection can even be up to10 years.

The inspection data such as the class of the state, the recommended intervention, the extent of the damage, details of individual cases of damage are compiled for each level of the hierarchy, and the inspection report is generated for all the examinations.

The prognosis of the deterioration of the state is calculated within probability theory with the help of Markov chains, which are defined for every type of element and type of damage. For each damage of a certain type, a catalog predicts an appropriate intervention. Standard interventions are predefined based on the evaluation of the state and process of damage (replacement, renewal, repair, etc.). The program provides recommendations for whether it is necessary to carry out certain interventions, or whether no measures should be taken. In addition, the owner should of his own volition decide at which level of the hierarchy certain standard interventions should be implemented.

6. Stochastic models

The theory of stochastic processes is increasingly being used in applied engineering and other applied sciences. The use of stochastic models makes a significant contribution in the field of modelling deterioration in the states of the infrastructure, due to exceptionally high uncertainty as well as coincidences which characterize the deterioration process affecting the state of the structures. The most frequently used technique for predicting the state of the infrastructure is the model of Markov chains[5].

6.1. Markov chains

One of the most popular stochastic techniques obtained from operational studies is the Markov process of decision making, which is today successfully being implemented in the system of managing bridge structures (2). This method is often used to develop stochastic models used to determine the states for various infrastructural objects.

Markov models of determining the state of protective structures are based on the concept of the definition of a state, in the sense of the evaluation of the state of constitutive elements of the shelter and the determination of the probability of the transition from one state to another. With the help of Markov chains we calculate the probability that certain elements of the shelter at some point of time can be found in a certain state. The states are discrete categories. The number of states in which a Markov process can be determined is final. Between two successive inspections a transition from a better to a worse state is possible for at most one level of the evaluation of the state.

The stochastic process is defined as the description of the change in the state of the system to some probability, over an accidental interval of time. This is the process in which the current behavior influences what will happen in the future. The behavior of the system is completely defined by states ($X_{(t)}$).

Generally speaking, the prediction of the distribution for the state *X(t)* which represents the function of all the previous changeable states *X(t-1), X(t-2)* is complicated. However, if the probability of the transition *p(t)* depends only on the previous state, then the process is known as the "Markov process".

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[5] Andrey Andreyevich Markov (14. 06. 1856 – 20. 07. 1922) was a Russian mathematician and a member of the Russian academy of science. He is best known for his research of stochastic processes, which were later known as Markov processes.

The Markov process is a stochastic process where the probability of the transition from the given state *X(t)* into a future state *X(t+1)*, depends only on the current state, and not on the way in which the current state was achieved. The Markov processes should meet the following requirements:

- the system is defined by a sequence of finite states and can only assume a single state at a given moment,
- the initial state of the system is known as is the division of probability of the initial state is well known,
- it is assumed that the probabilities of the transition are stationary over time and independent of the way in which the state itself was achieved.

The probability of the transition takes the following formulation

$$P_{ij}=P[X_t=i, X_{t+1}=j] \quad (1)$$

The transition from one state into another is represented in matrix form (n x n) which is also known as the transition probability matrix *P*, where *n* is the number of states. The transition probability matrix takes the following form:

$$\mathbf{P}=\begin{bmatrix} p_{11} & p_{12} & 0 & 0 & \cdot & 0 \\ 0 & p_{22} & p_{23} & 0 & \cdot & 0 \\ 0 & 0 & p_{33} & p_{34} & \cdot & 0 \\ 0 & 0 & \cdot & \cdot & \cdot & 0 \\ 0 & 0 & \cdot & \cdot & p_{n-1\,n-1} & p_{n-1\,n} \\ 0 & 0 & \cdot & \cdot & 0 & 1 \end{bmatrix} \quad (2)$$

In our case, the transition probability matrix **P** is a squared matrix with the elements p_{ij}, where:

$$0 \le p_{ij} \le 1 \quad (3)$$

Every element in this matrix p_{ij}, represents the probability that the component of the system will transition from the state "*i*" in the moment t_n into the state "*j*" in the moment $t_{n+1} > t_n$ (during a certain transitional period of time).

The assumption is that during a discreet period of time (from t_n to t_{n+1}), the process could either stay in the same state or transit into the first following higher state, which gives the final form to the transition probability matrix.

The elements of the transition probability matrix p_{ij}, should satisfy the equation (4).

$$\sum_{j=1}^{n} p_{ij} = 1 \quad (4)$$

If the present or initial state is known, that is **p(0)** = [$p_1(0)$ $p_2(0)$ $p_3(0)$ … $p_n(0)$], then the future state might be predicted at any moment *t*.

The probability vector of the initial state is formed during the first set of evaluations of the protective structures. The probability vector of the future state is obtained by multiplying the probability vector of the initial state **p(0)** with the transition probability matrix **P** to the power of *m* , as is presented in the matrix equation (5).

$$p(t_n) = p(0) \cdot P^m \quad (5)$$

where:

$$p(t_n) = [p_1(t_n)\, p_2(t_n)\, p_3(t_n) \ldots p_n(t_n)] \quad (6)$$

The probability vector of the future state $p(t_n)$ after *m* time intervals (years).

The Markov models of deterioration are applied in numerous advanced systems of bridge management, since their application has led to significant improvement in the modelling of deteriorated states, but are still based on assumptions and have certain limitations:

- Markov models assume that the past states do not have any effect on the predicted states.
- Markov models assume discrete transitions of time intervals, the constant population of the infrastructure and stationary probability of transition.
- the probability of transition is evaluated based on the subjective engineering evaluation and often requires frequent upgrading when new data have been obtained.

By applying this method, an evaluation of the influences of the protective structures on the architectural sites where they were erected and the evaluation of the improvement of the conditions for preserving the archaeological remains, the creation of the conditions for their better presentation to the professional and wide audience, as well as any work done on their conservation. Table 2 offers the preliminary results based on two inspections:

Table 2 Preliminary Archeological Site Healt Index (ASHI) for sites under shelters

	Location - Shelter	ASHI
1	Medijana – Villa with peristyle	157
2	Viminacium - Mausoleum	64
3	Viminacium – The north gate	65
4	Viminacium - Thermae	65
5	Drenovac - Slatina Turkish fountain	110

7. Conclusion

Based on the proposed method, a precise database is formed, which is made available to designers and investors. This allows their mutual positions to improve both in terms of the functional and constructional shape, the application of appropriate material, as well as the further monitoring of the effects on the monuments which are being protected and the prognosis of their future states by applying the stochastic method by means of Markov chains used to calculate the *Archeological Site Health Index*, which is the basis for the formation and development of the *Shelter management system*.

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CASE STUDY: RESTORATION AND STRUCTURAL STRENGTHENING OF HISTORIC BUILDING AT RAILROAD STATION "SKOBELEVO" ADOPTING TIMBER CONCRETE COMPOSITE (TCC) FLOOR SYSTEM

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***Abstract**: In this publication is presented a case study regarding application of TCC floor structure for restoration and structural upgrading of a historical building – railway station "Skobelevo", near City of Plovdiv, Bulgaria.Addition of a relatively thin concrete slab is an excellent structural solution for existing floor structures made of timber beams. The new composite structure is a very effective from structural point of view, possessing several times bigger bearing capacity and stiffness compared to the traditional timber floor grids. Additional advantages of a such composite structure are the significant increase of structural stability during seismic actions and high improvement of sound insulation, fire resistance and vibration resistance.Technique for strengthening and stiffening of existing timber floor structures through co-working concrete slab requires usage of appropriate shear connectors (dowels) for adequate transfer of longitudinal shear stresses between concrete slab and timber beam.Design basis and procedures for design of timber-concrete composite floor structures are developed based on Eurocode structural design system, especially БДС EN 1990:2002, БДС EN 1994-1-1:2005 and БДС EN 1995-1-1:2004.In parallel to the structural strengthening of the existing floor structures, respective global strengthening measures have been introduced to the bearing stone and brick masonry walls to meet the requirements of the current seismic regulations for seismically resistant structures.Current publication presents an overview about the whole process of designing and execution of structural rehabilitation, beginning with initial test loading and ending with a creation of a modern, safe and reliable building possessing highly increased service life.*

***Key words**: conservation, restoration, repair, timber concrete composite (TCC) structures*

1. Introduction

This paper presents one project concerning general reconstruction and strengthening of bearing structure of a two storey old building with stone masonry walls and timber floor grid (put in service around 1890) considering requirements of modern Standards and Codes ([1] and [2]) for design and execution of building structures.

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During this process of reconstruction and structural upgrade two main goals have been targeted.

Firstly, structural strengthening and retrofit of floor structures for design vertical load and increasing of their in-plane stiffness to match seismic Design Codes requirements.

Secondly, structural reconstruction and strengthening of the stone masonry walls has been performed for design vertical and seismic load, as applicable according to design standards.

In this paper the first task shall be presented as its application is of high importance for all kinds of old buildings with timber floors.

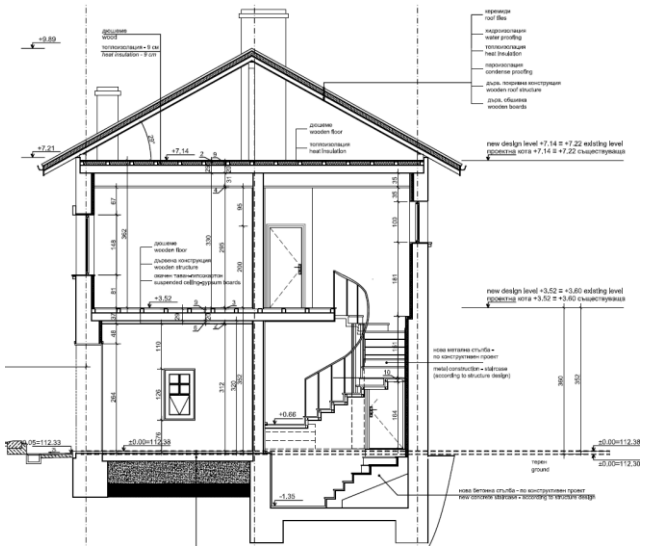


Fig. 1. Typical cross section of the building

2. Structural upgrade of floor structures

Floor structure at elev.+3.52 and under roof floor structure at elev. +7.14 consist of timber grid with clay, lime and sand filler between timber beams; above the beams the floor covering has been executed from wooden planks.

Layouts of these floor elevations are presented on Fig. 2 and Fig. 3 respectively.

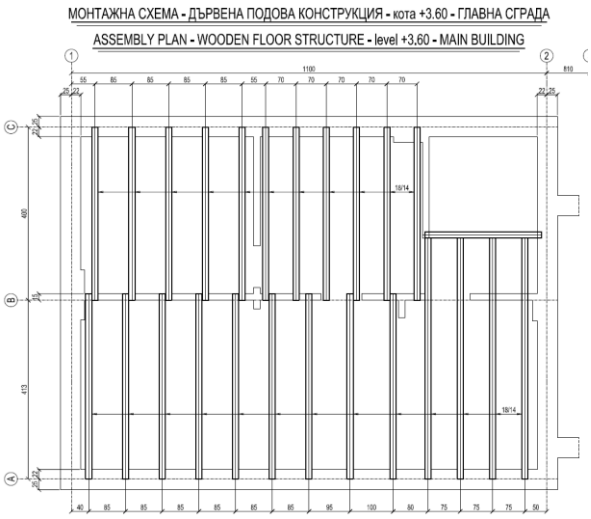


Fig. 2. Floor layout at elev. +3,52

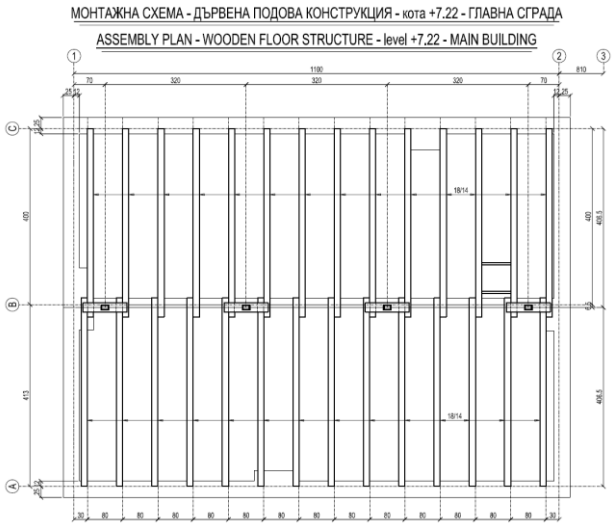


Fig. 3. Floor layout at elev. +7,14

Structural retrofit and upgrade of storey and under roof floor structures was realised by execution of timber concrete composite structures (TCC), keeping existing timber beams. The following upgrade measures have been undertaken:

1. Removal of all partition masonry walls with thickness 15 cm (non-bearing) above elev.+3.52;
2. Removal of all floor and ceiling cover, debris and filler materials between timber floor beams, exposing the beams for observation and visual check; performing as series of test loading over two adjacent beams to obtain technical data about timber mechanical parameters;
3. Careful visual check and replacement of damaged and rotten beams with the same cross section and material (or similar);
4. Impregnation and protection of existing (and eventually new) beams with proper primers, impregnants and varnishes;
5. Execution of new a continuous wooden plank over the beams and its protection with impregnants and varnishes;
6. Application of waterproofing folio and special shear connectors, to ensure composite behaviour of timber beams and floor concrete slab;
7. Mounting of reinforcement over new floor timber deck, and connection rebars.
8. Concreting and slab compaction

2.1. Shear connectors for TCC slab

Fig. 4 presents typical detail for execution of TCC slab and Fig.5 – connection details between TCC floor structure and bearing masonry walls.

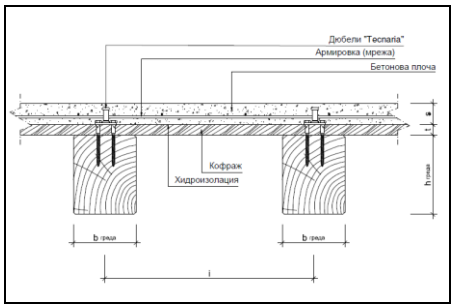


Fig. 4. Typical detail for TCC floor structure

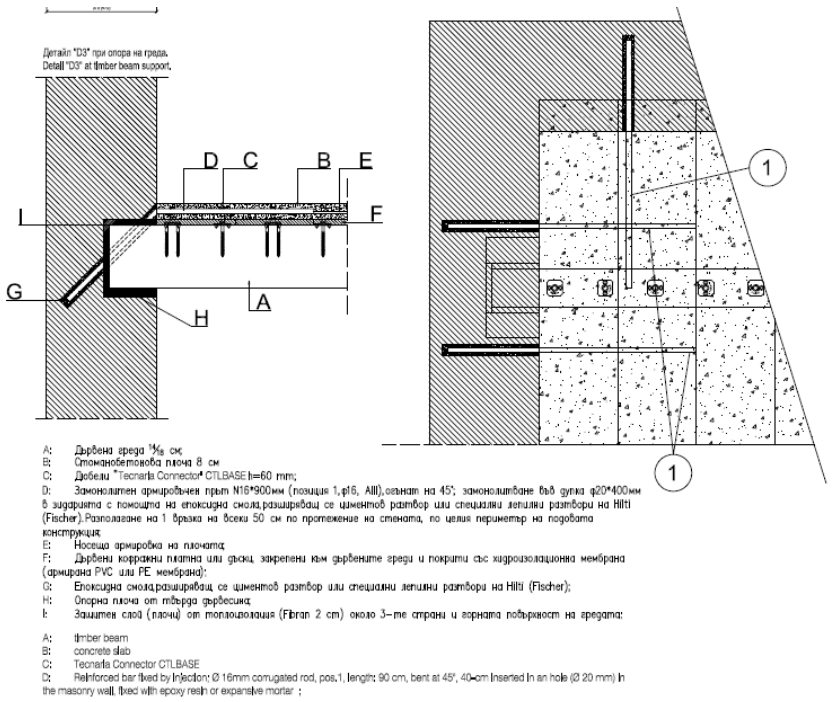


Fig. 5. Typical details for TCC floor structure connection to bearing masonry walls

2.2. Design calculation of TCC floor structure at elev.+3.52

Design calculation scheme is shown on Fig.6. This model is an extension and further development of basic Eurocode 5 design model for built-up beams (mechanically jointed beams) provided in Annex B of Eurocode 5. The respective formulas and equations are given in Fig. 7.

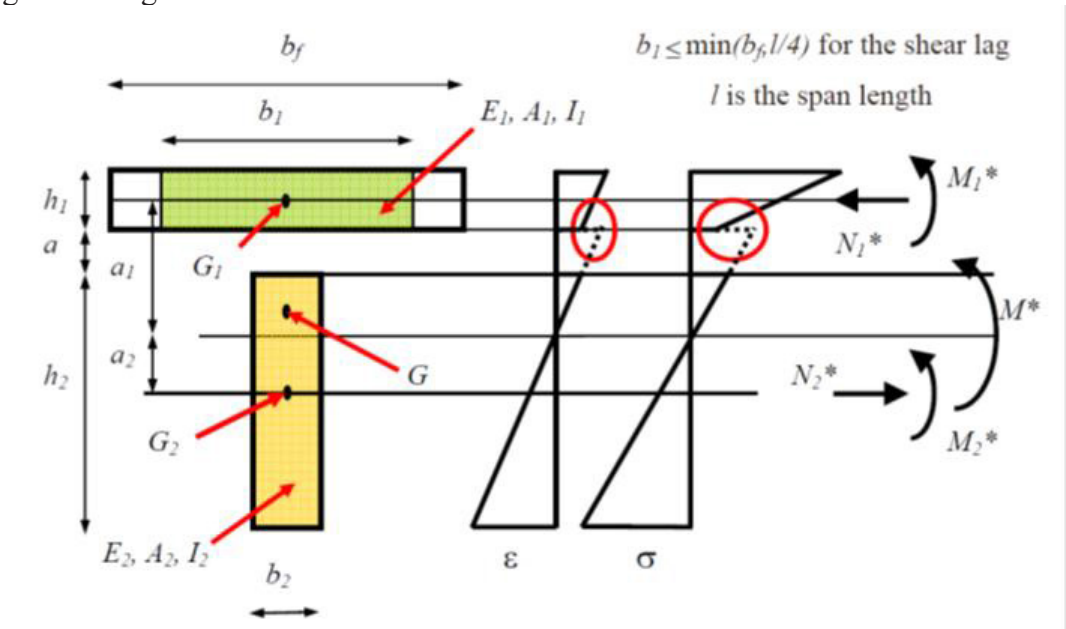


Fig. 6. Design model for TCC floor structure under vertical load

$$(EI)_{ef} = E_1 I_1 + E_2 I_2 + \gamma_1 E_1 A_1 a_1^2 + \gamma_2 E_2 A_2 a_2^2 \qquad \text{Eq. 7-3}$$

$$\gamma_1 = \frac{1}{1 + \frac{\pi^2 E_1 A_1 s_{ef}}{Kl^2}} \qquad \text{Eq. 7-4}$$

$$\gamma_2 = 1 \qquad A_i = b_i h_i \text{ with } i = 1,2 \qquad \text{Eq. 7-5} \quad \text{Eq. 7-6}$$

$$I_i = \frac{b_i h_i^3}{12} \text{ with } i = 1,2 \qquad a_1 = \frac{\gamma_2 E_2 A_2 H}{\gamma_1 E_1 A_1 + \gamma_2 E_2 A_2} \qquad \text{Eq. 7-7} \quad \text{Eq. 7-8}$$

$$a_2 = \frac{\gamma_1 E_1 A_1 H}{\gamma_1 E_1 A_1 + \gamma_2 E_2 A_2} \qquad H = h_1 / 2 + a + h_2 / 2 \qquad \text{Eq. 7-9} \quad \text{Eq. 7-10}$$

$$s_{ef} = 0.75 s_{min} + 0.25 s_{max} \qquad u = \frac{5 F_d^* l^4}{384 (EI)_{ef}} \qquad \text{Eq. 7-11} \quad \text{Eq. 7-12}$$

$$\sigma_{m,i}(x) = \frac{1}{2} \cdot \frac{E_i a_i M^*(x)}{(EI)_{ef}} \qquad \sigma_1(x) = -\frac{\gamma_1 E_1 a_1 M^*(x)}{(EI)_{ef}} \qquad \text{Eq. 7-13} \quad \text{Eq. 7-14}$$

$$\sigma_2(x) = \frac{\gamma_2 E_2 a_2 M^*(x)}{(EI)_{ef}} \qquad N_i^*(x) = \sigma_i(x) \cdot A_i \qquad \text{Eq. 7-15} \quad \text{Eq. 7-16}$$

$$M_i^*(x) = \sigma_{m,i}(x) \cdot Z_i \qquad V_2^*(x) = V^*(x) \qquad \text{Eq. 7-17} \quad \text{Eq. 7-18}$$

$$F^*(x) = \frac{\gamma_1 E_1 A_1 a_1 s(x)}{(EI)_{ef}} \cdot V^*(x) \qquad \text{Eq. 7-19}$$

Fig. 7. Design model for TCC floor structure – formulas and equations

Design calculations and checks for ULS, SLS and vibration safety for storey floor structure at elev.+3.52 are provided in the next

pages.

Input Data		go=	5.22	**Span:**	Ls=	4100	mm		
Geometry data		qo=	3.33	Materials	C16				
Timber									
b2=	130	mm			E2,mean=	13400	N/mm2	γ_m=	1.3
h2=	180	mm			f_{mk}=	50	N/mm2	f_{tok}=	40
A2=	23400	mm2		Creep factor	ψt=kdef=	0.6		$f_{c,90,k}$=	9.7
I2=	63180000	mm4			kmod=	0.6	permanent		
Plank						0.8	mediumterm		
a=	0	mm							
Concrete					C25/30				
b1=	900	mm			E1,o=	30000	N/mm2	γ_m=	1.5
h1=	60	mm			fck=	20	N/mm2		
A1=	54000	mm2		Creep factor	ψc=	2.5			
I1=	16200000	mm4							
smin=	300	smax=	300	**Dowels**	F dw,k=	20900	N	γ_m=	1.3
Spacing				M10x180	Kser=	17200	N/mm		
s=	300	mm			Kultr=	11467	N/mm		
n=	1	pcs/row		Creep factor	ψc=	2.45			

Short term behavior									
	ULS				E2=	13400	N/mm2		
H=h1/2+a+h2/2=		120	mm		E1=	30000	N/mm2		
$\gamma_{1\text{-}2}$=	0.039				K1-2=	11466.67	N/mm2		
a2=	20.1	mm		(EJ)min=(EJ)c+(EJ)t=		1.33E+12			
a1=	99.9	mm	a1+a2=	120.0	=H				
	E1*I1+g1-2*E1*A1*a1^2=			1.11623E+12	N.mm2				
	E2*I2+E2*A2*a2^2=			9.73293E+11	N.mm2				
			(EI)ef=	2.08952E+12	N.mm2				
gn=	4.70	KN/m	wn=1,00*gn+1,00*qn=		7.70	KN/m			
qn=	3.00	KN/m	w=1,35*gn+1,5*qn=		10.84	KN/m			
q_{tot}=w=		10.84	KN/m =	10.84	N/mm				
Mmax= q*l^2/8=		22772927	N.mm	22.773	KN.m				
Vmax=q*l/2=		22217.49	N						
			Kcomb= (EJeff-EJmin)/(EJmax-EJmin)=				0.189		
Concrete stresses					**Timber stresses**				
sc=	-1.27	MPa	(натиск)	st=	2.935	MPa	(опън)		
smc=	9.81	MPa		sm=	13.14	MPa	fto=		
				fm=	30.8	N/mm2	24.62		
sc1=	-11.08	MPa	(натиск)	st1=	-10.21	MPa	(натиск)		
sc2=	8.54	MPa	(опън)	st2=	16.08	MPa	(опън)		
				sm/fm+st/fto=	0.55	<1;			
Fdwl=	20128	N		dmax=	13.5	mm			
				L/dmax=	303				

Longterm behavior								
	ULS				E2=	8375	N/mm2	
H=h1/2+a+h2/2=		120	mm		E1=	8571	N/mm2	
γ_{t-c}=	0.058				K1-2=	4986	N/mm2	
a2=	14.5	mm						
a1=	105.5	mm	a1+a2=	120.0	=H			
	E1*I1+g1-2*E1*A1*a1^2=			4.37896E+11	N.mm2			
	E2*I2+E2*A2*a2^2=			5.70336E+11	N.mm2			
			(EI)ef=	1.00823E+12	N.mm2			
w=gn+0,3*qn=		5.60	KN/m =	5.60	N/mm			
Mmax= q*l^2/8=		11760906	N.mm					
Vmax=q*l/2=		11474.06	N					
Concrete stresses					**Timber stresses**			
sc=	-0.61	MPa	(натиск)	St=	1.417	MPa	(опън)	
smc=	3	MPa		Smt=	8.79	MPa	Fto=	
				Fm=	23.1	N/mm2	18.46	
sc1=	-3.61	MPa	(натиск)	st1=	-7.373	MPa	(натиск)	
sc2=	2.39	MPa	(опън)	st2=	10.207	MPa	(опън)	
				Sm/Fm+St/Fto=	0.46	<1;	OK	
Fdwl=	9673	N		dmax=	20.4	mm		
				L/dmax=	201			

L=	4.1	m						
m=	0.5706	t/m'						
EJ=	2089.525	KN/m2						
$f_{1,v}= \pi/2*(EJ/mL^4)^{0.5}$=			5.655	Hz >5,00				

Concrete slab design						
Nc= sc*Ac=		-68580	N		-68.58	KN
Mc=scm*Jc*2/hc=		5297400	N.mm		5.2974	KN.m
				6	N	12
Приета армировка			As=		678.24	\
x=	17.4	mm	fy=		500	MPa
a=	15	mm	Makros <CTRL+z>			
Nc+As*fy-bc*x*fc=			0	?=0		
Msd=Nc*(Mc/Nc+hc/2-a)=			6326100	N.mm		
Mrd=x*bc*fc*(hc-a-x/2)=			7581697.903	N.mm		
Msd/Mrd=	0.8	<1	OK!			

Check for longitudinal shear					
(equal strenght)					N
deltaX= s=	300	mm		deltaFd=Fdwk=	16077
ved=deltaFd/(hf*deltaX)=			0.893	N/mm2<fcd=	13.3
Theta=	45				
Asf/sf= ved*hc/ctg(Theta)/fyd=			0.107	sf=	1000
					mm
Asf=	107.2	mm2/m'		5N8/m'	

Conclusion: Design checks for bearing capacity, serviceability and vibration criteria are fulfilled.

3. Pictures and graphical presentation

3.1. Before reconstruction project



Fig. 8. Old building general view before reconstruction and upgrade project

On the façade of the building there is a plate stating that during years 1895-1897 great Bulgarian poet Peyo Yavorov served as telegrapher student in this building.

3.1. Structural retrofit at elev.+3.52



Fig. 9. Preparing of timber beams for check and test loading



Fig. 10. New wooden planking over beams



Fig. 11. Installation of waterproofing folio, shear connectors and reinforcement



Fig. 12. Concreting



Fig. 13. Preparation of TCC floor under roof structure

3.1. After implementation of structural retrofit and upgrade



Fig. 14. General view from interior



Fig. 15. General outside view after fulfilment of reconstruction activities

4. Conclusion

Old timber floors often need strengthening and stiffening as they were designed to bear moderate loads and almost always suffer from excessive deflections and vibrations with respect to current requirements. It was found possible to lay a thin concrete slab, adequately reinforced and connected, over the existing structure, to strengthen and stiffen existing timber floors to the required extent and/or to allow lower beams to be used as bearing elements of new floors.

In addition, this design methodology and construction technique make possible increasing of seismic stability of the building, fire and sound resistance of the floor structure and very effectively brings new floor structure performance to match all current criteria for serviceability and bearing capacity.

In the same way the main elements of the existing floor structure – beams- are still kept in service for the next design lifetime, thus maintaining an important architectural aspects for preservation of old buildings and their bearing timber floor structures.

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MONITORING OF STRUCTURE DOUBLE CASEMENT WINDOW

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Abstract: *The paper deals with structure of double casement window in the building from first decade of 20 century. There is description of monitoring of existing structure in–situ. First there is application of measuring instruments with record variables (temperature and relative humidity) of air and surface at regular intervals. Measuring instruments (dataloggers) were placed on the exterior, in between the two window panes and in the interior of double casement window structure and were connected to probes that measure surface temperature. The second there is description and analysis of measured data.*

Key words*: monitoring, double casement window, relative humidity, temperature*

1. Introduction

Measurement of actual valuables (in this case temperature and relative humidity) is important part of survey historical structures. It is used to describe real behaviour of the structure in specify surroundings of structure included for example solar radiation, wind, rain etc. In this paper there is described measurement of existing structure of double casement window made in the early 20th century. Double casement window is window with two casements, outer casement and interior casement, both with simple glazing.

Measured variables were temperature and relative humidity. Nowadays, the topic of renovation or replacement of double casement windows in the Czech Republic is quite common from renovation point of view. Obtain of real variables on the existing structure is important component for design of window structure. Design of new window structure includes this types of structures: refurbishment of the existing double casement window, using double insulating glazing on the interior window casement, using double insulating glazing on the exterior window casement, replacement existing window structure with a new structure (simple window).

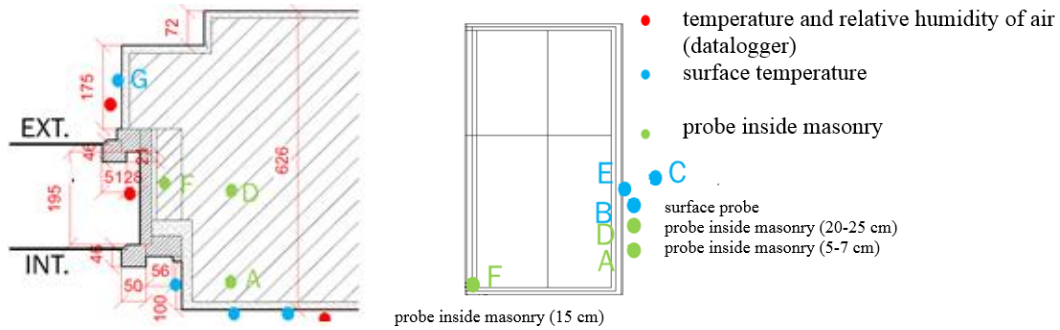
Shadow parameter is condensation of water vapour and mould occurrence on the interior surface. Critical internal surface humidity for the window pane is ϕ=100 % (risk of condensation) and for other structures ϕ= 80% (risk of mould). With regard to the window's surroundings, in rooms meant for habitation (20˚C, 50%) vapour condensation takes place at 9.3 ˚C and a high risk of surface mould at 12.6 ˚C. These temperatures are the evaluation criteria for the tested interior surface area with regard to outcomes.

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2. Measurement on the existing structure

The jamb framework was fitted with dataloggers and temperature probes. Dataloggers that measure and record temperature and relative humidity of air at regular one-hour intervals were placed on the exterior, in between the two window panes and in the interior. Precisely there were these types of dataloggers: Comet S0141 (1 instrument), Comet S3631 (3 instruments) and Comet thermo-hygro-barometr D4130 (1 instrument). The first two dataloggers regularly measured and recorded temperature and relative humidity. The third datalogger served to measure of instant data. The dataloggers were connected to probes (Comet Pt 1000) that measure surface temperature and to probes that measure the temperature of air (Pt1000).

The dataloggers were connected to probes that measure surface temperature and probes measuring ambient temperature. These probes were placed on the surface of interior walls (probes B and E), the surface of the exterior façade (probe G) and in the pre-drilled holes in the wall (probes A and D). Interior surface probes were located on the surface of the perimeter wall, approximately 10 cm from the face of the window wall and also in close proximity to the window jamb (i.e., 5 cm) perpendicular to the interior pane of the window. Figures 1 depicts the placement of the probes and dataloggers. Figure 2 depicts how the dataloggers were fitted.



a) b)

Fig. 1. a) cross-section of the window jamb and placement of probes and data loggers; frontal view b) frontal view on the measuring structure



Fig. 2. Photo of frontal view on the measuring structure

3. Measuring conditions

The measurement was taken in the term January 2015 – January 2016 (13 month). Data were regularly recorded to the instrument memory every one hour. Regularly (every 2 month) were data downloaded to the computer. Measured structure is situated on the west façade of building in the 2nd floor. In the term beginning of the April to half of September is measurement ruled by solar radiation in the daytime. Window structure was measured by common office surroundings and during the measurement it was used to room ventilation.

4. Measured data

The result are measured data, processed in software Microsoft Office 2013 - Excel. Directly measured variables are temperature [˚C] and relative humidity [%]. Derived variables are dew point [˚C], partial water vapour pressure [Pa] and humidity by mass [g/kg d.a.]. Table 1 shows example of measured variables during term (summer and winter).

Table 1 Excel table of measured data

probe	date	time	temperatur	relative humidity	dew point	partial water vapour pressure	humidity by mass
			[˚C]	[%]	[˚C]	[Pa]	[g/kg d.a.]
winter term							
exterier (air)	3.12.2015	02:00:00	10,3	64,3	3,9	1253	5,1
in between (air)	3.12.2015	02:00:00	13,8	47,9	3,0	1579	4,7
interier (air)	3.12.2015	02:00:00	20,3	35,8	4,7	2383	5,4
probe A	3.12.2015	02:00:00	19,3	-	-	-	-
probe B	3.12.2015	02:00:00	19,7	-	-	-	-
probe C	3.12.2015	02:00:00	19,6	-	-	-	-
probe D	3.12.2015	02:00:00	16,1	-	-	-	-
probe E	3.12.2015	02:00:00	19,3	-	-	-	-
probe F	3.12.2015	02:00:00	16,7	-	-	-	-
probe G	3.12.2015	02:00:00	11,1	-	-	-	-
summer term							
exterier (air)	11.8.2015	02:00:00	26,0	40,4	11,5	3363	8,6
in between (air)	11.8.2015	02:00:00	28,8	32,7	10,8	3961	8,2
interier (air)	11.8.2015	02:00:00	30,6	34,5	13,2	4393	9,6
probe A	11.8.2015	02:00:00	19,3	-	-	-	-
probe B	11.8.2015	02:00:00	30,9	-	-	-	-
probe C	11.8.2015	02:00:00	30,7	-	-	-	-
probe D	11.8.2015	02:00:00	31,5	-	-	-	-
probe E	11.8.2015	02:00:00	31,0	-	-	-	-
probe F	11.8.2015	02:00:00	32,0	-	-	-	-
probe G	11.8.2015	02:00:00	11,1	-	-	-	-

Measured data were processed to graph. Figure 3 to figure 6 show analyzed data. There are values of variables (temperature, relative humidity and humidity by mass) on the vertical axis and time (date) on the horizontal axis.

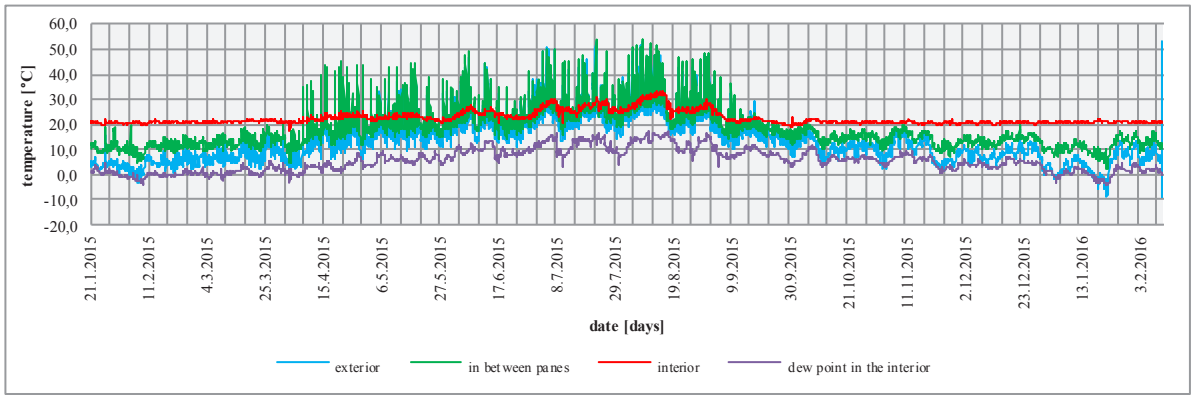


Fig. 3. Evolution of variables (temperature) during observation period (January 2015 – February 2016)

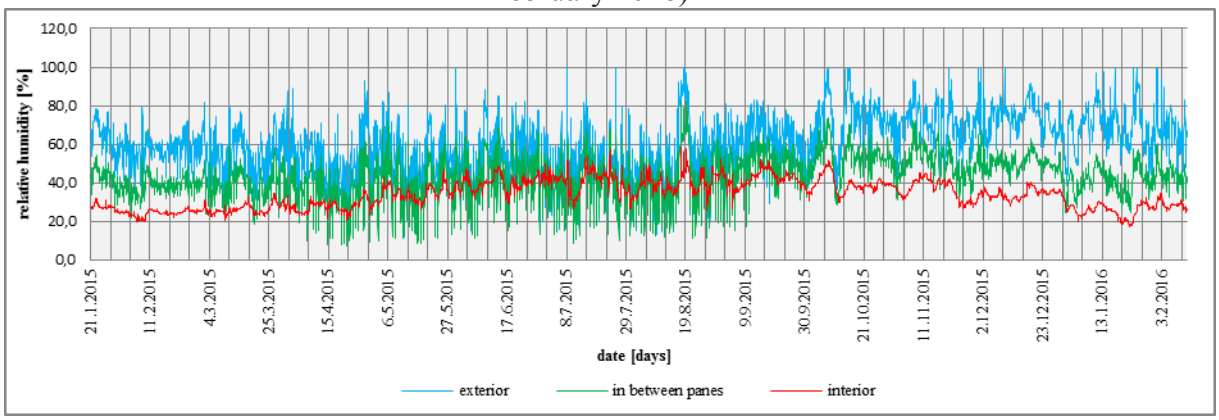


Fig. 4. Evolution of variables (relative humidity) during observation period (January 2015-February 2016)

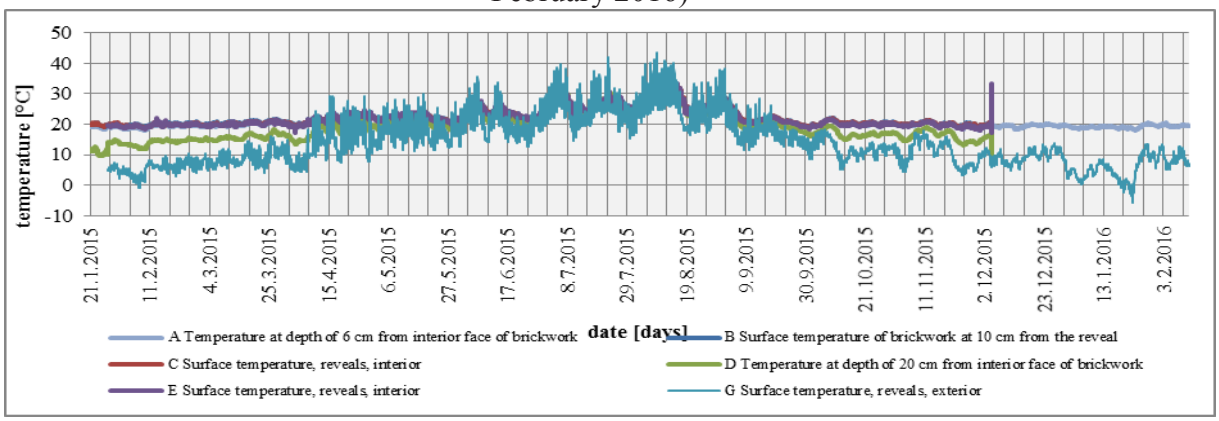


Fig. 5. Evolution of variables (surface temperature, temperature in the structure) during observation period (January 2015-February 2016)

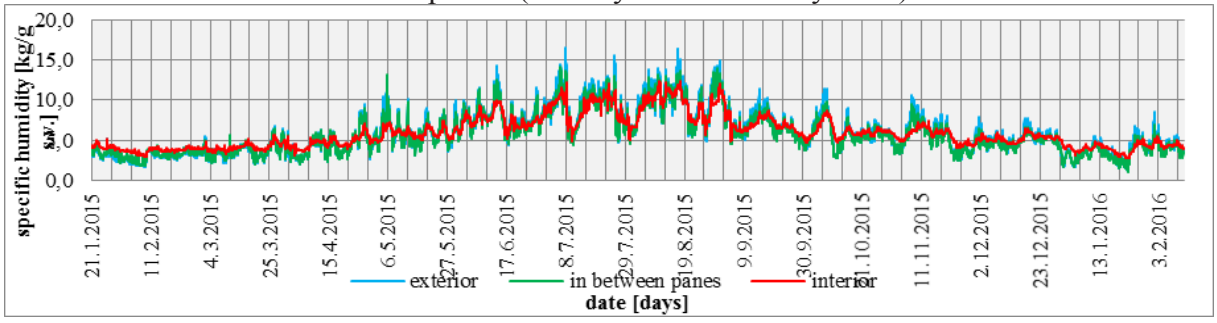


Fig. 6. Evolution of variables (humidity by mass) during observation period (January 2015-February 2016)

Figure 7 and figure 8 show short sections of measured term with more detailed in time by one hour interval. In figure 7 temperature of air in between has difference up 28 ˚C during 6 hours and 17 ˚C. Phase shift of peripheral (difference of dependence between exterior temperature and interior surface temperature) is approximately 5 hours.

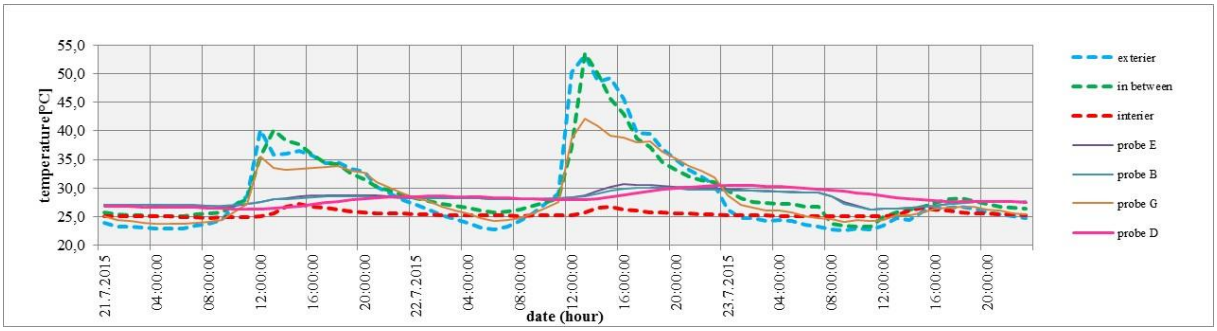


Fig. 7. Extreme surroundings measured in the summer term (21.7.-23.7.2015)

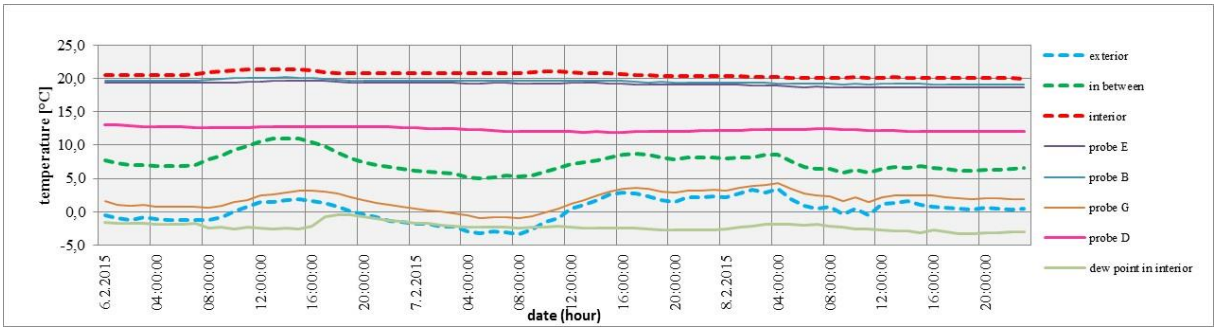


Fig. 8. Extreme surroundings measured in the winter term (6.2.-8.2. 2015)

Table 2 shows the minimum and maximum values.

Table 2. Minimal, maximal and average values measured for temperature and relative humidity at individual locations

	min [°C]	max [°C]	ave [°C]	min [%]	max [%]	ave [%]
Exterior (air)	-8,5	53,3	13,3	10,6	100	59,3
In-between window panes (air)	2,2	54,2	17,5	7,0	83,0	43,6
Interior (air)	17,2	33,6	22,8	17,2	58,4	33,9
Temperature at depth of 6 cm from interior face of brickwork (A)	17,5	32,7	21,6	-	-	-
Surface temperature of brickwork at 10 cm from the jamb (B)	18,3	32,9	22,4	-	-	-
Temperature at depth of 20 cm form interior face of brickwork (D)	9,6	32,5	19,6	-	-	-
Surface temperature, jambs, interior (E)	17,1	33,3	22,3	-	-	-
Surface temperature, exterior (G)	-0,9	43,5	15,7	-	-	-

Note B,C,D,E were measured in the period 21.1.2015-3.12.2015

5. Conclusion

In this paper is described measurement of double casement window structure. Measured values of variables were temperature and relative humidity. Actual valuables and behaviour of structure is important part of survey historical structures. Both the air temperature of each room (exterior, in between and interior) and the surface temperature were measured. Measured data are used for evaluation of real surroundings and surface temperature on the existing windows construction and for comparison with 2D model calculation. Measured and analyses data are useful for design of new window with aim of decrease of critical areas on the structure from occurrence of water vapour condensation and mould.

The measurement and model calculation led to the following conclusions:

- The existing window structure proved adequate in terms of critical surface temperature (condensation and moulding) during the monitoring period. The obtained data served for a maximally realistic model, i.e., to determine what thermo-physical processes take place in the window reveals.
- The graphs show the visible influence of sun exposure (beginning of the April to half of September).
- In contrast, in the summer the structure was subject to extreme conditions. The highest summer temperature measured by a data logger exposed to sun was higher than 50°C and temperatures in between the window panes were also higher than 50°C.
- Averaged values of humidity by mass are similarly (5,9 g/kg d.a. - exterior; 5,7 g/kg d.a. – in between; 6,0 g/kg d.a. - interior).
- Over the monitored period, winter temperatures did not drop to the extreme calculation conditions for Prague (-13°C). The minimal exterior temperature measured outside the frame on January 23, 2016, at 2 AM was -8.5°C with 57,3% relative humidity -> this makes the difference between the calculation and real design conditions almost 5°C.
- Monitoring of boundary conditions took place in the city centre, in a developed area. Therefore, the window structure was not exposed to extreme conditions (e.g., strong gusts of wind on an open plain).
- During the measurement period, the window served regular office users; the office represented an interior with low relative air humidity.
- Full-width wall heat transmission coefficient was calculated at 1.01 W/m^2K.
- Air temperature in between the window panes differs by up to 27.9°C over a period of 6 hours and by 16.5°C during 1 hour.
- It is possible to determine the phase shift of the wall, the difference between changes in the exterior temperature and the interior surface temperature, i.e., approximately 5 hours.
- Measured data show great difference thermally-physical processes in the „area under the casement".

Acknowledgement

This work was supported by the Grant Agency of the Czech Technical University in Prague, grant No. SGS 17/160/OHK1/2T/31.

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EXTENDING THE LIFE-TIME OF EARLY 20-TH CENTURY STEEL PORTAL FRAME BRIDGES

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Abstract: *In the city of Ghent (Belgium) 3 early 20-th century riveted portal frame bridges are in use for local traffic of cars, buses and trams. All of them are situated at historically important locations. During assessment of the consequences of the heavily corroded condition of the bridges, some particular characteristics were found. The member cross-sections show rather asymmetrical characteristics. This seems related to the reduction of the section's area due to weakening by rivet holes in tension areas, whereas this reduction does not apply to compressed areas. In addition, the corroded state of the cast steel hinges may prevent free rotation of the supports. However, the redistribution of bending moments, due to rotational clamping is shown to be relatively low. Thirdly, tests on material samples taken from the structure, have demonstrated that the yield stress and failure stress of the early 20-th century steel is close to modern mild steel and there is some degree of weldability. Several alternatives for low-cost refurbishment are presently being considered. The most effective way to deal with the insufficient load-carrying capacity of the outside frames is to reduce the dead weight on top of them, thus allowing to resist the crowd load.*

Key words: *early-age steel, portal frame bridge, load-carrying capacity, riveted connections, Template, formats, instructions, length, Times New Roman, 12 pt, Italic, Justified*

1. Introduction – historic locations

The city of Ghent was founded around the 9th-century nearby 2 abbeys and at the confluent of rivers Lys and Scheldt [1]. Some relics of Roman settlement were equally found, but the development of larger cities and communities as we know today was originated by trade and workmanship. In the 14-th century 64000 people lived in Ghent, a number only surpassed by Paris. However, in the early days of the 20th century, the city was fundamentally reshaped, in view of the 1913 world exposition [2].

In view of this history, the confluent of rivers is an important location. To the East was the St-Bavon abbey, while on the West bank of the river the community was developing. The rivers were crossed by several bridges, which in 1907 were merged by the construction of the excessively wide St George bridge. Although river Scheldt modified its course, this location is the original confluent of the rivers and from that point of view of

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historic importance. In addition, the Portus Ganda, a pleasance harbour has been developed nearby and twice a year fireworks are organised above the water.

During the first half of the 11th-century a 580 m long canal was built, connecting both rivers Scheldt and Lys. This materialised the southern limit of the city. Bridges allowed crossing the canal and in 1912, at the exact location where the canal joins river Lys, the Cauldron bridge was built.

Finally, around 1750 the Coupure canal was built, allowing vessels to avoid the city centre and travelling from south to the coast. The most important crossing of Coupure is the road towards Bruges. Today the entire canal environment is protected as a monument.

As mentioned, in 1913, on the eve of World War 1, the city organized a world exposition. During the preceding period, many historic objects and buildings were refurbished according to the methods used in that time. In addition, large areas with buildings were rebuilt and new streets were created. Especially movable bridges were replaced by fixed crossings and steel, which was becoming a popular building material, was used to build these bridges. At present, only 3 original bridges from that period are still in use.

Today these waterways are no longer in use for shipping. However, they are an essential part of the city's character and life.

2. Geometry and composition

Since the 3 bridges were built across waterways with intense fluvial traffic at that time, the vertical clearance was important for the actual design. This has conducted to the idea of the portal frame, as shown in Fig. 1. The latter allows transferring of bending moments towards the portal nodes, where the vertical clearance is less critical. Consequently, the beam height at the span centre can be reduced, which is exactly what is needed for obtaining maximum vertical clearance of the subjacent waterway

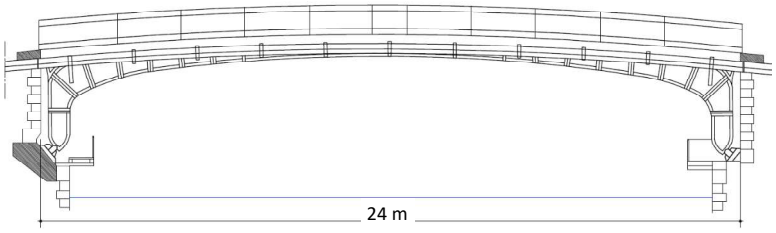


Fig. 1. Portal frame bridge



Fig. 2. Parallel frames

The 3 bridges have parallel ribs, showing similarity to recent bridges and coverings. The parallel frames clearly show variable beam depth, in accordance with the expected bending moment distribution. In case of the St-George bridge, the depth is varying from 1.071 m to 0.356 m at mid-span, which probably cannot be reduced by any modern bridge design. The parallel frames are shown from below in Fig. 2.

In the aesthetical perspective, the intrados shape of the frames is the visible characteristic. At the time of construction, shapes and forms were still inspired by vaults and arches. To assemble a numerical model, the knowledge of the intrados shape was also essential. After measuring 17 points of the central part, a polynomial regression resulted in the best fit of a 6th degree curve.

$$y = 2\ 10^{-7}\ x^6 - 3\ 10^{-15}x^5 - 3\ 10^{-5}x^4 + 4\ 10^{-12}x^3 - 0.0031\ x^2 - 10^{-11}x + 3.658 \quad (1)$$

expressed in m. The R^2 value equals 0.9984 demonstrating that the approximation is adequate. It is surprising that this curve, applying to the St George bridge, does not correspond to any of the well-known curves from arches or vaults. Certainly, the existence of several reinforcing plates near the span centre may influence the result. However, for the Cauldron bridge, having less reinforcing plates, the best fit also resulted in the 6th degree polynome. The third bridge, named Barge bridge across Coupure canal, shown in Fig. 3 was not analysed in detail, since from April 1st 2017 motorized traffic has been banned on this road.



Fig. 3. Barge bridge across Coupure

A particular detail concerns the hinged supports. It is widely known that the fabrication of hinges is more complicated than clamped connections, the latter presumably showing some lacking of complete blocking of angular rotation. Hinged supports require free rotation, as well as the introduction of locally concentrated reactions with variable direction. These concentrated loads bust be spread across the nearby structural sections, often requiring particular stiffening and variable plate thickness.

In the case of the St George and Cauldron bridges, the hinges were fabricated from cast steel, the flange thickness being locally increased, as can be seen in Fig. 4. The cam itself is eccentrically positioned, thus allowing more horizontal thrust force and its spreading across a large portion of the webs. Reproductions of the old drawings of the St-George bridge mention the direction of the reaction varies by some 10.8°, the fixed part of reactions showing a slope of 45°.



Fig. 4. Cast steel hinges

As shown in Figure 4, the cam radius is rather small (61 mm), thus introducing large contact stresses, since the length is limited to 500 mm. A central stiffener has been added to the cast steel part. In many historic structures, various types of hinged joints have been included and till 30 years ago, high-strength steel roller supports were widespread. Modern bridge design has practically left out this type of support and has reduced the options to pot and spherical bearings, which are more adapted to simple beam bridges.

Both the St-George and Cauldron bridges have recently been inspected. The general aspect of both bridges is worrying and corrosion is seen in every detail. However, it is more important to find out whether the steel sections show considerable reduction and to quantify this. Although modern riverside walkways have been installed below all bridges, the corrosion protection has not been renewed since 30 years. For both bridges especially the outside frames are corroded, either the angle profiles of the lower flanges or the top plate of the upper flange. In a single case of a vertical frame post, the complete upper flange is corroded.

These observations have been supplemented at the most critical locations by grinding to blank metal and by measuring the exact length of the main corrosion areas. This enabled to derive exact data of the remaining profile and plate sections and to assemble characteristics of an alternative damaged structure. Superficial deterioration has not been considered in this alternative.

The hinged supports were inspected more in particular. Already Fig. 4 shows a rather bad condition. However, after manual brushing, hammering and subsequent local grinding, the corrosion of these parts appeared equally superficial and limited to less than 1 mm. Most probably the hinge is not sufficiently functioning and will require serious refurbishment.

3. Particular characteristics

The analysis of the load-carrying capacity of the deteriorated condition of both the Cauldron and St-George bridge, revealed some particular characteristics of these structures.

3.1. Asymmetrical cross-sections

The cross-sectional characteristics of the frames are varying, according to the portal beam depth. In particular, near the span centre, the total depth is reduced at maximum, thus requiring reinforcement of the flanges by additional plates. The cross section characteristics are shown in Fig. 5a for St George's bridges and Fig. 5b for the smaller

Cauldron Bridge. Obviously, at this location the upper flanges are compressed, whereas the lower flange is in tension. As the normal force has little influence, an important difference of areas is noticed. For the left part of Fig 5 the upper flange is 86 % of the lower one, whereas for Fig. 5b right the difference equals 55%. There must be a fundamental reason for the smaller compressed flanges.

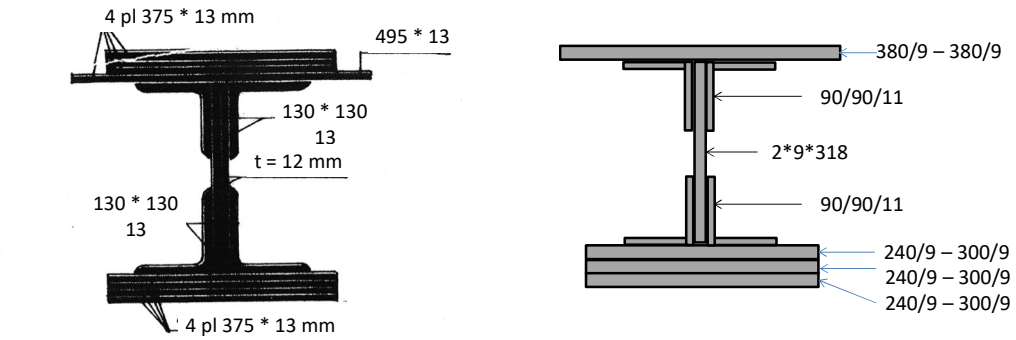


Fig. 5. a) Cross section span centre St George's Bridge; b) Id Cauldron bridge

Clearly, adding plates to flanges, connected to the webs by angles angle profiles and rivets, does not increase linearly the resisting moment, since the rivet holes are weakening the area. The latter must be subtracted from the initial properties. In the case of Fig. 5a the ratio of reduction equals up to 86% and if the corrosion is taken into account, even as much as 83%. Hence, the reduction of resisting cross-sectional properties by holes may be of paramount importance. In [3] it is demonstrated in various ways that no section area reduction should be considered in compressed parts, whereas this reduction is needed in tension parts. Hence, in pure bending, compressed flanges may well show smaller areas than tensile flanges.

3.2. Effectiveness of rotation in hinges

The free rotation of the hinged supports of the portal frames seems crucial to the bending moment distribution. Partial or full clamping introduces bending at the frame springs and creates larger stiffness of the vertical posts. Hence, the bending in the spans may reduce. However, even taking into account the corroded cross-sectional properties, the span is not the critical section. The increase in bending moment is mostly found in the portal nodes, which are the critical locations.

The maximum rotation in SLS of the hinges due to dead load and the highest of movable loads (trams) equals 0.0581°. This represents a total displacement at the road level of 3.5 mm. However, the part of variable loads is 48.1 % and the displacement that should occur freely 1.78 mm. This is hardly noticeable and may very well occur partially. The largest stress increase due to lack of free rotation and cause by variable loads is limited to 8.5%. Again, there is no actual proof that this increase actually exists, since the hinges may appear to be corroded, these slight rotations are invisible to the eye.

3.3. Material characteristics

At the period of construction (1907 for St-George bridge and 1912 for Cauldron bridge), more precisely from 1901, the Siemens-Martin and Thomas fabrication processes of steel had been developed and all data point towards the material being early age steel. Within the 6th European Framework [4], an impressive amount of samples has been used to characterise steel produced at various periods. In addition, the Swedish Standard BVS 583.11 and the work done at Lulea univ [5] are highly relevant. A very clear document,

ready for use, is the German Richtlinie 805 [6]. The latter was applied successfully in the project for refurbishing the Rendsburg viaduct [7]. This method would imply that a material γ_M factor would be applied of 1.15 and an additional factor γ_B to account for superficial corrosion. This results in the use of f_d = 235 / (1.05*1.15) = 195 MPa. According to [7] a characteristic value of 454 MPa has been adopted for the tensile strength of rivets.

The owner preferred experimental verification of the material properties. Hence, in July 2017, 3 samples were taken from the webs of crossbeams, which do not carry directly any of the traffic loads. The samples of 300* 75 mm and 10 mm thickness were further resized to 20 mm width. The cutting by grinding can be seen in Fig. 6a and a collected sample in Fig. 6b.

The results of the 3 samples gave values of 254, 277 and 238 for $f_{y\ 0.2\%}$ and 338, 382 and 347 for f_u. In view of the limited number, and not knowing the dispersion, application of annex D of EN 1990 is virtually excluded.



Fig. 6. a) extracting of sample b) Collected sample

The material may be close to steel S 235 but the dispersion of the characteristics is rather large. This may be expected from early-age steel, since the constant character of quality was poorly monitored and large variations may have occurred. In conclusion, the value of f_y = 205 MPa was adopted for verification.

4. Critical loading conditions

Considering exclusively the St-George bridge, the undamaged and corroded structure has been verified by using extensive numerical simulation, supplemented by classical analysis of the riveted connections. The numerical model is shown in Fig. 7, which also shows some of the damage introduced, like a totally missing flange and reduced thickness of some plates.

A particular condition is the type of variable loads. Obviously, these old structures were never designed for the present loading schemes. Various references have been used for determining the live loads. Among these, Eurocodes EN 1991-2 should be mentioned as well as its Belgian National Annex [8], which is a very useful document. The latter allows verification of existing infrastructure through the use of a realistic LM 5. This LM 5 may replace LM 1 for existing structures, provided the bridge carries local traffic only, which applies to bridges inside historic cities. LM 5 corresponds to one of the frequent lorries of FLM 2. It requires the use of adapted factors such as a combination and an impact factor. If such a model is used, it was felt that it should be supplemented by other loading schemes, from actual loads of trams and buses.

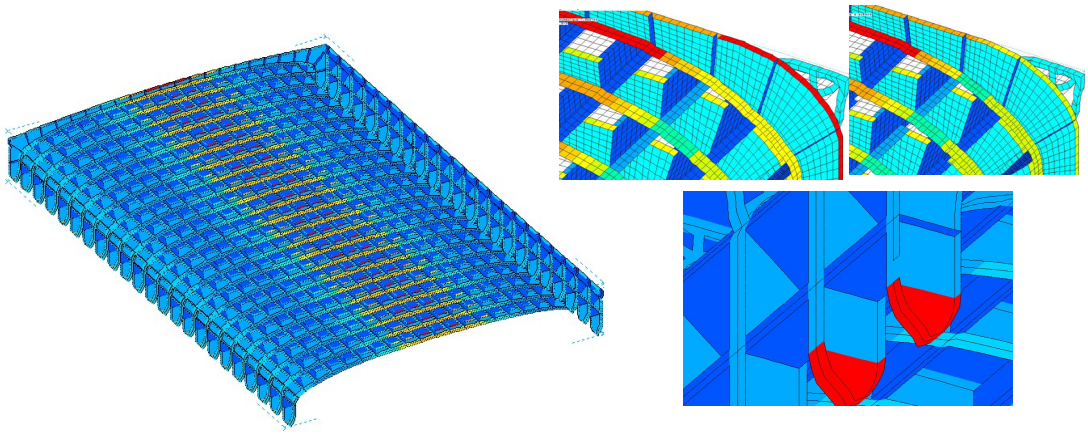


Fig. 7. Numerical model including corrosion damage

An appropriate modern tram loading has also been considered. As these trams have a metric track width, the distance between wheels equals 1 m. In addition, most bridges are frequently crossed by public transport buses and touristic coaches. A realistic loading scheme for buses obtained from a fabricator was also considered. This scheme includes high knife loads of 135 kN, this being derived from a comprehensive survey, made in 4 EU-countries [9] and stating that official axle loads are frequently exceeded in real traffic. However, this survey was unable to detect which vehicle axle is the heaviest. Consequently, knife loads of 135 kN were taken into account.

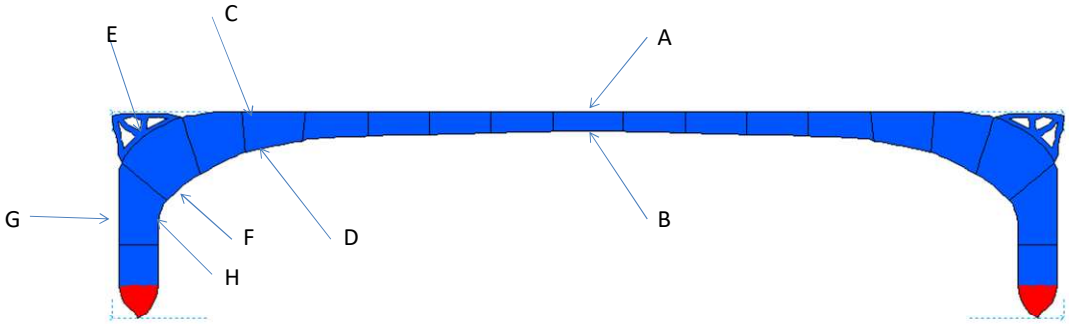


Fig. 8. Critical locations

Table 1: Unity check for undamaged and corroded structure.

Unity-check	Undamaged structure	Degraded Frame North	Degraded frame South
Span centre upper flange	0.311	0.221	0.206
Span centre lower flange	0.318	0.551	0.361
Frame node upper side	0.461	0.949	0.53
Frame node lower side	0.666	1.251	1.091
Post outside	0.411	0.719	0.349
Post inside	0.966	0.94	0.772

Table 1 summarizes the results for load carrying-capacity through the values of the unity check at the critical locations of the undamaged and degraded frames. Obviously, the undamaged structure satisfied the requirements and can resist the relevant loading of trams,

buses and lighter trucks, as well as the crowd loading. The degraded frames have insufficient load carrying capacity to resist the crowd load. The critical locations are shown in Fig. 8 and apply all to the outside frames and crowd loading. Finally location D does not satisfy the unity check.

5. Reburbishment alternatives

The results of calculations clearly showed that, in spite of heavy corrosion, the St George bridge still is capable of carrying realistic loads in normal safety conditions, except for the 1st and 2nd edge frames, both on the Northern and Southern side of the bridge. Two alternatives may be considered to restore the load-carrying capacity.

Apart from the possible repair of the outside frames, all other steel parts of the bridge should be grit blasted to obtain corrosion free material. One of the difficulties will be to limit blasting pressure, in this case to 0.17 MPa. If higher pressure is used, the work may be faster, but the risk of damaging further some of the weaker parts may be high. Adequate filler products may be used to repair serrated edges of plates and profiles. The latter also applies to the hinged supports, which will need to be free of dust and blasting material before greasing.

Local replacement of corroded plates or profiles becomes rather difficult, since the original length of the elements mostly covers the entire frame. In addition, if an outside angle profile must be replaced, the opposite will also be disconnected from a member web, since the rivets are connecting both. The difficulty is shown in fig. 9 for a single cross-section at the span centre. This has to be taken into account when a refurbishment process is developed.

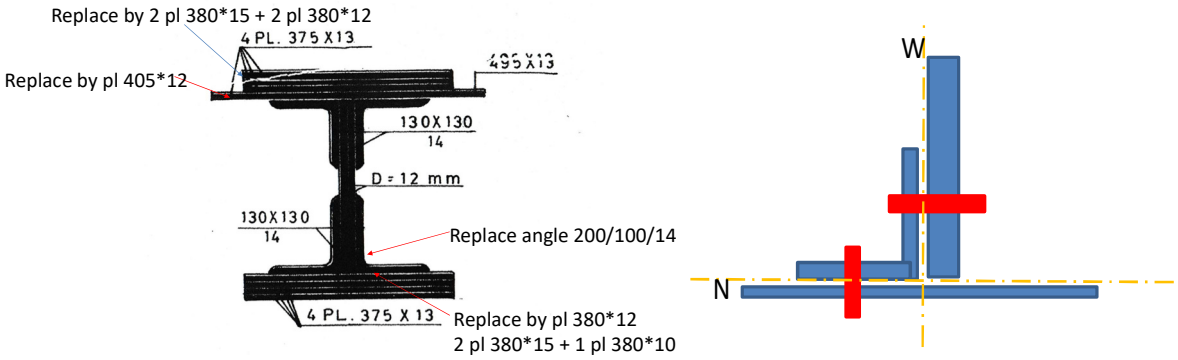


Fig. 9 : Replacing of individual plates and angles

Hence, the first alternative would be to replace corroded profiles either on site or at a workshop. If a frame is still connected to the crossbeams and the heavily corroded angles and flanges are removed for replacement and if the new elements are put in place, before connection of these reinforcements a heavily weakened condition will exist. Should there not be any temporary support, the steel stresses in this situation reach 105.6 MPa due to the dead weight alone. Compared to the stress in the damaged condition, there is an increase of stresses from 14.50 to 117.3 MPa tension at the lower flange at the span centre and from -39.95 MPa to -96.12 at the node. Evidently, these stress increases are unacceptable and the reinforcement on site is not a valid solution, unless temporary supports would be placed in the river bed. Unfortunately, the administrator of the river cannot accept such disturbance in fluvial traffic. Hence, the reinforcement would then require to remove the 4 outer frames by disconnecting them, and subsequently reinforce them at a workshop.

A second alternative, more daring, is to replace the Northern and Southern portal frames by an existing frame from inside the bridge, for instance numbers 4 or 5. These extracted frames would then be replaced by a new, welded portal, to be connected by bolts to the remaining structure. In doing this, the outside appearance of the bridge would remain identical and the structure can satisfy all requirements. However, the detailed design of such a transformation will probably reveal a series of connection problems. In addition, the removal and replacement of the frames introduced temporary stresses of the same magnitude as in the first alternative.

A final alternative would be to apply anti-corrosion treatment on site, without replacing any part of the structure. Instead, the walkway loads may be reduced. Indeed, at present the walkways consist of some 0.40 m of cement-mixed sand and concrete tiles. Most of this may be replaced by lightweight concrete of 11 kN/m³ of 0.32 m thickness and concrete tiles. This reduces the total load by 5.4 kN/m², which is already compensating the lack of load-carrying capacity. This alternative allows to limit procedures to the blasting and repainting as well as cosmetic finishing of jagged edges and injections of open edges and ribs, as well as an adjustment of the composition of the roadway. Obviously, this is far more attractive for the owner. The former applies to the case of the St-George bridge, whereas corrosion damage is much lighter for the Cauldron bridge. However, a similar solution will also apply for this structure.

6. Conclusion

Only 3 bridges built for the 1913 world exposition are original early age steel structures. Maintenance of these structures has been at low frequency, since it was repeated in average every 30 year. Consequently, each period extensive refurbishment was needed.

Assessment of the future use and load carrying capacity of the older bridges required the use of appropriate references, in this case a National Annex to EN 1991-2. This has been supplemented by real traffic data and appropriate load and dynamic factors.

The analysis and assessment of the load-carrying capacity of 2 of the oldest bridges, early age steel portal frames, was carried out, taking into account the damage due to corrosion, found during inspection. The results of the St-George bridge have been discussed and showed insufficient capacity if the corrosion damage is included in the model. However, this insufficiency applies only for 2 of the 23 portal frames, located below the walkways and is due to crowd loading, which actually occurs on this bridge during events. Three alternatives have been considered for increasing the load-carrying capacity of the damaged portal frames. Refurbishment on site by replacing heavily corroded members creates temporary reduction of the frame resistance up to unacceptable increase of steel stress. This cannot be compensated by the final increase in resistance. Hence, reinforcing the weakened frames, requires removing them from the site and carry out the refurbishment at a workshop. Internal shifting of portal frames has similar setbacks as the former alternative.

The final alternative of reinforcing of the structure is to proceed with corrosion protection and to reduce the dead load on the walkways. Calculations have shown the validity of this option, which is preferred by the owner. Refined analysis and the use of detailed numerical modelling, as well as realistic loading schemes, may thus contribute to thorough refurbishment and extended life of 2 historic early-age steel portal frame bridges.

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SOME GENERAL MISTAKES DURING THE DESIGN AND THE EXECUTION OF THE STRUCTURAL RETROFITTING OF HISTORICAL BUILDINGS

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***Abstract**: The process of retrofitting includes the assessment of the historical structures and the design and the execution of the remedial measures. The paper is dedicated to the review and the analysis of the commonly made mistakes in the process of the design and the construction of the retrofitting of historical buildings. The paper discusses the specific approaches in the case of historical buildings. Based on the presented discussion, some general recommendations are given.*

***Key words**: historical buildings, assessment, retrofitting, interventions*

1. Introduction

Intervention in heritage structures, involving either stabilization, repair or strengthening, (in particular, seismic retrofitting), should be subjected to a series of requirements or criteria oriented to ascertain the efficiency of the solution together with its compliance with recognized conservation principles. These principles are stated in international documents such as the Venice Charter of 1964 and, in a more specific way, in the ICOMOS / ISCARSAH Recommendations for the Analysis and Restoration of Architectural Heritage, (ICOMOS/ ISCARSAH, 2005), and the Annex on Heritage Structures of ISO/FDIS 13822, (ISO/TC96/SC2, 2010) [4].

Trying to satisfy these conditions will assist in conceiving and designing both efficient and respectful interventions consistent with conservation principles.

Very often the application of inappropriate repairs can lead to problems in the behaviour of the original structure. The faulty repairs may result also in inadequate performance of the structure and can lead to serious damages.

2. Challenges in retrofitting of historical buildings [5]

2.1. Safety versus historical preservation and upgrading of the seismic performance

The following considerations should be taken into account:

1) *Condition of the building* - past conditions should be compared and to be expected future conditions. Seismic actions are more problematic, because of the random nature of the earthquakes.

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2) *Cultural value of the building* - if a structure is of great cultural value it might even be considered not to be used at all but to be left as a monument of the historical time it represents. If there is need of any structural interventions, they are to be chosen with great care so that any impact is minimized.

3) *Current use of the building* - sometimes the significance of the function of the building (that might be its original function) prevails over its cultural value. Safety should be considered primary to historical preservation, because in the case of an extreme action (e.g. earthquake) the social cost could be much higher than the cost of affecting the cultural value of the building.

4) *Continuity of history*- The main question here is why the building is valuable. It is always better to make some changes to a building, that has curtain cultural value, even if they are significant, and keep it living this way, than to leave it into ruins or even demolish it.

2.2. Choice of interventions

For the correct choice of the interventions is necessary to follow some technical recommendations and criteria:

1) *Transparency of interventions* - the interventions should be as inconspicuous as possible. For example, seismic isolation of the foundation is highly transparent interventions, as it could not be seen unless a detailed inspection of the building is performed. External steel braces are at the other end – they would dominate the façade which makes them highly inappropriate for strengthening of heritage buildings.

2) *Reversal of interventions* - in the case that better decision is available in the future, or if the interventions proof themselves inappropriate, there should be the possibility to undone them without any lasting damage to the original building. The examples above are on the opposite sides here: seismic isolation is almost irreversible, while external steel braces could very easily be dismantled without any significant lasting effect on the façade.

3) *Compatibility of interventions*- any new materials and structures that are used should be compatible with the original structure. The compatibility has several aspects: appearance, stiffness, deformation capacity, modal modes, etc. If the new structure is not compatible, in the case of an earthquake the original structure may suffer significant damage even if the new structure behaves splendidly on itself.

4) *Durability of the new materials*- not only the initial durability is to be considered but also future survey and maintenance of both original and new structure.

5) *Avoidance of historical forgery*- if there are visible structural elements that are to be replaced (because it is damaged beyond repair or because it no longer exists) or added this should be done in such a way, that the new elements could not be confused with the original structure.



Fig. 1. Poor results of applied injection, (Binda et al., 2003)



Fig. 2. Damage to the change of stiffness (source POLIMI)

3. Some general mistakes

3.1. Assessment and understanding the building [1]

To know the building—to study and understand its materials, systems, condition, cultural and physical context, and history and alterations, before planning any intervention or alterations. The building is examined as a whole, in context, and each of its parts individually assessed. It is evaluated for aesthetic merit, unity or disunity as a formal work of architecture, and its historical value as a document of the passage of time or the flow of history. A historic structure report typically documents this analysis. Any perceived conflicts among these fundamental values are resolved at this juncture through the evaluation of all available information and the contextual weighing and balancing of evidence to reach reasoned, justified, and nonarbitrary decisions. As a part of this information-gathering and analysis process, the need for alterations to the structure is assessed, which may be anything from replacement of a broken windowpane to seismic retrofitting.

Multidisciplinary assessment of the structure and identification of all its cultural values and historic fabric at varying levels of significance are necessary. It is possible that previous unsuccessful results can underscore the need for adequate assessment prior to any restoration or rehabilitation.

The assessment and the understanding of the building entail a four-step process:

1) The structure is fully characterized;

2) Important features and significant characteristics are identified;

3) An understanding of the structure in its historical context is established;

4) Design options that are creatively respectful of the structure's historic fabric are developed.

In professionally managed projects, the historic structure report is prepared by a multidisciplinary team. The historical structure report provides a comprehensive overview of the significance of the building and its components, as well as details about specific features and construction history.

Mistakes or omissions in identifying the various values of a specific structure occur when undue weight is attached to one category of values or the other, reflecting the disciplinary background of the investigator.

Evaluation by a multidisciplinary team avoids this problem. In practice, aesthetic (visually manifested) values are often emphasized over the less tangible ones (such as historical significance or research potential) because the public generally expects historical monuments to look attractive and conform to contemporary notions of good taste regardless of their historic appearance.

A historic structure report is advisable when a major change of use, termed an adaptive reuse, is proposed that is likely to alter the physical manifestations or the record of the flow of history of the building. When major modifications are proposed, even for the sake of restoring the building to an earlier and possibly more structurally viable condition (such as reconstructing missing adobe transverse walls), the process of reversing the changes sets back the historical clock, so to speak.

When neither the real state of damage nor the effectiveness of repairs is known, the results of the intervention are also unpredictable. This was clearly shown by some repair failure even when advanced materials had been used; there are now enough information to support the choice of compatible materials and techniques for repair.

3.2. Minimal interventions, respect for structural authenticity

Interventions or alterations are minimized to preserve as much of the significant fabric of the building as possible, thereby safeguarding its authenticity while accomplishing whatever goal motivated the initial decision to make alterations. Interventions causing only a reduced impact on the original structure should be preferred, provided that they are enough to warrant the required safety level.

From an engineering point of view, the optimal solution is obtained after envisaging and analysing a series of alternative solutions. Reaching an "optimum" solution requires, as a first step, conceiving and tentatively developing a set of alternative solutions. Similarly, and as a first step, reaching an "optimum" intervention requires to the engineer or architect to foresee and develop a set of alternative possibilities involving different strategies, techniques or materials. Each different solution should then be evaluated regarding both its structural efficiency and compliance with conservation criteria (or, in other words, its cost in terms of loss of authenticity and cultural value). According to the ISO/FDIS 13822 final draft, the solution finally adopted should consists of a "minimal intervention", defined as "an intervention that balances the safety requirements with the protection of character-defining elements, ensuring the least harm to heritage values".

Monuments are also interesting and valuable because they constitute a structural achievement and provide an immediate and tangible experience on past construction technologies. Structures of monuments do not only constitute a document; they are in fact living legacies which, centuries after their construction, still carry out their resisting mission and keep on enduring loads, wind and earthquakes; they are a living and persistent proof of the skills of their creators and builders. Proper restoration of monuments must focus on preserving the original features of the structure. If repair or strengthening works are needed, they should cause the minimum possible alteration. This is not only applicable to the geometry and materials: the authenticity of the mechanical and resisting principles governing the structural response (the nature of the structure and its resisting mechanisms) is also to be preserved to the possible extent.

3.3. Reversibility

Reversibility holds that the alterations made to the building should be able to be removed in the future without significant damage to the building. Reversibility allows for the use of improved technologies as they are developed and the removal of inappropriate alterations. This principle encourages alterations of an additive nature and discourages the removal of material or architectural features. In addition, the permanent storage of any removed material or feature is important, to provide the opportunity for future replacement. Since all alterations become part of the history of the building, failure to achieve complete removability of the intervention results in permanent alterations.

3.3. Construction materials- old and new

The ICOMOS/Iscarsah Recommendations clearly state that "The characteristics of materials used in restoration work (in particular new materials) and their compatibility with existing materials should be fully established (as for example to avoid risk of negative chemical reaction, etc...). In any case, it has to be clear that compatibility is a necessary condition but not sufficient to accept a product because its benefit has to be demonstrated. This must include long-term effects, so that undesirable side effects are avoided" [4].

The new materials used for repair and strengthening will be chemically, physically and mechanically compatible. In fact it will be never possible to find after centuries mortars, bricks, stones, timbers equal to the original ones, even if they come from the same place or quarry. As a consequence of that, repair and improvement will be better than substitution.

The important tenets of contemporary architectural conservation include encouraging the use of identical, or similar but compatible, materials in the retrofitting and repair or replacement of deteriorated features to obtain similarity of performance.

The experience of the last decades in repair, strengthening and prevention for the preservation of historic buildings in seismic areas did teach that compatible or friendly techniques have to be chosen in order to obtain positive results.

In fact, the chemical, physical and mechanical differences were not as large as the ones between masonry and reinforced concrete, FRP or organic resins, lime mortars and cement mortars, etc...

Today the need for rehabilitation and repair of damaged buildings allows for the application and experimentation of both traditional and advanced techniques.

3.4. How to upgrade the performance? Seismic retrofit?

Natural disasters, combined with rehabilitation and adaptive reuse, have taken a serious toll on historic structures. Seismic retrofit measures must first minimize the risk to lives and then satisfy the conservation principles of minimal intervention and reversibility. Only when all those criteria have been met should retrofit measures address the issue of minimizing damage to the building during moderate and major earthquakes. For example, measures that provide for life safety might have little effect on cracking during a moderate earthquake and could allow significant and nonreparable damage to occur during a major seismic event. A building's shared history, beauty, place in the community and social life all contribute to its heritage and community value, which must be considered alongside its condition, occupant use and location in any retrofit strategy.

A strong heritage context may mean improvements to performance rely on services (heating, renewables) and behaviour (including maintenance and repair). The occupation context of overcrowding can put a lot of pressure on the health of building and occupants, particularly if the services and fabric are in poor condition. This can dominate performance more than external contexts.

The challenges of the retrofit in terms of performance are largely about understanding risks. These risks can be broken down into three overarching categories [3]:

1) Risks to building fabric and human health

2) Risks to heritage: damage to, and loss of, historic fabric; impact on neighbourhood or community

3) Risks to achieving expected energy savings; resulting environmental impact.

According to [2] the following issues should be considered: life-safety issues and conservation issues.

3.5. Functional changes

The new function assumed for the building should be compatible with its performance. In fact, if the new function requires the resistance to much higher state of stresses than originally, an invasive strengthening is usually required. A consideration, therefore, is to make sure that planned retrofit measures are appropriate for both current and future occupiers of a building.

4. Conclusions and recommendations

Several unsuccessful results have underscored the need for adequate assessment prior to any restoration or rehabilitation. In fact, when neither the real state of damage nor the effectiveness of repairs is known, the results of the intervention are also unpredictable. This was clearly shown by some repair failure even when advanced materials had been used; there are now enough information to support the choice of compatible materials and techniques for repair.

The preliminary condition and structural assessments should provide the information necessary to determine whether the building is in such condition that the design of a seismic retrofit project can proceed directly or whether other preservation treatments, such as additional structural stabilization or repairs, are needed first. The building must be physically surveyed, findings made, and recommendations formulated from the perspective of both preservation architect and engineer.

1) A deep knowledge is needed of the [4]: (i) building history and evolution, (ii) geometry, (iii) structural details, (iv) crack pattern and material decay, (v) wall construction technique and materials, (vi) material properties, (vii) structure stability. This knowledge can be reached through on site and laboratory experimental investigation, structural analysis using appropriate models and final diagnosis. Acquiring a clear and comprehensive understanding of the existing building prior to commencing design is crucial for the final results.

2) The preservation will be successful only if it is respectful of the existing buildings.

3) Very often providing new structure for seismic action is the only possibility. Using the existing elements sometimes is very expensive solution and very often this solution is contrary to the original vision of the historic buildings.

4) The innovative technologies for retrofitting are not suitable for every case.

5) When sustainable upgrades are being considered for an existing or traditionally-constructed building, the design approach must be based on a comprehensive understanding of the building's original materials.

6) The most important factor for the existing and historic buildings is occupancy. Not every building is suitable for every occupancy. The inappropriate occupancy may dictate inadequate interventions.

7) The project for retrofitting of historical buildings must take into account the position of the building, the type of structure and used materials, the constructive system, the damages in the structural elements, etc. Sometimes the change of those factors may make the project infeasible.

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НАПРАВЛЕНИЕ 4

Възстановяване на сгради от културно-историческото наследство, изпълнени от зидария, дървени или метални конструкции-критерии, иновативни решения и казуси; Сеизмична рехабилитация на сгради от културно-историческото наследство; Опазване и съхраняване на недвижими културни ценности; Консолидация, възстановяване и укрепване на исторически структури; Сгради и съоръжения принадлежащи към културно-историческото наследство - след катастрофални събития: пожар, наводнения и взривни въздействия.

TOPIC 4

Restoration of historical masonry, timber and steel structures-criteria - innovative solutions and case studies; Seismic rehabilitation of historical masonry, timber and steel structures; Conservation and preservation of historical masonry, timber and steel structures; Consolidation, rebuilding and strengthening of historic structures; Culture heritage under catastrophic events: fire, flood and blast loading.

STRUCTURAL IMPROVEMENT OF MASONRY HERITAGE BUILDINGS – THE CASE OF THE TEXTILE INDUSTRY „KOSTA STAMENKOVIC" IN SERBIA

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***Abstract**: In order to improve practice of preservation industrial heritage buildings and wider, this article attempts to explore and to present practical and applicable techniques of structural renovation of masonry buildings with monumental qualities. The appropriate constructive measures and techniques of improvement and strengthening the masonry structures were investigated on the case of the protected buildings of the textile industry complex „Kosta Stamenkovic", situated in the city of Leskovac, in Serbia. Due to the perennial influence of various internal and external factors, certain damages has been caused to building structure elements as mold, stains, soft and waste mortar, scratching bricks in the wall, cracks and bricks out of the wall. With the intention to return the former architectural glory to the complex and to activate new function (creative park), it is necessary to implement measures of reconstruction in accordance with the intended purpose and current legislation in the field of protection of cultural heritage. Based on the experience from the presented case study, recommendations will be made for further development and optimization of the applied techniques.*
***Key words**: structural improvement, practical techniques, masonry, industrial heritage*

1. Introduction

The industrial heritage plays an important role in the urban scene of the cities today. It represents a particular type of heritage caused by deindustrialization of society and advanced economies. Many sites known as industrial brownfields are neglected, abandoned and empty or in risk of destruction. The preservation of these buildings is delicate and complex tasks that claim a multidisciplinary approach which makes it difficult to fortify general rules and principles. The motives for conservation, maintaining and reusing existing buildings are not only a matter of preserving cultural and historical, meaningful and worthy buildings and identity of locations and "cities panorama" in general, but also economic and ecological profits for community.

In recent years, concept of integrated conservation is commonly adopted, which denotes a modern approach to preserving the architectural heritage, implies its

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reconstruction and integration into contemporary life, and emphasizes the incorporation of principles for the protection and management of cultural heritage in the decision-making process, especially at the level of urban planning. [1] Such an approach is applied in many countries that conduct complex actions to protect and enhance architectural heritage, not only important cultural properties, but also modest facilities, often only with ambient values. According to Petzet for a historic building conservation includes all measures that prevent further decay and preserve the historic fabric. This can encompass structural strengthening with appropriate auxiliary constructions, or the replacement and completion of components insofar as this prevents their further deterioration. [5] Penelis quotes basic principles for the structural restoration which are related to preserve and reveal the aesthetic and historical values, and classifies restoration techniques on reversible and irreversible with giving priority to reversible interventions. [4]

A notable number of industrial heritage buildings were constructed with masonry stone and brick. Older constructed and neglected buildings under the authority of the state suffered physical defects and as such demand certain repair or reinforcement for the aim to solve structural deficiencies, deterioration effects, or to remove problems related to seismic performances. These buildings often require severe interventions, conducted with techniques that include expanded use of the modern era innovations. Absence of precise guidelines can easily cause obscurities and poorly funded decisions on the basis of the past experiences that may also be with disadvantages.

Interventions on existing buildings are major issues insufficiently dealt with in Serbia. In view of that fact this paper examines practical and applicable techniques of structural renovation of masonry buildings with monumental qualities. The appropriate constructive measures and techniques of improvement and strengthening the masonry structures were investigated on the case of the protected buildings of the textile industry complex „Kosta Stamenkovic", situated in the city of Leskovac, in Serbia. With the intention to reuse the complex and to activate new function (creative park), it is necessary to implement proper measures in accordance with the intended purpose and current legislation in the field of protection of cultural heritage.

2. Methodological approach to the process of structural improvement of masonry heritage buildings

Existing building stock is exposed to processes of deterioration with time, followed by a situation in which they became unable to fulfil the original purpose and function. When it comes to masonry structures, one of the most vulnerable type of structures, in addition to the age of buildings and materials that are embedded, i.e. internal factors, numerous damage are caused by the influence of the outside environment factors, such as: settlement of supporting structure (land consolidation), inadequate drainage of atmospheric water, the presence of moisture in the walls, temperature changes, seismic effects, fires, explosions, usage and maintenance modes of the building and other influences (anthropogenic actions such as architectural alterations, intentional destruction, inadequate restorations). Mentioned damage in the matter of historical buildings, refer to damage of structural and non-structural elements. In such cases, it is necessary to take comprehensive approach and to conduct appropriate measures and procedures, depending on the degree and cause of the damage, and level of protection, in order to bring the structure of the building to a satisfactory condition.

The protection of building heritage is systematic implementation of passive and active protection measures (legislation and revitalization), aligned with the rules of conservation and restoration profession in order to preserve the cultural property, i.e. the implementation of measures in order to extend the duration of the monumental properties

of the cultural artefacts with the systematic monitoring of its condition and ensuring its protection against any threat. Protecting the cultural property requires a methodologically correct, logical and systematic approach. Depending on the level of protection, type, characteristics and status of the facility that need to be protected, different types of treatments and techniques can be applied. In general, the execution of works is preceded by a whole series of preliminary actions and steps that must to be conducted and which are commonly applicable to all types and methods of technical protection that are intended to preserve, conserve and protect buildings. Protection of existing buildings can be construed as the process composed of phases, mutually connected operations that happen in established order within certain time interval. Generally it can be separated the two main phases, diagnosis with preparation of projects and the execution of works.

Approximate display of the stages, through which protection of building is conducted, is illustrated in Fig. 1, and understands the following:

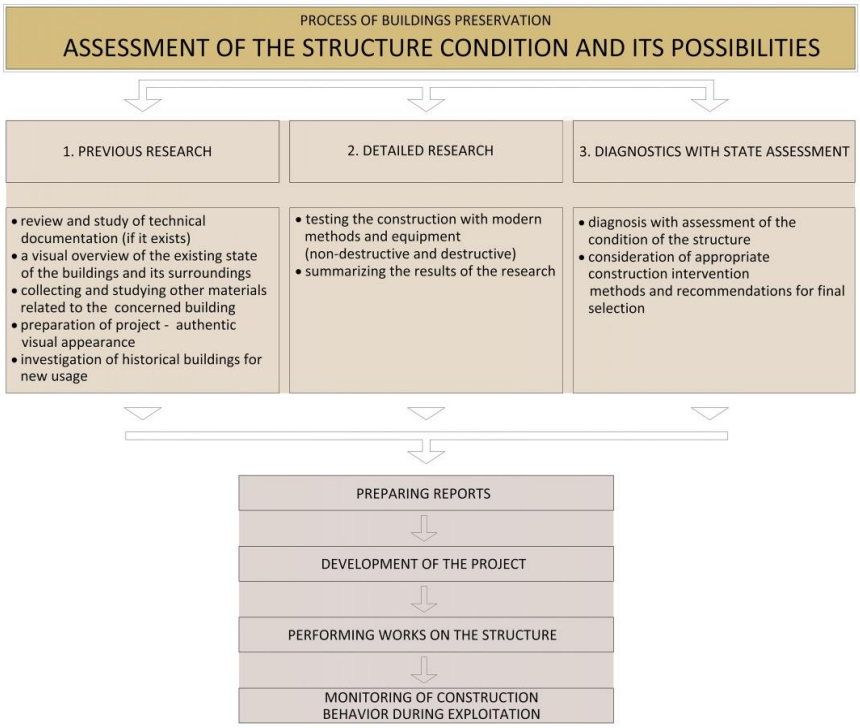


Fig. 1. Mandatory stages of the building protection process, adapted scheme [6]

- Research and recording of conditions in subject of protection from standpoint of organization and functioning; research and recording of conditions in macro and micro surrounding (value and significance, condition, architectural integrity and nature of the facility) – observing, photographic records, measuring, sampling, noting and documenting, interviews;
- Initial report based upon visual inspection listing all the defects (voids, cracks discontinuities, etc.);
- Analysis and evaluating of researched condition of the total structural performance, which includes analysis of obtained research results, tested indicators, measuring, calculations, etc.;
- Creation of authentic visual appearance, its detailing, dimensions, colours and textures (it is of prime importance and must be considered).
- Estimation of facility's condition as well as estimation of the factor of environment that acts either as a risk or threat;
- Adapting historical buildings for new usage may require applying new technologies or materials to enhance the level of comfort and suit the new usage of the building.

- Synthesis of findings and determining of field of protection;
- Final estimates and proposals with specifications and full report.
- Implementation of solutions into individual projects;
- Creation and development of selected project;
- Project realization (execution of works);
- Monitoring of construction behaviour during exploitation.

3. Analysis of substantial building in complex "Kosta Stamenkovic"

As one of the first factories of the city of Leskovac, located in a wider surrounding area of the city of Leskovac in Serbia, the complex of the textile factory originates from the golden age of prosperity of the city. Due to its historical and architectural values are rightfully protected as buildings with historical architectural properties and deserve to be revitalized (Fig. 2).



Fig. 2. Location of textile factory Kosta Stamenkovic, Leskovac, Serbia

According to the analysis of written sources it was built in 1921 as a factory of canvas and textile cables. It was founded by an Lazar Dundjerovic and com., later named factory of woolen and cotton fabrics Vranjkic, Stamenkovic and com (Fig.3). Nowadays the complex is abandoned for at least a decade. The existing complex is characterized by edge construction on the plot, with centrally positioned main factory. In site there is only one access to the complex from the street Dimitrije Tucovic over the entrance with the original iron gates design. Factory was built before the adoption of building regulations, so there is no technical documentation according to which buildings were constructed. Some of the facilities that are not under the state protection are in relatively poor condition and therefore it is necessary to foresee their removal. Buildings suffered numerous external and internal influences. The first large-scale attack on observed buildings was the flood that happened in 1948 (Fig. 3), which was repeated couple of time during the period 1960-1970. Due to the perennial influence of various internal and external factors, certain damages has been caused to building structure elements as mold, stains, soft and waste mortar, scratching bricks in the wall, cracks and bricks out of the wall.



Fig. 3. Kosta Stamenkovic in its former appearance and flood in 1948; Main building in copmlex.

3.1. Building's Structure and Technical Condition

Actually, complex consists of four buildings, constructed in different time period. The paper covers only building in the central part of the site that is under the state patronage and possess the first level of protection (building marked as "B" Fig. 2, Fig 4.) and building extending along the street Dimitrije Tucovic that possesses certain architectural values and the monumental features (preservation of the original facade shape, building marked as "A", Fig. 2, Fig. 4.).

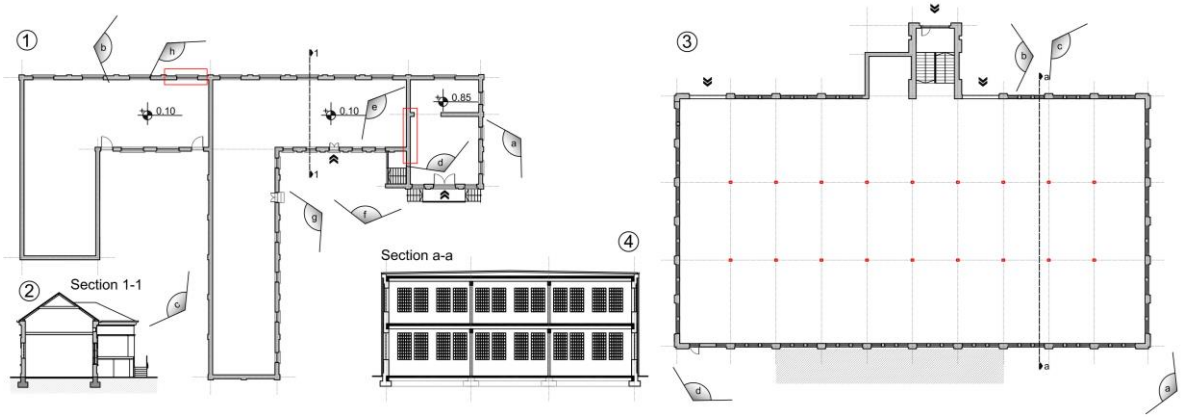


Fig. 4. Building "A", 1) Ground floor plan;2) section 1-1; building "B", 3) ground floor plan 4) section a-a

Building on the plot regulation "A". Building consists of ground floor and basement below one section of the building. In sense of structure, the constructive system of the building is massive, traditional, with longitudinal and transverse layout of bearing walls. External load-bearing walls are made of full brick, with a thickness from 51 cm in the basement to 38 cm on the ground floor and 25cm thickness on the brick cornice. Interior wall thickness ranges from 51 cm in the basement to 38 cm on the ground floor. Headers are made of arched bricks and in some parts of flat masonry clay element. Some severe cracks were spotted in the external and internal walls (Fig. 5. d and e). Roof structure is made of wooden elements and covered with clay roof tiles.



Fig. 5. External and internal appearance of building

Visual overview of the southern façade indicates serious damage caused by overload of the roof structure and poorly executed construction works (Fig. 6, f). Some of the main roof wooden elements were not fixed to the ceiling beams and then large horizontal forces were directly transmitted onto wooden beams and then on the brick cornice wall. This resulted in visible deformation of the walls in these places. Consequences of moisture due to capillary rise, unsuitable introduction of hydro isolation, damaged downpipes and

damage due to overloading were obvious (Fig.6, g and h). Deterioration due to moisture was extensive throughout the basement and partially in the ground floor. The main causes that led to a current condition of the facility are: non-maintenance over a long period of time, durability of materials, lack of waterproofing of the walls, and the appearance of cracks in the walls is a consequence of the overload from the roof, poorly derived connections between the roof, floor slabs and the wall cornice (deformation of wall cornice accompanied with cracks).



Fig. 6. Overloading - cornice wall deformation, deterioration due to capillary rise, freezing-thawing

Building under total protection "B". Building has two floors above ground. The structural system of the building in vertical sense is combination of steel pillars in the interior space, and massive peripheral walls. The floor slabs of the building are made of reinforced concrete slabs with ribs at 70 cm. Roof construction is in form of flat roof, covered with final layer of gravel. The massive walls of the building are made from brick with 51cm thickness. Windows and doors are damaged and ruined in many places, and in process of disassembly some of them, massive walls are shattered. In order to monitor the condition of the facility, it was of great importance to provide adequate photo documentation. Today's appearance of the building is shown in displayed Fig. 7. Visual inspection of buildings structure pointed to damage, such as scratching bricks in the wall, brick out of the wall, vegetation attack, mold, soft and waste mortar.



Fig.7. General view of building façades, Deterioration due to capillary rise, inadequate drainage, freezing-thawing

Detailed visual inspection of the buildings registered the following damages and defects:

- waterproofing of walls was not performed,
- removal of mortar, brick and other coatings,
- cracks and crevices of the building walls,
- the appearance of vegetation at base of the building,
- rotting the wooden roof structure, inadequate wood elements for roof construction are used,

- poor condition of joist ceiling,
- traces of water leakage, wet stains and scratching of wall elements.

Based on collected data through detailed visual inspection, it was concluded:

- global stability of the buildings is not disturbed, but the local stability of certain structural elements is threatened,
- load capacity is reduced,
- functionality of the buildings is disturbed,
- durability of the facilities have been significantly reduced.

4. Proposals for structural improvement

Building on the plot regulation "A". The durability of the masonry building was reduced, and in order to prevent further deterioration and progressive development of damage, the emergency protection measures are proposed. The dynamic unity of the walls should be achieved by performing reinforced concrete on top of existing wooden beam construction and anchoring it into the walls. The walls should be additionally strengthened by injecting with a mixture of hydrated lime, crushed brick, bentonite and cement. Anchoring and prestressing of the wall corner fallowed by confining of masonry wall with steel ties has been proved as very successful measure and it is strongly suggested (Fig. 8.). Due to low quality of the original walls, it is important that the roof walls-cornices should be partially reconstructed and the new roof constructed entirely from scratch. Recommended materials must be custom developed and compatible to match strength, stiffness and moisture transmission properties of existing materials. Aesthetics of the existing masonry should be unchanged despite the extensive nature of the wall injection, stitching, and the structural enhancement.

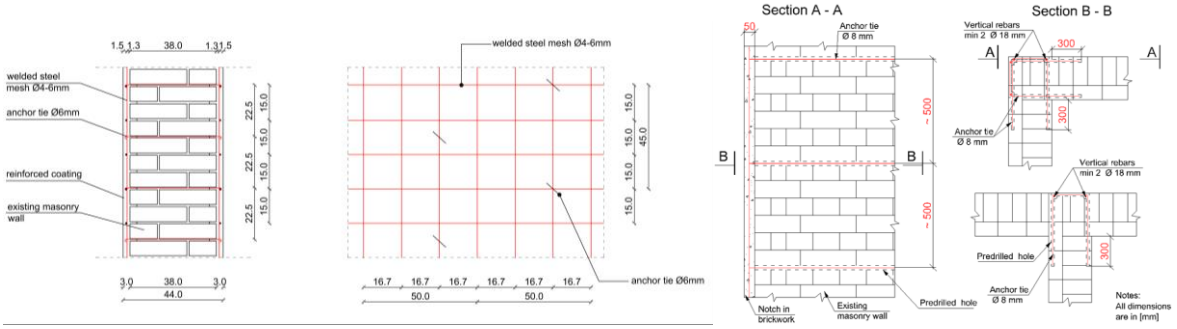


Fig. 8 . Detail for confining of masonry walls using vertical steel ties [10]

Building under total protection "B".The damage recorded on this building relates to façade walls defections, such as: mold, stains, waste and soft mortar, brick face flaking off and crumbling. In order to prevent further deterioration and progressive development of damage, it is proposed emergency sanitation technique that includes removing, cleaning, washing, replacing deteriorated mortar and filling joints with a new mortar. Injected mortar properties should be compatible with the properties of the masonry units, resistant to deterioration and it should have similar mechanical properties and durability as the original one. The main goal is increased the compressive and the shear strength, improved the appearance and reduction of deformation. [2, 3, 8].

5. Conclusion

The primary aim of protection of historic buildings and conservation is to prolong the life span, and to do so in a way that protects what is valuable about it. [7] After synthesis of exposed matter following conclusions can be made:

- It is of crucial importance to implement all stages and steps of the process of protection building, starting from field work, evaluating and establishing the state of structure, preparing of the project documentation up to the execution of works and monitoring of the facility.
- The two major goals of the strengthening in historic buildings are human safety and the protection of historical, architectural values of the building. Because each treatment can potentially harm fragile historic material and change building's historic character, it is important to put together a team of experienced professional in both construction and historic preservation. Intervention techniques in cases of masonry buildings deal with traditional intervention techniques being known preservation and restoration principles.

According to Valluzzi interventions to existing structures to repair damage, or strengthening and retrofitting to prevent it, should focus on the specific structural deficiency, and proper materials must be select to avoid worsening of current conditions from the physical, chemical and mechanical viewpoints. [9]

Typical improvements may be arranged according to their influences and can be sorted into three groups: surface repairs that improve the appearance, restores the non-structural properties and protect chosen component against the influence of weather, such as: repointing, pinning; structural repairs: injections of cracks, structural repointing; structural strengthening, which is relates to repairing or replacing of certain parts of a structure or entire structure, with the final result of restoring load capacity or additional strength. Basic goal is to change the structural damaged elements rather than to restore them, by adding new elements like bands, overlays, etc.

This research will serve as a guide for future similar operations.

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SEISMIC VULNERABILITY AND MAIN VIBRATION PERIOD OF HISTORIC MASONRY TOWERS: A PARAMETRIC INVESTIGATION

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***Abstract**: Italy is a country rich of masonry churches seriously damaged or collapsed after recent devastating earthquakes. In this framework, the current work shows the results of instrumental researches and structural analysis on the seismic behaviour of masonry bell towers made of light squared stones, which represent a peculiar construction of the church apparatus. Forty case studies representative of the different seismic areas of the country and belonging to thirteen regions have been examined through macro-elements models implemented with the 3MURI software. Parametric non-linear static analyses have been performed on masonry towers with assigned plan dimensions by changing their height and thickness. From the achieved results the individuation of the bell towers most at risk placed in different parts of Italy has been faced. Finally, a new third order polynomial formula has been proposed to calculate the main vibration period of inspected masonry towers as a function of their slenderness.*

***Key words**: Masonry bell towers, Parametric analysis, Non-linear analysis, Vibration period, slenderness.*

1. Introduction

Historical masonry towers are distinctive monumental constructions widespread in Italy, where in the past they assumed a remarkable strategic and military importance and were erected according to different geometrical configurations [1]. In general, the construction process and the architectural alterations, under form of additions and/or reconstructions due to previous hazardous events like, for examples, fires and earthquakes,

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should be known for evaluating the static and seismic behaviour of historical masonry towers.

Apart civic towers, masonry bell towers with different architectural styles and geometries were built in the past as typical constructions belonging to ecclesiastical complexes. These towers are very sensible to earthquake actions. In particular, their dynamic behaviour is strongly influenced by the slenderness, a parameter conditioning the failure, that can be of shear (massive towers) or compression-bending (slender towers) type. For slender construction a cantilever behaviour is expected, but a good connection among walls is needed in order to prevent local failure modes.

The seismic behaviour of bell towers is also influenced by other structures, like the façade of the church, which limit their movements under seismic actions, both modifying the vibration periods and inducing some stress concentrations in the tower parts attached to other structural walls.

Spire, belfry and other architectural elements on the top of towers could modify their seismic vulnerability. Moreover, due to both the low stabilising vertical load and the dynamic amplification effects which they are subjected to, these elements are very sensible to earthquakes. In addition, diffuse openings introduce in the structure highly vulnerable zones towards horizontal actions.

In the current paper, in order to assess the seismic behaviour of bell towers, the simplest cases of stocky and slender structures, considered as isolated structures without the presence of openings and any other architectural elements, have been examined.

A wide parametric analysis on numerous case studies having variable thickness and height has been done, as shown in the next sections, with the final goal to both individuate the bell towers most at risk on the Italian territory and to propose a new formula for assessing the main vibration period of towers made of light squared stones.

2. Classification and distinctive geometrical features

From the architecture point of view masonry towers can be framed within the following typologies: 1. Cobble bells; 2. Tower bells; 3. Ribbed bells; 4. Bell towers crowned with wrought iron; 5. Bell towers with pseudo-dome having multilinear profile; 6. Bell towers from the Atriana's school; 7. Bell towers from the Lombardy's school and 8. Twin bell towers. Independently from the typology, masonry bell towers can be also classified on the basis of their slenderness λ, intended as the ratio between the total height H and the width B. In particular, for λ values within the range [11÷19], the slenderness can be defined as medium-high, whereas for λ values within the range [3÷11], a medium-low slenderness can be assigned to towers. In addition, for these structures another slenderness based on the ratio between the height H and the thickness t can be also defined.

In the current study a review of masonry bell towers placed on the Italian territory has been made. In particular, forty towers located in thirteen Italian regions with medium-low values of slenderness have been examined. The geographical distribution of towers into Northern, Central and Southern parts of Italy is shown in Tables 1, 2 and 3. In these tables the main geometrical properties of towers, namely the longitudinal direction width B, the transverse direction width b, the height H, the thickness t and the radius r (in case of towers with circular plan) are reported.

3. The macro-element models and parametric analyses

The generation of tower specimens to be investigated under numerical way has been done through the 3MURI non-linear analysis software [2], a structural program for seismic calculation and verification of masonry structures according to the Italian code M.D. 14-01-2008 "Technical standards for constructions" [3]. 3MURI examines large and small masonry structures and mixed structures made up of masonry elements with r.c., steel and

wood members for the seismic design of new structures or for the check of existing ones. The strength point of the program is the Frame by Macro Element (FME) innovative calculation method, which schematises before the structure with a number of macro-elements (piers, spandrels and joints) and subsequently transforms them into an equivalent three-dimensional frame. In this way it is allowed to grasp the most of masonry structures seismic behaviour, so to provide to the designer necessary information for an accurate exam of the structures themselves. The software studies the structure global behaviour through linear static, linear dynamic and non-linear static analyses, allowing to know the structural behaviour under gravity loads, information on vibration frequencies and periods and seismic checks according to a Force-Based approach.

Table 1 Masonry bell towers in the Northern Italy

Region	B (m)	b (m)	H (m)	t (cm)
Emilia-Romagna	1.8–12	1.8–12	7-86	30-257
Friuli Venezia Giulia	6.35	6.65	47	200
Lombardia	10	10	54-74	140-200
Piemonte	r = 4.22		30	200
Veneto	3.7-15	3.7-10	20-58	60-200
Toscana	5.2-10	5.6-9.4	27-55	130-260

Table 2 Masonry bell towers in the Central Italy

Region	B (m)	b (m)	H (m)	t (cm)
Abruzzo	4-10	4-10	20-50	130-150
Lazio	9	8.14	36	115
Marche	2.40-9	2.40-8	4-45	65-120

Table 3 Masonry bell towers in the Southern Italy

Region	B (m)	b (m)	H (m)	t (cm)
Campania	6-13	4.8-13	30-75	60-100
Molise	4.8-6.8	4.8-7.1	20-35	100-200
Puglia	9	8	60	136
Sicilia	6	6	35	88

According to the Italian Ministerial Circular (M. C.) n. 617 [4], considering that the in situ checks on materials have not been performed since ideal case studies of masonry towers have been considered only, the knowledge level LC1 has been assumed. Therefore, a confidence factor FC equal to 1.35 has been considered. Mechanical strength and Young modulus of masonry, considered made of squared blocks of soft stones (e.g. tuff, limestone, etc.), have been calculated as the lowest value and the average one, respectively, of those reported in the Table C8A.2.1 of the M.C. n. 617 (2009). About the nominal life V_N of towers, 50 years have been considered. Under seismic actions, with reference to the consequences of either an use interruption or an eventual collapse, these constructions have been classified into the II class (normal importance structures).

The subsoil category of structures, in absence of specific investigations, has been assumed of type *C*. Due to the lack of appropriate local seismic response analyses, the T_1 topographic category, with S_T=1, has been taken into account. The towers have been covered by a r.c. plane roof having dead and permanent loads of 4 kN/m^2 and live loads of 1 kN/m^2. Therefore, on the basis of the information collected on masonry towers in a previous research work [5], average values of their planimetric dimensions have been considered for each of the three areas in which Italy is divided. Particularly, plan shapes

measuring 7.8 m x 7.8 m (North), 7.0 m x 7.0 m (Centre) and 7.5 m x 7.5 m (South) have been selected as representative dimensions of structures. Starting from these values, the variation of both the height and the thickness of the towers has been considered in order to generate a set of samples to be investigated under numerical way. More in detail, the variation of height has been made for each 20 m, whereas a variation step of 20 cm has been considered for the thickness. A summary table indicating the main geometrical dimensions of investigated towers and their variation ranges is illustrated in Table 4.

Table 4 Variation ranges of geometrical dimensions of examined towers

Italian area	H (m)	λ (H/B)	t (cm)
North	20-100	3-13	40-260
Centre	20-60	3-9	60-160
South	20-100	3-11	60-200

For seismic verification it has been considered also the location of towers in different Italian regional capitals. Therefore, in the Northern Italy seismic analysis checks have been performed in Bologna (Emilia-Romagna), Firenze (Toscana), Milano (Lombardia), Torino (Piemonte), Trieste (Friuli-Venezia-Giulia) and Venezia (Veneto). The same checks have been performed in Ancona (Marche), L'Aquila (Abruzzo) and Roma (Lazio) as representative cities of the Central Italy, whereas Bari (Puglia), Campobasso (Molise), Naples (Campania), Palermo (Sicilia) have been taken as regional capitals distinguishing the Southern Italy.

The generic macro-element model representative of investigated towers and the main failure mechanisms occurred under static non-linear analyses are depicted in Figure 1.

In Figure 2 the pushover analysis curves covering the seismic behaviour of all the analysed towers within the three Italian areas are shown.

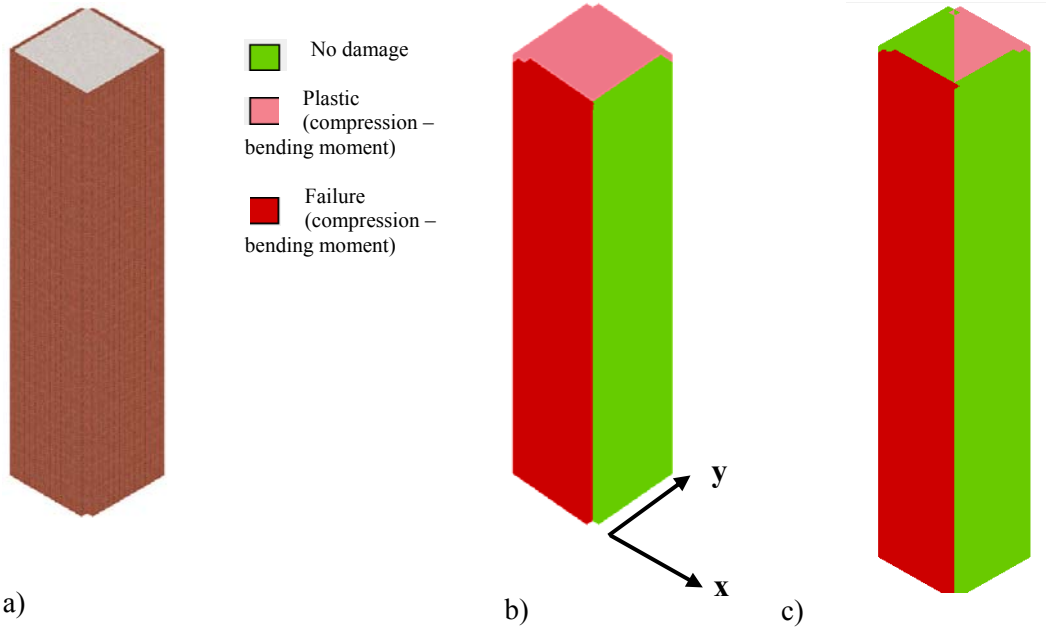


Fig. 1. Seismic pushover analysis in direction *x* with accidental eccentricity: the 3Muri model (a) and collapse mechanisms for the shortest tower (b) and the remaining ones (c)

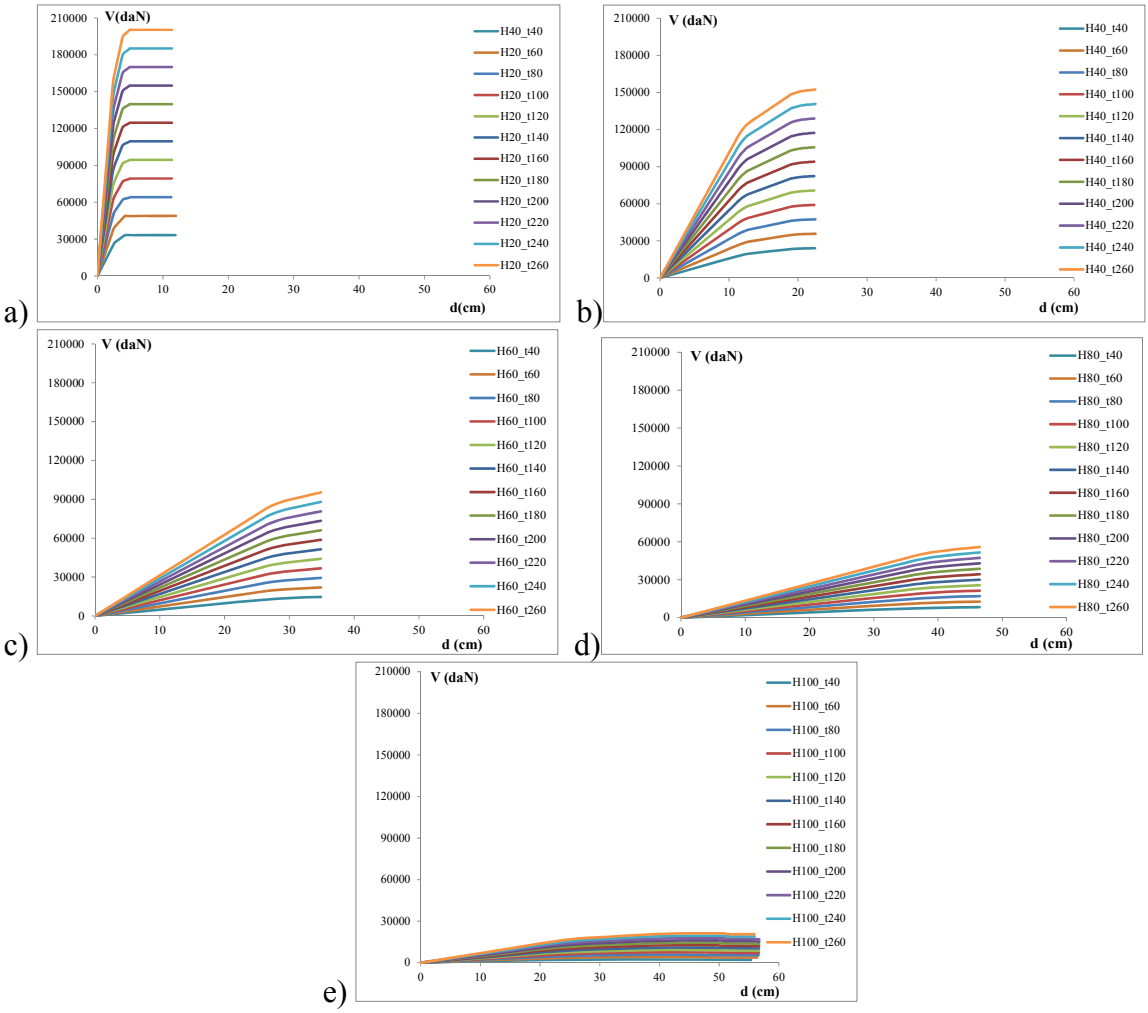


Fig. 2. Pushover curves of examined masonry towers with height of 20 m (a), 40 m (b), 60 m (c), 80 m (d) and 100 m (e)

From the above analyses the maximum base shear V_y and the initial stiffness K of curves can be derived and subsequently plotted as a function of the tower thickness t. The variation trend of these physical quantities with the tower thickness for the three different areas of the Italian territory are illustrated in Figures 3 and 4.

From results depicted in Figure 3 it appears that the increase of the base shear with the thickness follows more or less the same trend for towers within Northern and Southern Italy, whereas this augment of performance is less marked for Central Italy towers. This is due to the slight difference in terms of planimetric dimensions of the investigated towers.

In Figure 5 also the variation of the pushover curve initial stiffness K with the slenderness H/t of the towers is plotted.

From the above figures it is evident that the increase of stiffness as the thickness increases (or the slenderness decreases) is more pronounced for towers having height of 20 m. Differently, towers with height greater or equal than 40 m have not a significant variation of stiffness with increasing thickness.

In addition, the seismic behaviour of towers placed in the three Italian areas has been investigated in the non-linear static field by evaluating the pushover results in terms of the α_{SLV} parameter, that is the ratio between the capacity acceleration and the demand one. Therefore, α_{SLV} factors greater or equal than one are representative of safety conditions.

The minimum α_{SLV} factors deriving from pushover analysis results achieved on the towers sited in the different Italian regional capitals considered are listed in Table 5.

The seismic checks of Table 5, except for the cases of L'Aquila and Campobasso, are always satisfied. Moreover, the average values achieved from recorded safety factors allow to declare that the Northern Italy towers are less susceptible to undergo damages under earthquake. On the other hand, the lowest average safety factor observed has been found in Central Italy, with a value similar to that recorded for Southern Italy towers. However, since these average factor values are greater than one it can be declared that, if towers behave as a unique structural element without showing local failure mechanisms, their seismic behaviour is averagely satisfactory.

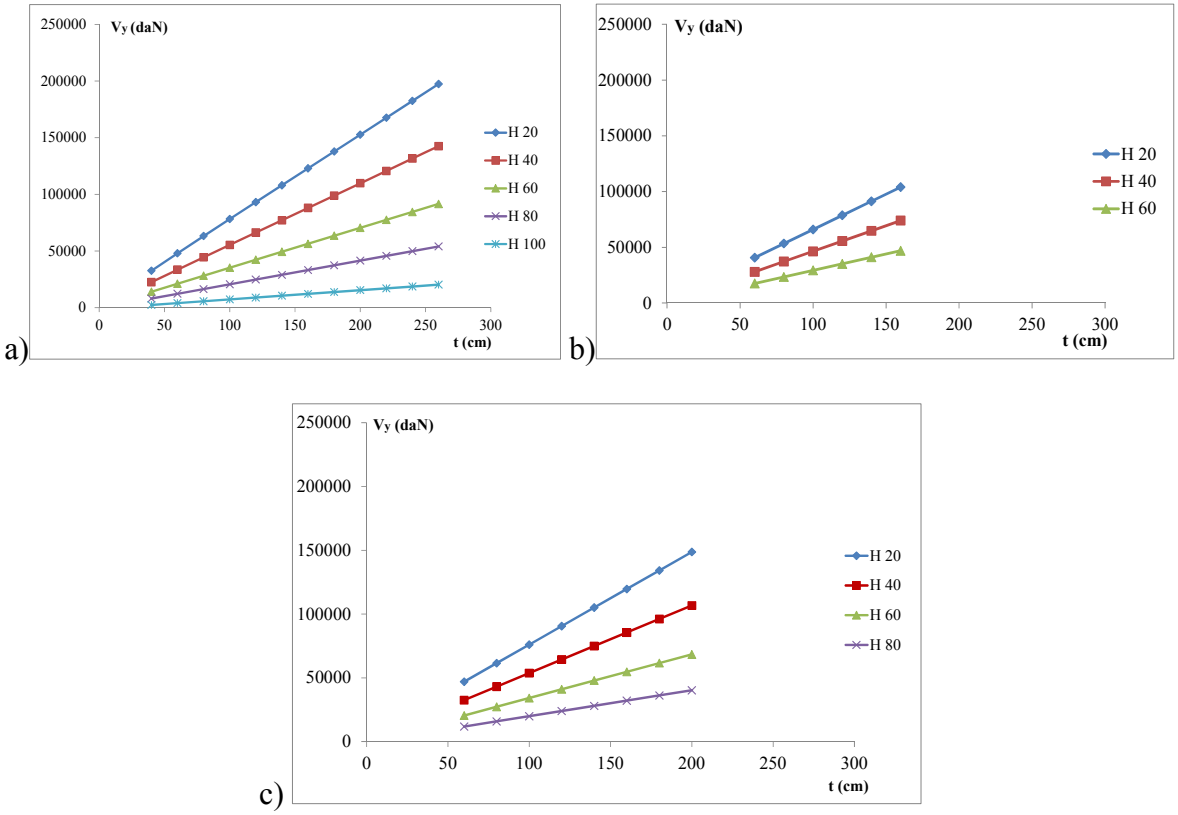


Fig. 3. Maximum base shear vs thickness for masonry towers from Northern (a), Central (b) and Southern (c) areas of Italy

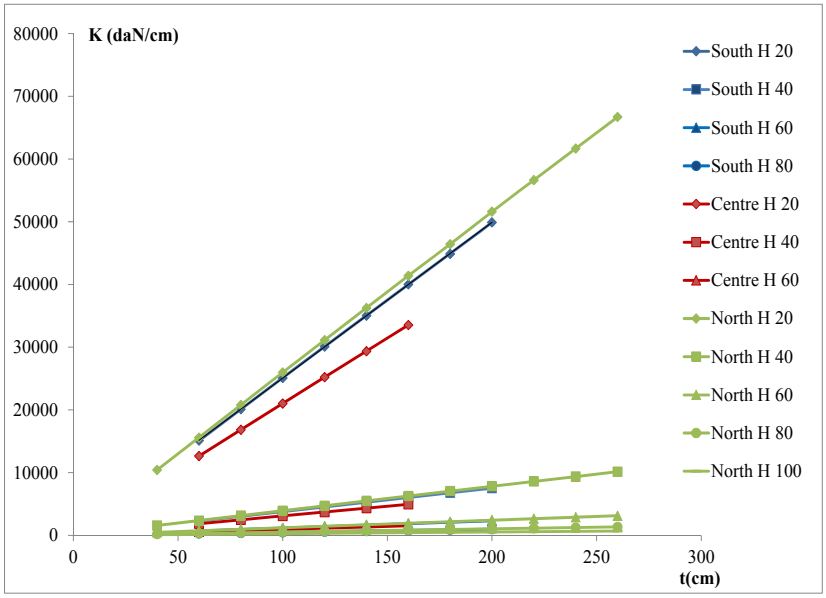


Fig. 4. Stiffness *K* vs. thickness *t* curves for the investigated towers

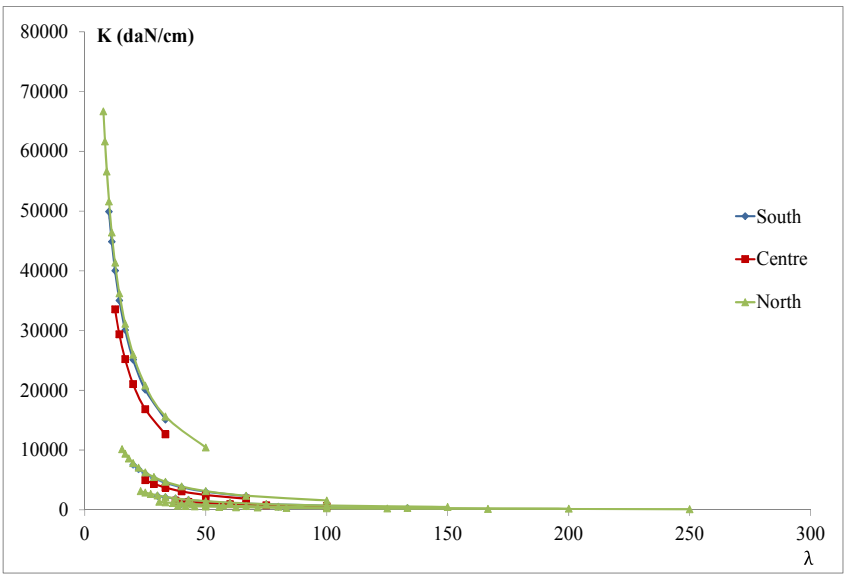


Fig. 5. Stiffness *K* vs. slenderness *H/t* curves for the investigated towers

Table 5 Minimum safety factors derived from pushover analyses on towers placed in different Italian areas

Italian area	City	min α_{SLV}
North	Bologna	1.41
	Firenze	1.81
	Milano	4.56
	Torino	3.98
	Trieste	1.80
	Venezia	2.66
	Average	*2.70*
Centre	Ancona	1.37
	L'Aquila	0.81
	Roma	1.95
	Average	*1.38*
South	Bari	2.12
	Campobasso	0.88
	Napoli	1.34
	Palermo	1.39
	Average	*1.43*

4. Linear dynamic analyses

In order to know the seismic behaviour of masonry towers in the linear field, the main vibration periods deriving from the 3MURI analyses have been compared with those achieved from the following relationships:

$$T = 0.05 \cdot H^{3/4} \quad (1)$$

$$T = 0.013 \cdot H^{1.10} \quad (2)$$

$$T = 0.06 \cdot \frac{H}{\sqrt{B}} \sqrt{\frac{2H}{2B+H}} \tag{3}$$

$$T = 0.2 \div 0.25 \cdot \sqrt{\frac{H}{B}} \tag{4}$$

In particular eq. (1), valid for tower height up to 40 m, and eq. (3) are derived from M.D. 14/01/08 [3] and the Ministerial Circular of Public Works 24/05/82 n. 22631 [6], respectively, whereas eq. (2) has been provided by Fabbrocino and Rainieri [7] and eq. (4) has been found in Di Gennaro [8].

The comparison among numerical curves and those derived from the previous equations is reported in Figures 6 and 7 as vibration period T vs. tower height H and as vibration period T vs. tower slenderness λ (overall height/width), respectively.

From this comparison we can see that the vibration period values calculated with the above formulas are too on the safe side, since they largely underestimate the T values achieved with the 3MURI software.

Moreover, the formula that approaches better the trend of the 3MURI curve is the eq. (4). Nevertheless, for tower height less than 40 m, this relationship provide unsafe results, with vibration periods greater than numerical ones. Contrary, for heights greater than 40 m, the results are better than those provided by the other formulas and are more close to the 3MURI ones.

In conclusion, after this comparative comparisons, a new theoretical third degree polynomial formula (eq. 5) has been proposed to calculate the vibration period T as a function of the tower slenderness λ. This new relationship is able to predict on the safe side the tower vibration period, since it envelopes inferiorly the 3MURI curves.

$$T = 0.0117\left(\frac{H}{B}\right)^3 - 0.1127\left(\frac{H}{B}\right)^2 + 1.109\frac{H}{B} - 1.6902 \tag{5}$$

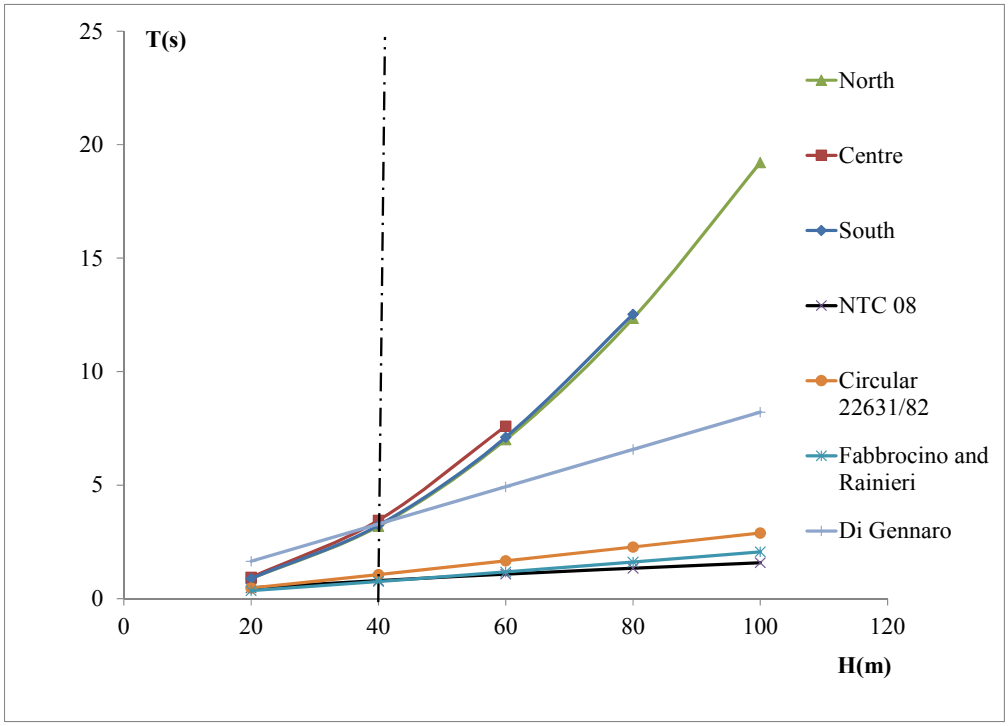


Fig. 6. Numerical-theoretical comparison of vibration period T vs. tower height H curves

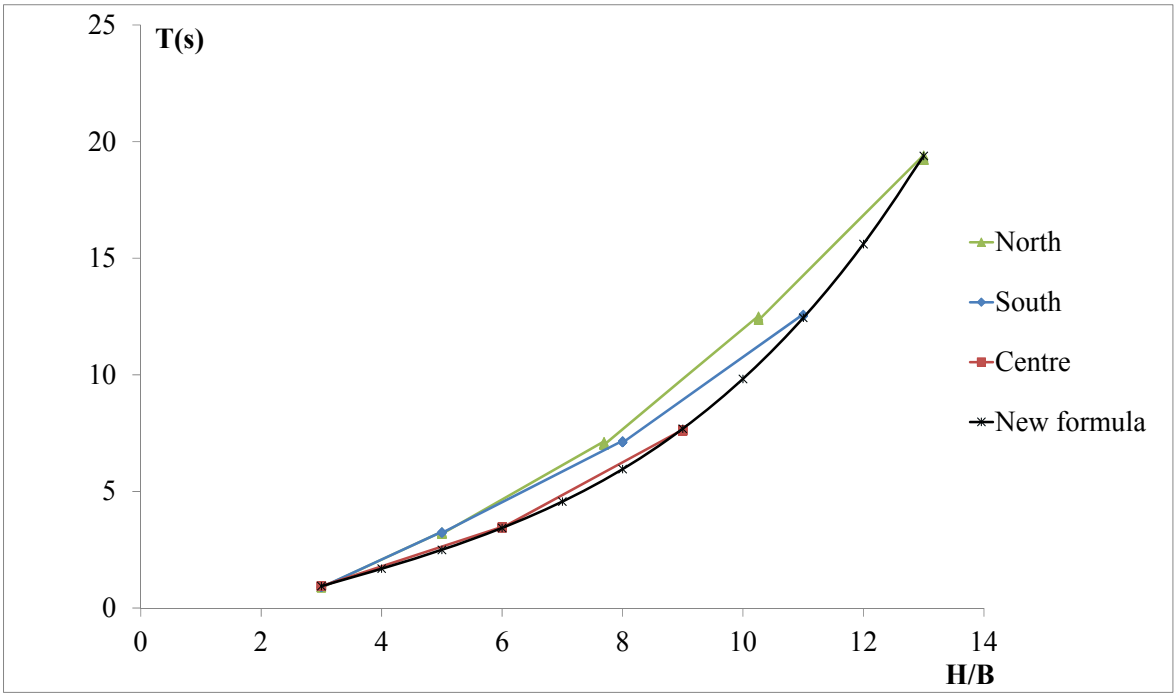


Fig. 7. Vibration period T vs. tower slenderness λ numerical curves

5. Concluding remarks

In the current paper, after a brief introduction on masonry bell towersmasonry bell towers, an extensive parametric numerical analysis by using the 3MURI software has been performed on some case studies representative of Northern, Central and Southern areas of Italy.

Non-linear static and linear dynamic analyses have been carried out on forty specimens of masonry towers having fixed plan layout and different geometrical properties in terms of total height and thickness.

From non-linear static analyses, considering the slight difference in terms of plan dimensions of masonry towers of the diverse Italian areas, it has been found that the base shear increasing with the thickness is similar for Northern and Southern Italy structures, whereas this trend is slightly less marked for Central Italy towers. Moreover, the stiffness of pushover curves strongly increase with the thickness for towers having height of 20 m only, while for larger heights this augment is not basically remarkable.

The pushover analysis results have been evaluated also in terms of the α_{SLV} safety factor, that is the ratio between the capacity acceleration and the demand one, related to a series of case studies belonging to several Italian regional capitals. These seismic checks have been always satisfied, except for towers within cities of L'Aquila and Campobasso. Moreover, the average values of recorded safety factors allow to say that the Northern Italy towers are less vulnerable than other inspected structures. Instead, Central Italy towers have the lowest average safety factor, with a value similar to that of Southern Italy ones.

Finally, linear dynamic analyses have been performed by the 3MURI software with the purpose to compare the main vibration periods with those deriving from four literature relationships. The comparison has shown that these relationships provide results largely on the safe side in predicting the main vibration period of towers. In addition, the relationship providing the best prediction of periods is that found in Di Gennaro (2010). In conclusion, based on the 3MURI analysis results, a new third order polynomial formula has been proposed to calculate the main vibration period of towers made of light squared stone towers as a function of their slenderness.

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MANHATTAN BRIDGE:1909-2017

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Abstract: *With a main suspended span of 448 m, Manhattan Bridge never held a record for length among suspension bridges worldwide, but it established many firsts, when it opened in 1909 in New York City. The design was the first to apply Melan's theory of large displacements. The suspension cables were the largest to date. The rail and vehicular traffic was the heaviest for any suspension bridge. At age 108 in 2017 traffic is still the heaviest for an East River, and the bridge continues to attract state-of-the-art engineering innovations. Over the last 35 years it has undergone unique structural modifications at a cost approaching $US 1 billion. The suspension cables have been rehabilitated with new reconfigured suspenders. The decks are replaced and their stiffening is upgraded. Finding optimal life-cycle solutions for Manhattan Bridge has advanced the art and science of bridge management. Some of these solutions are described herein.*

Key words*: Bridge, cable, fatigue, suspender, torsion*.

1. The Origin

Following the Brooklyn (1886) and Williamsburg (1903), the Manhattan was the third East River suspension bridge to provide vehicular and rail traffic between the New York City boroughs of Brooklyn and Manhattan. It was opened officially on December 31, 1909, by Mayor George B. McClellan, Jr., whose term was expiring on that date. About 30 m of the Bridge lower roadway over Division Street in Manhattan consisted of temporary planking to allow the passage of the Mayor's motorcade (NYT, Jan 1, 1910). The Second Avenue elevated portion of the subway had to be lowered 6 feet over a length of 244 m to accommodate the bridge clearance in that area. Manhattan Bridge is 1,761.4 m long between abutments at the lower level, and 1,855 m between portals on the upper levels. Both approaches are supported by 3 and 4 span continuous Warren trusses. Several stringer and floorbeam spans support the upper roadways between portals and abutments. The main suspension bridge is 890 m long, with a main span of 448 m and two 221 m side spans. Four 7.3 m deep stiffening trusses (designated as A, B, C, and D from South to North) run between abutments. They are supported by piers on the approaches and by the four main cables on the suspended spans. Their spacing is 8.5 – 8.7 – 8.5 m. The Brooklyn and Manhattan bound upper levels rest on trusses A – B and C – D, respectively. All other traffic is at the lower chord level. Figure 1–a shows the original elevation and cross section of the bridge along with some details related to its construction. Figure 1–b illustrates its location. As illustrated in Fig. 2, the bridge has

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always carried the most people of any East River crossing. Originally, it was designed for railroad on the upper level, trolley cars underneath, and vehicular traffic on a wood block deck in the center of the lower level. The structure now supports four vehicular lanes on the upper level; three lanes of vehicular traffic, four subway transit tracks, a bikeway and a walkway on the lower level. Recent traffic counts surpass 400,000 commuters daily (75,000 vehicles, 330,000 mass transit riders, and 2,000 bikers and pedestrians). Figure 3 shows a general view of the bridge.

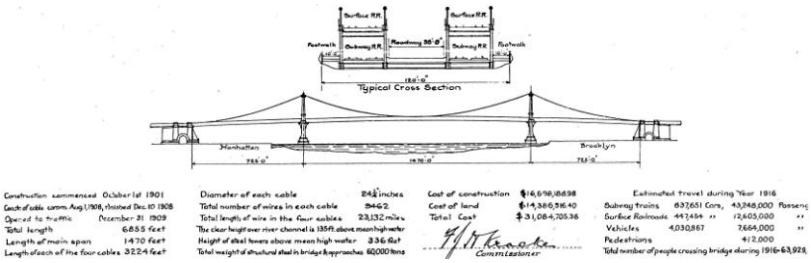


Fig. 1. – a) Manhattan Bridge, 1909

The calculations for the design were performed by Leon S. Moisseiff who graduated from Columbia University in 1895 and worked as a draftsman under R. S. Buck on both the Queensboro Bridge and the first Manhattan Bridge design. During the third design, Moisseiff worked under R. S. Buck (who was employed again by the Department of Bridges after George Best was appointed Commissioner) and O. F. Nichols (Griggis, 2008, p. 271). Some features from a previous design by Gustav Lindenthal were retained, but the pinned bases and much of the bracing were removed between the center columns (Griggis, 2008, p. 271). Moisseiff designed the wire suspension bridge in six months using the newly developed Deflection Theory to reduce steel weight and cost. This was the first application on a bridge, let alone an eccentrically loaded railroad bridge. Prior suspension bridges were designed with elastic theory emphasizing deeper trusses (Winpenny, 2004, p. xvii).



Fig. 1. – b) Brooklyn, Manhattan and Williamsburg Bridges across East River

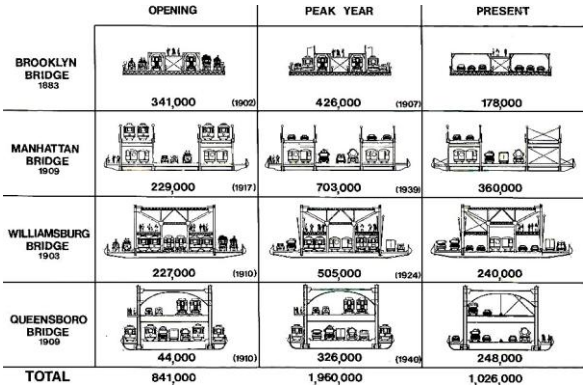


Fig. 2. Use of the East River crossings from their opening to 1988



Fig. 3. Manhattan Bridge viewed from the Brooklyn tower, 2016

The "deflection" or "more exact theory" was developed by Joseph Melan in 1888 and received an authorized translation by another Columbia graduate, David Steinman (1913). Prior suspension designs had used the Elastic Theory developed in 1826 or Rankine Theory developed in 1858. A Fourier Series Treatment of Deflection Theory was added in 1930 (Steinman, 1941). Steinman noted that the values of the bending moments and shears produced by the Elastic Theory are too high, thus satisfying safety, but not economy, and that the Elastic Theory is generally sufficient for short spans with deep rigid stiffening systems (Steinman, 1929). Unlike the Elastic Theory, the Deflection Theory does not assume that the ordinates of the cable curve remain unaltered under live loads and the lever arms of the cable forces are taken into account (Steinman, 1929, p. 248). According to the deflection theory, the work of the dead and live loads equals the total internal work expended in deforming the cables, suspenders, and stiffening trusses throughout the span. The stiffening truss is erected and adjusted at mean temperature to minimize its response to dead load (Burr, 1913, p. 212). The live load is distributed in two parts. A smaller fraction of it is resisted by the stiffening truss, while the rest is transmitted to the cable through uniformly distributed suspenders (Burr, 1913). As a result, reducing the truss stiffness simply sends more loads to the cables. Melan theorized that the maximum span of 4,694 m was obtainable if the bridge carried only its own weight (Steinman, 1913, p. 17). The effect on the torsional resistance of the cross section to transverse loadings, such as wind, is ignored. Armed with Melan's large deflection theory, designers consistently reduced the stiffness of suspension bridges, culminating with the collapse of Moisseiff's Tacoma Narrows Bridge in 1940 in a moderate wind. Only then the aerodynamic characteristics of suspension bridges were adequately linked to their stiffening by longitudinal trusses or box-girders. Fabrication for the superstructure steel for the main bridge began in August 1906. One year later, towards the end of the workday on August 29, 1907, the south arm of the cantilevered Quebec Bridge collapsed sending 83 workers into the St.

Lawrence River, killing 75 (Winpenny, 2004, p. 90). The company supplying steel on the Quebec Bridge happened to be the same as the one awarded contracts for steel fabrication and erection of the superstructure of the Manhattan Bridge. Phoenix had the contracts to provide the structural steel for the anchorages, towers and trusses (Winpenny, 2004, p.16). Memories also held that Phoenix was involved in construction of the Louisville Bridge which collapsed in December 1893 during high winds, killing 20 (Winpenny, 2004, p. 27). Although the construction of a suspension bridge is inherently safer than that of a cantilever bridge, there were justifiable calls for precautions, and in response, the Department of Bridges retained Ralph Modjeski to investigate the Moisseiff design. This included investigating the type of the foundations, stresses in the cable and stiffening truss, corrected dead load values and conductivity of heat in the main cables. At the time, the maximum theoretical and practical loading was 27,226 kg/m. Modjeski's report (1909) noted that this rare maximum loading would not reach 80% of the elastic limit stress. The towers and floor system are of carbon steel and the trusses are of nickel steel. This was the first use of nickel-alloy steel on a major bridge in significant amounts, including for the riveting. Investigation showed that the first slip of the plates detected 650 to 1000 kg/cm^2 for field riveted joints (by pneumatic hammer) and 720 to 1,260 kg/cm^2 for shop riveted joints (by a pressure machine). Modjeski observed that, had these higher values been known, no doubt some allowance would have been made for stress reversals, resulting in a more efficient design. He concluded that "the structure as a whole has been carefully designed, and when complete will be amply strong to carry the heaviest traffic…as well as any reasonable increase in weight of properly regulated traffic it may be called upon [to support] for many years to come." The original design loads assumed four lines of crowded Long Island Rail Road cars, 4 lines of Brooklyn Rapid Transit cars, 4 vehicular lanes and two pedestrian walkways. At an average of 2,812 kg/cm^2, the yield stresses for the fabricated carbon steel used in the towers were 20% higher than specified. The yield stresses for the fabricated nickel steel trusses averaged at 4289 kg/cm^2, or 10% higher than specified. The suspended structure was designed for dead load, including the cables of 37,180 kg/m and a working live load of 11,672 kg/m or congested live load of 2,722 kg (Perry, 1909, p. 51). The maximum dead load of 29,743 kg/m, resulting in a factor of safety (FS) of 2, stretches the cables up to 91 cm. Reaching the elastic limit would produce a 3 m extension (Perry, 1909, p. 65). "The maximum stress on the tower and stiffening truss would occur at congested loading and maximum temperature…Snow loading is offset by the lower temperatures… this principle would not apply to cantilevered bridges."

1.1. Construction Firsts

The Manhattan Bridge was originally referred to as the Third East River Bridge, but because of the redesigns and rebidding of contracts, it became the fourth East River Bridge to be completed. Even though the construction timeline shows 17 years from the beginning of the tower foundations in 1901 to opening for full service on the Brooklyn Rapid Transit lines in 1918, once the steel tower work started, construction of the towers and superstructures set records and the bridge was substantially completed in 3 years, totaling 42,000 tons between anchorages. Many of the modern construction techniques for suspension bridges were developed and used for the first time on the Manhattan Bridge. The speed of constructing the main bridge was partially attributed to the fact that the steel towers, cables, suspenders and suspended superstructure were included in one contract, thereby "eliminating multiplicity of plant, friction between contractors and possible consequent litigation with the City." (Johnson, 1910, p. 28) There had been 3 contracts let for the main bridge steel of the Williamsburg Bridge. The single contract facilitated orderly fabrication and building of the towers, cables and suspended spans in an overlapping sequence, without intervals of lost time.

- Caisson Construction

The foundation contracts for the Manhattan and Brooklyn Towers were advertised separately. The caisson for the Brooklyn tower foundation was floated into place in February 1902 and the cutting edge rested at an average depth of 27.75m below MHW or about 18.9 m below the river bottom, the material was described such that it required a pick ax to loosen and was a perfectly reliable foundation. A few cases of caisson disease developed, two of which were fatal. The 23.8 x 43.9 m timber caissons were constructed 13.7 m high in Manhattan for the tower foundation and 17 m high for the Brooklyn foundation to accommodate the plans showing an anticipated depth of 24 m below Mean High Water to a bed of gravel in Manhattan and 28.7 m below MHW in Brooklyn. The Manhattan Tower caisson was floated into place July 1903 and the foundation reached "course sand with fine gravel being very firm in character" at -28.2 m in December 1904 (Modjeski, 1909, p. 4). Attempts were made for weeks to force grout into this material which was useless and the pressure of up to 3.2 kg/cm^2 caused the death of several men (Johnson, p. 26). A study of the conditions resulted in the decision to fill the caissons some 6 m above rock (Nichols, 1906).

- Towers

In contrast to the Brooklyn and Williamsburg bridges, which combine relatively rigid towers and sliding saddles, the Manhattan was the first to combine fixed saddles and towers, braced only in the transverse and vertical directions. Moisseiff eliminated G. Lindenthal's pivot at the tower base. Instead of relying on the rollers under the saddles at the towers which were largely ineffective on previous bridges, the slender towers resist elastically the varying longitudinal forces caused by ambient service conditions. Under maximum loading and temperature, the actual towers can accommodate a movement of 61 cm each way from the tower tops. Under ordinary conditions the movement was estimated at less than 15 cm, producing stress in the extreme fiber under 7,258 kg (Perry, 1909). Previously unseen in bridge design were also the cellular spaces within the tower legs, replacing exposed elements (as for example at the Williamsburg Bridge). This design allowed construction of the tower columns without falsework. An ingenious derrick could advance vertically up each leg after each 62 ton section was installed (Steinman, 1929, p. 337). The derrick had a platform supported by two struts; the tip moment was resisted by a pair of wheels engaging the vertical edges on the tower. When the 62 ton full section had been added, blocks were added to the top and falls attached to the derrick platform, by which it then lifted to the next level. In addition to the two stiff-leg derricks, each tower had two hoisting engines, a power plant with air compressors, 30 pneumatic riveting hammers, six forges and a work force of 100 men and six rivet gangs. This system allowed erecting a record 2,000 tons of steel at one tower in 16 working days (Steinman, 1929, p. 165). In order to offset the deformations caused by congested live loads, the towers were pulled 10 cm toward the shores when the cables were completed and prior to placing the dead load (Perry, 1909).

- Cable Spinning

With diameters of 54.2 cm, the four main cables were the largest in the world when spun, and remained so for 17 years. The two 76.5 cm diameter cables on the Ben Franklin Bridge were completed by 1926, but only the four cables of the George Washington Bridge, completed in 1931 had greater carrying capacity. At age 108, Manhattan Bridge still carries the most traffic and has the largest capacity of all six East River suspension bridges. Roebling and Sons made good on their marketing promise that the wires for the largest cables

in the world would also be spun in record time. In the spring of 1908, the contractor was claiming that the cables would be completed within 12 months of stringing the first wire and at "far greater celerity than the Brooklyn and Williamsburg Bridges". Stringing would be done by late spring 1909 and the bridge would be ready to open by the summer of 1911. (Scientific American, 1908). All cable work was performed by Glyndon Contracting (Perry, 1909). Preparations began with four reels of 4.45 cm wire ropes for the footbridge cables towed across the East River by barge with other traffic stopped. The free end of each rope was hauled up by line over the tower tops, placed on temporary saddles, and adjusted with hoist engines at the Brooklyn Anchorage (Perry, 1909, p. 55). The inner and outer cables were braced by working platforms, and hauling rope towers were stationed every 76.25 m. The work platform was stayed against wind vibration with four 4.46 cm storm cables connected to the footbridge at 16.8 m intervals (Perry, 1909, p. 56). Guide wires were adjusted to the designed deflection and slippage in the tower and anchorage saddles prior to loading (Hool, 1943, p, 350). The four hauling wire ropes featured 1.9 cm diameter endless loops with two traveling sheaves. The hauling rope at the Manhattan Anchorage passed around two 0.915 m diameter deflecting wheels and one 1.525 m diameter idler wheel that could adjust the tension (Perry, 1909, p. 57). The machinery for operating the hauling rope was housed at the Brooklyn Anchorage with four 50 W.C.W. motors. The wires for the main cables were delivered in 24,384 m continuous length, wound on reels. Four reels were placed at each anchorage, eight total on the bridge, allowing for 8 traveling sheaves at a time (Hool, 1943, p. 352). Strands were supported at the anchorages and tower saddles by cast iron sheaves bolted temporarily to the saddles on each side of the groove, several inches above the tops of the saddles and 30.5 to 61 cm above their final position (Perry, 1909, p. 56). Movement of the traveling sheave was monitored. A system of electric bells and telephone notified controllers at the break wheels, greatly assisting all operations and adjustments (Perry, 1909). A loop placing two wires was pulled by 91.5 cm diameter traveling sheaves which made the round trip from anchorage at anchorage in 15 minutes. The traveling sheave on the opposite side for each cable also carried a loop, allowing placement of sixteen wires at a time (Steinman, 1929, p. 339). Since the length of each cable is 983.3 m, the eight sheaves were laying wire at a rate of 64.4 km/hr. The 37,224 km of wire, 7% shy of the earth's circumference, were spun in less than 4 ½ months - a record speed which inspired others to pursue more efficient spinning methods. For comparison, the amount of wire spun on the best day at the Brooklyn Bridge was 20 tons and 75 tons at the Williamsburg Bridge. The maximum amount of wire spun in one day on Manhattan Bridge was 130 tons (Steinman, 1929, p. 190). The Manhattan cables required the first hydraulic squeeze rings adaptable for different diameters: for the seven strands in the first stage, and for the entire 37 strands in the second stage. The method was replaced with flat band seizing on later bridges (Steinman, 1929, p. 340). Holton D. Robinson was the Engineer-in-Charge for the Department of Bridges on the Manhattan Bridge in 1905 and worked for the contractor during the wire spinning. He designed and patented the cable wrapping machine. This machine used an electric motor and was self propelled for the first time. The 454 kg wrapping machine used a 1.5 hp electric motor and pressed the wires against the preceding coil at 13 revolutions per minute with two spools at the same time advancing at a rate of 5.5 m per hour (Steinman, 1929, p. 183; Hool, 1943, p. 355). In 1921 Robinson and Steinman started a consulting firm.

Total length of the loaded cable between the pins of the anchor chain is 983.4 m and for the unloaded anchor chains 982.25 m. Thus the extension due to the dead load of the trusses and floor is 1.14 m (Perry, 1909, p. 52). The lengths and dead load forces were computed for parabolic curves. In an early demonstration of sustainable economy, the 4.45 cm footbridge cables were cut and used for the short suspenders (Perry, 1909). Upon galvanization, the cable wires demonstrated outstanding ductility. They could bend cold around a rod 1.5 times their own diameter without signs of a fracture (Perry, 1909, p. 52). For protection from the weather

and to facilitate handling and stringing, the wires were covered with grease during all operations (Perry 1909, p. 66). The wire surfaces retain remnants of an oily coating 105 years later.

- *Stiffening Trusses*

Manhattan was the first suspension bridge to use the lighter Warren truss. Erection proceeded at four separate points, simultaneously working from both directions of the each tower. The first pass was started in March 1909 and connected at mid span a little more than a month later. In it, the lower chords of the truss and floor system were temporarily connected to the suspenders. The truss diagonals were installed on the second pass, followed by the upper decks and transverse bracing. For the trusses, 300 men were employed erecting a record 300 tons per day. (Steinman 1943, p. 181). To achieve proper profile of the travel way and the cables, the chord members of the truss were closed only after the dead load was on the structure and adjustments were made to the suspender lengths. The slender design of the bridge, apart from the deflection theory, is also due to incorporating nickel steel with a working stress of 2,812 kg/cm^2 in the upper and lower truss chords. The working stress of the nickel rivets was 1,406 kg/cm^2 (Scientific American, 1908). In all, 8,100 tons of nickel steel were used for the trusses, and 44,000 tons of steel were used for the entire bridge (Steinman, 1941, p. 367). Not half a dozen men lost their lives during construction (NYT, Dec 5, 1909). The construction cost estimate in 1908 was $US 26 m (Winpenny, 2004, p.18, Scientific American Article) and the structure was completed for $US 32 million.

1.2. Traffic

Surface trolley service started in 1912. Light rapid transit opened in 1915 (Steinman, 1955, p. 2). In 1918 the number of motorized vehicles in New York City surpassed horse drawn vehicles. In July 1922 Manhattan Bridge was closed to horse drawn vehicles which had to cross via the Brooklyn Bridge (Winpenny, 2004, p. 40, NY Times, July 7, 1922, p. 1:5). The bridge became much cleaner. In 1929 NYC Mayor Jimmy Walker bought the franchise and rolling stock of the trolley line operating on the bridge (south side) which was converted for bus use (Winpenny, 2004, p. 43, NY Times, November 10, 1929, p. 19:2). Vehicular traffic increased from 65,000 to 110,000 cars daily (Winpenny 2004 NY Times, April 23, 1931, p. 27:6). Pedestrians were banned from the East River bridges at night starting December 17, 1941 (Winpenny, 2004, NY Times, December 17, 1941, p. 32:2).

2. Life-Cycle Performance

The loads and responses are uniquely dynamic in suspension bridges. In that respect, they resemble mechanical devices more closely than "rigid" structures. At the Manhattan this is especially true, and as a result, the bridge has been a foremost subject of bridge management studies (Birdsall, 1971). The live loads on the bridge are exceptionally high in both frequency and amplitude, and cause a similarly extreme structural response. To begin with, the bridge not only moves with seasonal and daily thermal changes, but can deflect over 0.3 m at center span every time one of the more than 900 daily trains crosses. This change in profile translates to frequent movement in the expansion joints at the towers and differential vertical movement, most pronounced between the inner trusses. Figure 4 shows a 36 cm sliding bearing under a subway track stringer, extended almost beyond its pedestal as a result of regular movements caused by service loads and temperature. Figure 5 illustrates a typical live load configuration, producing the torsional response that has been questioned, investigated and mitigated throughout the life of the bridge.

2.1. Torsion

"For reason unknown, Moisseiff placed the heaviest live loads, the subway trains and elevated-car traffic, outside rather than inside …. resulting in excessive torque…" (Winpenny, 2004, p. ix). The effects of the differential deflection became apparent within 6 years of the bridge opening, as the bracings between the top chords of the inner trusses ("B" and "C") cracked. Originally intended as support for traffic signs, they were removed in 1924. The three approved designs may have contributed to a gap between the originally anticipated and the actual use of the bridge. Complicating this was the fact that the NYC Bridge Commissioner was not in control of the transit facilities as had been the case at the Brooklyn and Williamsburg Bridges. Under the new charter (NYT, Dec 5, 1909), control had passed to the Public Service Commission which succeeded the Rapid Transit Commission. Thus the bridge responded to the very different vehicular and rail modes of transportation with a "split personality", respectively flexural and torsional.



Fig. 4. Over-extended sliding bearing under the subway tracks

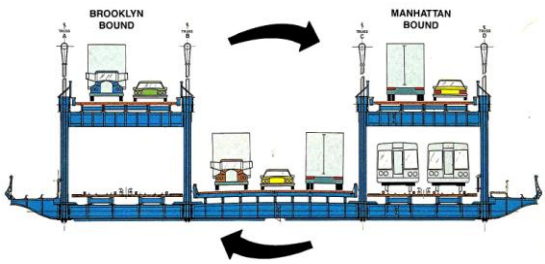


Fig. 5. Typical cross section with asymmetric loading

The original suspenders passed through the upper truss chords without centering devices. As each passage of one or two adjacent trains tilted the bridge to one side, the suspenders moved towards the upper chord webs, engaging in friction mid-span. To improve the clearance and reduce chaffing against the upper chord, the original two part suspenders were replaced with single part suspenders near mid-span in 1937. Measures mitigating the torsional movements were implemented between 1930 and 1940. In 1938, the stiffening truss connections at the towers were changed from rockers to pinned hangers along with expansion joints in the lower deck. Commissioner Zurmuhlen noted in 1952 that the BMT trains placed "a terrific strain on the cables and all structural parts". With 6 cars weighing 40 tons each, a train weighs 240 tons. It was estimated that passengers add up to 44% to that load, or 105 tons of human weight. Thus the total weight of a loaded train was 345 tons. The current trains are composed of 8 R68 cars, weighing 42 tons each, for a total of 336 tons unloaded and 484 tons loaded. Thus the new train loads exceed those of the 1950's by at least 40%. Added passenger capacity may have raised that ratio by as much as 60%. In 1955 the City retained the firm Steinman Boynton, Gronquist & London for an in-depth study of the bridge. As one option,

Steinman's (1955) report recommended relocating the subway train to a new $US 90 million tunnel. This was rejected by Mayor Wagner as financially impractical. The second option - to relocate three tracks to the center for $US 30 million was also never pursued due to operational difficulties. Many of the actions preceding the recent rehabilitation program aimed at reducing the torsional rigidity, under the assumption that no stiffness is better than inadequate stiffness (Birdsall, 1971). Consistently with the original premise of the Deflection Theory, this logic produces excessive flexibility. The 1955 study established that torsion was responsible for cracks in the floor systems. The upper floor systems were replaced with a system supposedly less sensitive to torque during 1959 - 1962. During a subsequent study in 1971, Steinman, Boynton, Gronquist & London tested a two -dimensional model at Columbia University in order to determine the spring constant of the cable truss system, acknowledging that the perfect dimensional model was the bridge itself. The study concluded that side span supports, combined with either transverse stays at the main span quarter points or with tower stays, and four panels of diagonals at the main span centerline would most efficiently and economically reduce deflections and hence, the torque. It was noted that the torsion had not compromised the performance of the primary members, namely the cables, towers and foundations (Birdsall, 1971). Several schemes to reduce the torsion were recommended, including tie cables, diagonals in the main and side span, tower and transverse stays. None of these schemes was adopted. The early 1980's in-depth inspection found widespread and severe deterioration in the floorbeams and stringers under the joints and at the fascias. Track bearings were worn out and laterals were broken. On at least one occasion in 1988 all nine stringers supporting the lower roadway were found broken at the same floorbeam. The central finding of the inspection was distressing: the upper roadways of the suspended spans, floor-beams and stringers, installed in 1962, had cracked extensively. Cracked members were clustered near anchorages, towers, and the center span. Figure 6 shows a typical crack in a stringer, propagating beyond a 2.5 mm hole, drilled in an attempt to arrest it. Figure 7 - A shows a crack through the entire bottom chord of truss "D". A typical crack in the bearing of a stringer supporting the subway tracks is shown in Fig. 7 - B. Advanced corrosion and poor bearing details contributed to an unquantifiable degree to the crack initiation and propagation. New studies of potential stiffening schemes were conducted in two phases. As a foremost constraint, any structural modification had to increase the dead load on the cables and towers within acceptable margins of safety. In the first phase, a two-dimensional computer model analyzed 14 stiffening schemes including those considered in 1955 and 1971. Figures 8. -A, -B, and -C show 11 of them. In 1982 Weidlinger Associates first recommended reducing the differential deflection between the trusses due to eccentric train loading by "twin torque tubes". Three-dimensional models of schemes C1 and C2 (Fig. 8) were studied analytically and validated through results of a load test. The high friction forces in the stringer bearings of the upper roadways were identified as the key factor in causing widespread floorbeam cracking. Failure of the stringers to slide over the floorbeam could magnify the stress caused by the torque eight fold.



Fig. 6. Typical crack in roadway stringer, propagating beyond a drilled hole

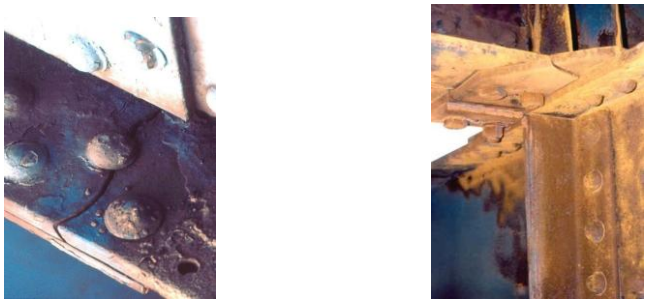
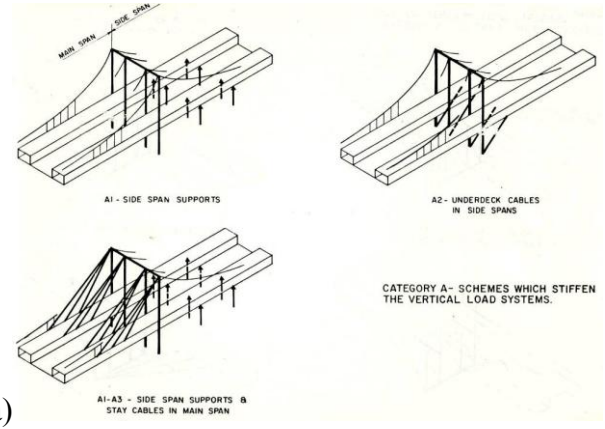


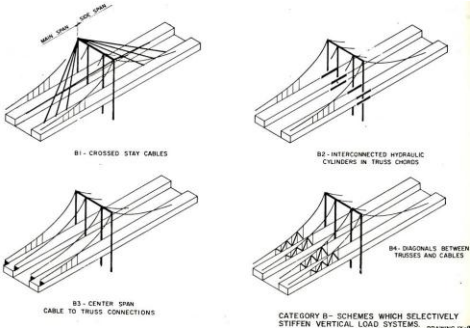
Fig. 7. Cracks in: a) Truss "D", bottom chord; b) Stringer bearing

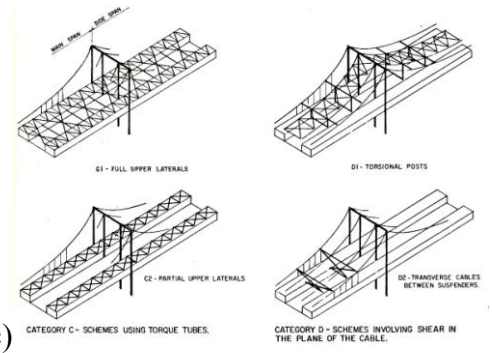
Scheme C2-A was selected. It included stiffening the suspended torque tubes connected by strengthened floorbeams at the lower level and providing eight rigid end frames around the transit envelopes adjacent to the towers for each suspended span, by installing diagonals between the outer and inner trusses of the upper level and diagonals between the outer trusses of the lower level. The partial upper laterals, removed in 1924, were replaced with stronger members. All steel sliding bearings under the stringers were replaced with elastomeric bearings, thus, isolating the stringers from the torque tubes. Along with its structural benefits, the torque tube solution retained the appearance of the bridge. Each torque tube ends with a new rigid frame, as shown in Fig. 9.

a)



b)





c)

Fig. 8. Stiffening schemes

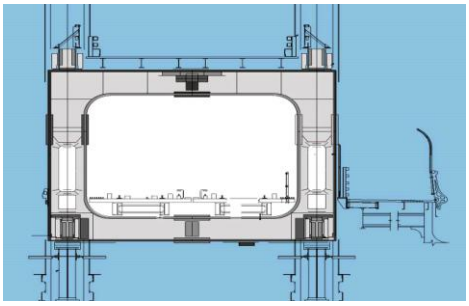


Fig. 9. Rigid frame constraining the end of each torque tube

2.2. The Stiffer Performance

The torque tube and lower roadway stiffening system have performed under actual loadings over a relatively short period. The projected fatigue life of the stiffened structure has yet to be demonstrated. In the meanwhile its dynamic response is measureable. For Steinman's (1971) study, Prof. R. B. Testa of Columbia University measured up to 2.44 m in differential vertical movement between the bridge fascia in the uneven loading case of two trains, either on the north or south tracks, as in (Fig. 5). Since the retrofit, measurements have been obtained by laser tilt-meter mid – span (Fig. 10), by interferometric radar scanner, by Ingegneria Dei Sistemi (IDS), Pisa, Italy (Fig. 11), as a demonstration of the latter system, and by a Global Positioning System (GPS) from the tops of the bridge towers (Fig. 12), as part of the dynamic re-analysis of the bridge for seismic retrofitting (Mayer et al., 2010).

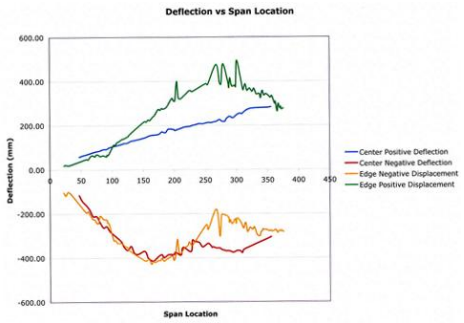


Fig. 10. Laser tilt-meter measurements of deflections vs. span location

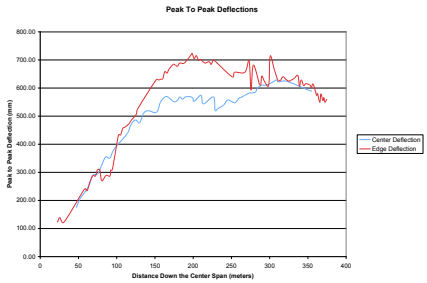


Fig. 11. Results of interferometric radar scan of half the main span

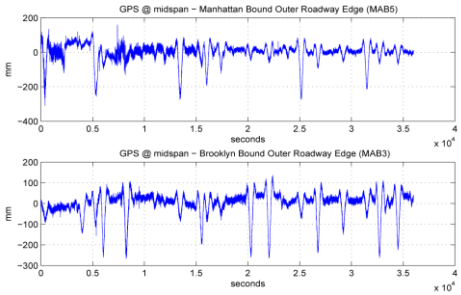


Fig. 12. GPS receiver data showing outer roadway edge deflections

The three methods consistently indicate maximum differential movement of roughly 60 cm between the fascia. Thus, the reduction of the original displacements by 50%, projected in 1981, has been exceeded.

2.3. Rehabilitation / Reconstruction

By 1980, for structurally different reasons, the three great suspension bridges over the East River underwent major rehabilitation / reconstruction through multiple contracts. Between 1981 and 2020 the expenditures for the various capital projects on Brooklyn Bridge will reach $US 900.52 to 970.52 million. The suspenders and stays, the decks and some of the approach spans have been replaced. In 1988 the total replacement of the Williamsburg Bridge was considered. The cost of its rehabilitation / reconstruction (1983 – 2002) eventually amounted to $US 1,086.66 million. At Manhattan Bridge between 1999 and 2009 State and City inspectors issued 691 structural and safety flags for potential structural and safety hazards. Of those 206 were addressed by in-house forces and 485 were routed to various contracts. The following summary describes the scope and cost of the reconstruction / rehabilitation contracts from 1982 to 2018 (NYC DOT, 2016).

REHABILITATION ITEMS — TOTAL ESTIMATED COST (million $US)

- Repair floor beams. (1982) 0.70*
- Replace inspection platforms, subway stringers on approach spans. (1985) 6.30*
- Install truss supports on suspended spans. (1985) 0.50*
- Partial rehabilitation of walkway. (1989) 3.00*
- Rehabilitate truss hangers on east side of bridge. (1989) 0.70*
- Install anti-torsional fix (side spans) and rehabilitate upper roadway decks on approach spans on east side; replace drainage system on approach spans, install new lighting on entire upper roadways east side, including purchase of fabricated material for west side of bridge. (1989) 40.30*
- Eyebar rehabilitation - Manhattan anchorage Chamber "C." (1988) 12.20*

- Replacement of maintenance platform in the suspended span. (1982) 4.27*
- Reconstruct maintenance inspection platforms, including new rail and hanger systems and new electrical and mechanical systems; over 2,000 interim repairs to structural steel support system of lower roadway for future functioning of roadway as a detour during later construction contracts. (1992) 23.50*
- Install anti-torsional fix on west side (main and side spans); west upper roadway decks, replace drainage systems on west suspended and approach spans; walkway rehabilitation (install fencing, new lighting on west upper roadways and walkways); rehabilitate cables in both Brooklyn and Manhattan anchorage chambers; dehumidify Brooklyn and Manhattan anchorages. (1997) 141.82*
- Installation of test panels. (1982) 1.55****
- Removal of existing suspender ropes and sockets in the suspended spans; replacement With new suspender ropes and sockets in the suspended spans and re-tensioning of Suspender ropes bearing plates; re-tensioning of cable band bolts; removal of existing main cable wrapping; cleaning of main cables; new protective paste on main cables; new main cable wrapping; reinforcement of truss verticals and gusset plates. Replacement of necklace lighting and multirotational bearings at truss "C" and "D," installation of access platforms at towers, rehabilitation of south upper roadway lighting. (2010) 159.12*
- Interim Steel Rehabilitation and Painting - cable and saddle repairs lower roadway Floorbeams at PP 37/38 on approaches and at anchorages; west side truss rockers and grillages on approaches; cable and suspender repairs. Removal of parking deck. Painting entire west side, all four cables. (2001) 127.98*
- Stiffening of Main Span; Reconstruction of North Subway framing; reconstruction of North upper roadway deck at suspended spans; rehabilitation of north approach span trusses; replace overlay on north upper roadway approach spans; rehabilitation of North elevated structures and subway tunnels; removal of railing on truss "D" in the North spans; painting of north side of bridge; new inspection platforms and debris protection in approach spans; construction of new north bikeway, replacement of approach span bearings and grillages; installation of Intelligent Vehicle Highway System for North and South Upper Roadways as well as for Lower Roadway (In Progress). 184.78*
- Rehabilitation of Lower Roadway; rehabilitation of anchorage roofs under lower roadway; rehabilitation of substructures and retaining walls in Brooklyn and Manhattan approaches; installation of new signage on bridge and at plaza areas; installation of new lighting on lower roadway and plaza areas; clean and paint lower roadway; installation of grating platform under towers at lower roadway; canopy lighting at towers. (Present) 143.80*
- Structural and component rehabilitation (2018) 0.00 to 120.00***

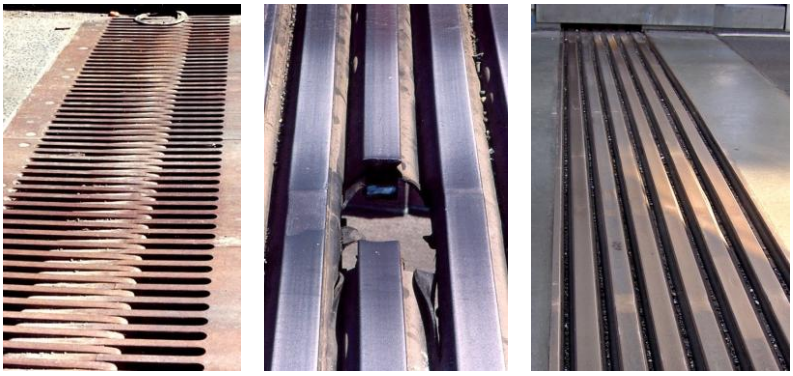
TOTAL: $ 900.52 to 970.52

* Complete; ** In Construction; *** In Design; **** Research and Development (completed)

Major repair and construction of the torque tubes with full service closures were conducted at the south tracks from December 1990 to July 2001 and at the north tracks from July 2001 to February 2004. Figure 13 shows a new stiffening lateral member under the upper roadway. As during the original construction, the slip-critical bolted connections were tightened only after all laterals were installed. Modular joints and finger joints had been alternatively used in the old roadways (Figs. 14. -A, - B). New modular joints are as shown in Fig. 14. - C. A full transition to modular joints is anticipated.



Fig. 13. New lateral bracing under the upper roadway



a) Finger b) Modular (broken spacer bar) c) New Modular

Fig. 14. Expansion joints

The new decks are steel grids (Fig. 15) with concrete overfill. The entire 18,600 m^2 of lower roadway deck, 2,344 stringers and 305 floorbeams were replaced between October 2006 and August 2007, introducing the following innovations: - The work plan developed by the Contractor (Koch Skanska) maximized access to the work zones by providing for delivery of equipment and materials from both the Manhattan and Brooklyn approaches. The construction began with 2 crews at the Manhattan Anchorage with one crew working westerly toward the Manhattan abutment and one crew working easterly toward the Brooklyn abutment. To save time, the existing deck and stringers were removed in panels; - The new stringers were preassembled in groups of two in the shop. To accelerate the erection of the structural steel, floorbeams were delivered with the elastomeric pads pre-installed; - To ensure that the Contractor met the 12 month closure schedule, the contract specification required that 50% of the grid deck (Fig. 15) and structural steel be fabricated prior to closure. To minimize the risk, the Contractor fabricated 100% of the main steel elements prior to the closure. The described measures not only lessened the impact on traffic but improved the quality of the final product and reduced the duration of construction; - Full closure of the lower roadway eliminated the need for construction joints in the grid deck. Concrete placements were made from deck joint to deck joint – no cold joints were required. The grid deck panels run the complete width of the roadway with no need for splicing of the main bars.

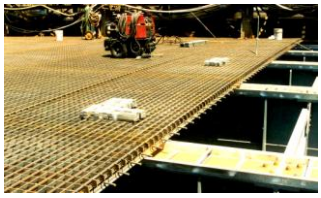


Fig. 15. Lower roadway steel grid deck under construction

- The maximum incentive ($US 65,000 per day for 60 days) resulted in re-opening the lower roadway 60 calendar days early. All East River bridges were repainted with full containment, without traffic interruption.

3. Anchorages

Anchorages and towers are the cable supports and thus must precede their construction. The Manhattan Bridge ones are particularly monumental. With age, they became the center of uniquely innovative rehabilitation measures as well. By the early 1980s, the eyebars anchoring cable "C" in the Manhattan anchorage had lost so much of their cross section to corrosion, that their elastic elongation was beginning to affect the tension of the strands. Weidlinger Associates designed the re-anchoring of the cable, shown in Fig. 16 and Karl Koch Construction were the contractor (Mayrbaurl and Good, 1988).

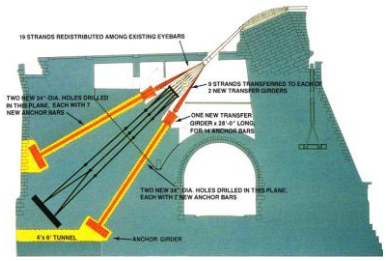


Fig. 16. Manhattan Anchorage, Manhattan Bridge

Two new transfer girders were anchored into the monolith by two drilled shafts, post-tensioned and grouted. Nine strands were transferred to each girder (Fig. 17). The sockets were filled with molten zinc. On a smaller scale, strands were subsequently re-anchored in the Brooklyn anchorage, using polyester raisin for the sockets. The anchorages are now actively dehumidified.



Fig. 17. Nine transferred strands, cable "C", Manhattan anchorage

4. Cables

Each of the 4 Manhattan Bridge cables comprises 9,462 parallel high-strength galvanized wires with overall diameter of 55.2 cm (21.7 in). Given a wire diameter of 5 mm (0.198 in), the void ratio of the cable is close to the typical, if not optimal 20%. Figure 18 - A. shows a portion of the cable, unwrapped and wedged for inspection. The cable wires are considered in good condition. The red pigment is a remnant of the original lead paste. None was applied during the current rehabilitation. The original grease coating still covers the zinc surface of the wires. Figure 18 - B. shows a test of the magnetic flux method developed by Tokyo Rope for estimating the amount of steel in the cable without unwrapping.



Fig. 18. – a) Unwrapped wedged cable; - b) Testing of cable "D" by magnetic flux

As the age of the parallel wire suspension cables surpasses a century, their life-cycles enter unfamiliar territory. In 1988 the need to estimate the remaining strength of the Williamsburg bridge cables was particularly urgent. That task intensified the research and development of methods for suspension cable inspection, evaluation and preservation. The four suspension bridge owners in the New York City Metropolitan area (NYC DOT, NY State Bridge Authority, Port Authority of New York & New Jersey, Metropolitan Transit Authority) commissioned a joint report on the condition of the cables at their 10 bridges from Columbia University (Bieniek and Betti, 1998). The Transportation Research Board (TRB) funded NCHRP Report 534 by Mayrbaurl and Camo (2004) on the evaluation of cable strength. The Federal Highway Administration (FHWA) followed with the report on field cable strength evaluation (Chavel and Leshko, 2009). The need to unwrap cables periodically, wedge them at numerous locations along their length and circumference and examine the conditions of the wires was emphasized. Guidelines for statistical evaluation of the limited findings were recommended. Concurrently, the non-invasive monitoring of cable wire condition by means of new technologies for data acquisition and transmittal has become a potentially cost-effective alternative to unwrapping and wedging. Through an Act of Congress, FHWA funded an investigation of that possibility with particular emphasis on the East River bridges, as well as for general application. Many sensing technologies were tested on a 7 m, 10 000 wire model at the Carlton Laboratory, Columbia University. The most promising ones were field-tested on Manhattan Bridge in 2012. The locations of the implanted sensors are shown in Fig. 19. The cable model is shown in Fig. 20. The results are reported in FHWA-HRT-14-024 (Betti et al. 2014). The sensors installed with the assistance of NYC DOT at Manhattan Bridge monitored temperature, humidity, wind velocity, vibrations, and the rate of corrosion. A clear correlation was demonstrated between ambient temperature and internal humidity. That finding may explain why certain locations on a cable are more corrosion-prone than others. While non-invasive monitoring is explored in this manner, the Honshu – Shikoku Bridge Authority (HSBA) developed cable dehumidification by injection of dry air. The method precludes corrosion by maintaining humidity below 40%. The first applications were at the (then) new Kurushima and Akashi – Kaikiyo bridges. The method is now contemplated for older structures, where linseed oil, led and zinc paste, as well as other active and passive corrosion inhibitors have been applied since the time of construction. The rate of air penetration under such conditions was investigated on the cable model at Columbia University (Fig. 20).

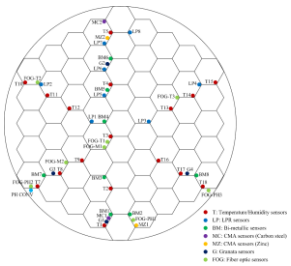


Fig. 19. Location of various sensors the cable model



Fig. 20. Cable model, Carlton Laboratory, Columbia University

5. Suspenders

Cables were rewrapped and suspenders replaced at the Brooklyn Bridge in the early 1980s, and at the Williamsburg Bridge in the early1990s. The Manhattan Bridge suspenders dated back to 1956. They were installed for $US 2.2 million by Roebling & Sons Bridge Division in one of their last contracts, before closing in 1964 (Zink et al., 1992). The contract included an attempt to determine the dead load stresses in the suspenders. By the mid-1990 suspenders were exhibiting wire breaks due to corrosion near the sockets at the level of the bottom truss chord (Fig. 21) and due to friction with the top chord. The latest contract to rehabilitate the main cables, install new wire wrapping, neoprene barrier, hand ropes, and replace the 1256 individual suspender ropes at the 628 suspender panel points was bid for $US 149.5 million. It was executed between 2009 and 2013 by Koch Skanska for $US 159.12 million. The loading of the suspenders was measured by jacking prior to removal in order to calibrate current as-built loading of the main bridge. As shown in Fig. 22, the new suspenders represent a significant innovation. With the exception of the portions mid-span and near the anchorages, they are anchored at the top chords of the trusses, rather than (as originally) at the bottom. Due to the unique aspects of this new configuration, much attention was given to proper distribution of the loading on the suspenders. Following the initial calibration with hydraulic jacks and pressure gauges, frequency measurements were taken in the longitudinal and transverse direction at the equalizer bars of the new suspender assemblies with tri-axial accelerometers. The method considerably accelerated the project. The consultants (Weidlinger Associates and Parsons Transportation Group) also considered checking the new suspender loads using a "laser load" technique. The geometry layout of the bridge and the requirement for only off-peak upper roadway lane outages dictated the replacements of the suspenders along each of the four cables before proceeding to the next.



Fig. 21. Wire breaks in suspender at the bottom socket

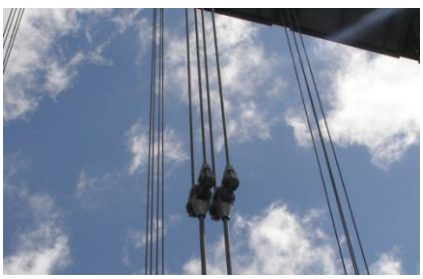


Fig. 22. Old and new suspenders

6. Lifecycle Management

Manhattan and Williamsburg provide the only mutual re-routing alternatives for truck traffic between Brooklyn, Manhattan and New Jersey. The capacity of the Brooklyn Bridge is limited to passenger and emergency vehicles. Hence, the concurrent rehabilitations had to be coordinated for minimum service interruptions. The very choice of rehabilitating, rather than replacing the Williamsburg Bridge was influenced by the demand for uninterrupted traffic. Staging the rehabilitations of the East River bridges over the same 30 years elevated their costs to roughly $US 1 billion per bridge. These expenditures, and the incalculable user costs, incurred during the inevitable traffic interruptions, underscored the need for cost-effective management of such unique assets over their entire lifecycles. It was recognized that, while any material structure and all of its modifications over time have finite useful life, the service provided by an essential bridge permanently changes the local geography and must have a perpetual lifecycle. Similarly perpetual is the need for structural maintenance and preservation, since, by the conclusion of decade-long projects, new needs have already arisen. The experience at the East River bridges during the 1980s and 90s contributed to the interest in Lifecycle Bridge Management (LCBM) on the federal to local management levels. Resulting bridge management developments include the following: - The Federal Highway Administration (FHWA) recognized expenditures for major maintenance, such as repainting as eligible for federal funding; - New York City Department of Transportation (NYC DOT) commissioned a Preventive Maintenance Report for the bridges in its purview (Bieniek et al., 1989, 1999). As a follow-up, detailed maintenance manuals were compiled for all East River bridges. Tasks, such as oiling of wires in the anchorages, repairs of wrapping, painting, spot-painting, maintenance of travelers, cleaning of joints, and so on, were scoped, scheduled and budgeted; - On the East River Bridges and their approaches, de-icing with salt by the City Department of Sanitation was replaced by anti-icing with potassium acetate by NYC DOT. In a significant change of policy, FHWA treats this costlier activity as eligible for federal funding; - Estimated direct and user lifecycle costs are gaining in importance over first direct costs (e.g. construction costs) as criteria in design selection. For example, the re-anchoring of the Manhattan Bridge suspenders to the upper truss chords was motivated primarily by maintenance considerations. All designs for rehabilitation and replacement of structural elements must specify the expected useful life and supply maintenance instructions; - The biennial inspection of 2016 reported advanced corrosion in the decorative tower ornaments (up to 64 per tower, some weighing hundreds of kilograms, shown in Figure 23 – a, - b, and c). Because of their landmark status, the ornaments will be restored during a future contract, however in-house Bridge Maintenance had to remove them temporarily in a costly emergency operation. Apart from emphasizing the risks and costs associated with non-functional elements on bridges, the incident illustrated another useful lesson. The "rosette" shown in Figure 23 – b, and – c is supported by 8 bolts with d_1 = 12.5 mm, total cross section area A_1 = 981.25 mm^2, and total circumference L_1 = 314 mm. 3 bolts with diameter d_2 = 20.4 mm, and total circumference L_2 = 190 mm would provide area $A_2 = A_1$. Traditional strength design might opt for the 8 bolts with diameter d_1 because of their greater redundancy and more uniform load distribution. However, their larger circumference and smaller individual area render them more susceptible to corrosion. Life-cycle resistance to loads and to corrosion must satisfy sometimes contradictory constrain

a)  b)  c) 

Fig. 23 a, b, c. Ornamental rosettes on the towers

6.1. Ownership and Landmark Status

Although the Manhattan Bridge is currently considered a City Street, both New York City and State are currently looking for ways to collect revenue from the Manhattan Bridge which is busiest trucking routes for the 7.7 million people living in Long Island (third most populous island in the western hemisphere). Tolling of the East River bridges is repeatedly considered, most recently (2017) by the New York State Governor, as a source of funding for the New York City Transit Authority. So far it has been rejected by voters. The Manhattan Bridge is an ASCE National Engineering Landmark. The City Landmarks Commission considers the bridge a City landmark and work is underway to designate it as such (NYC Landmarks Preservation Commission, 2009).

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RESCUING OF ST. CYRIL AND METHODIUS CATHEDRAL IN BURGAS – FROM STRATEGY TO REALITY

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***Abstract**: The main purpose of the presented paper is to introduce and follow the execution of the strengthening strategy for saving of the St. Cyril and Methodius Cathedral in Burgas (Bulgaria). The church has suffered severe damages caused by uneven settlement due to an excavation nearby. It has been retrofitted in such a way that currently it is capable of bearing gravity and seismic loads. The strengthening measures for rescuing have included the following: injection of the cracks with slightly expanding self-compacted mortar, applying of CFRP in the vulnerable zones (domes and vaults), pre-stressing of the structure in closed contours on two levels, implementation of ties and strengthening of the foundation by application of jet grouting. Finally a monitoring system for measuring the water level as well as the horizontal movement and tilting of the cathedral was installed. At present the St. Cyril and Methodius Cathedral operates normally and is one of the most noticeable sights for tourists and citizens of Burgas.*

***Key words**: retrofitting, strengthening, CFRP, pre-stressing, jet grouting, geotechnical monitoring*

1. Introduction

The construction of St. Cyril and Methodius Cathedral in Burgas started on 15.08.1895. The architectural design was provided by the Italian architect Ricardo Toscana. The building was completed as we know it in the 1940s when the two towers were topped. The church is a three nave basilica approximately 32 meters long, 21 meters wide and has an area of 516 m^2. The height of the main dome above ground is around 25 meters. The structure comprises of load bearing brick and stone masonry walls and stone columns, supporting brick masonry arches, barrel vaults and domes. The main dome with 8.4 m diameter is a timber structure developed over an immense brick masonry tambour.

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The whole structure is supported by single and strip stone foundations. Since 1979 the church is announced as a national heritage architectural monument.

In 2011 construction of a new underground parking started immediately next to the church, with only 6 m. between the anchored secant pile retaining wall and the north bell tower. Due to the large horizontal displacements of the wall the soil under the church experienced considerable deformations, thus subjecting the structure of the church to significant uneven settlements. By the time the final excavation elevation of 15 m. was reached, the church has suffered severe damage. At this stage measures for emergency support of the structure were implemented and full geodetic, geologic and structural survey were carried out. A view of the church and the unfinished parking is shown in Fig. 1.



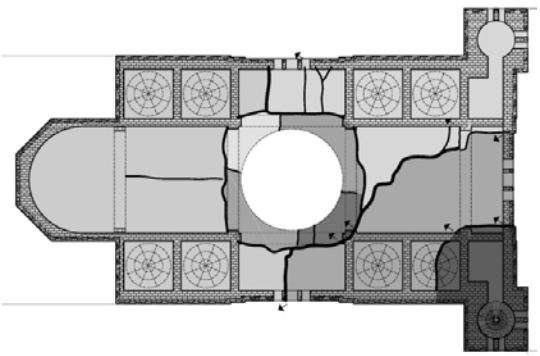
Fig. 1. A view of the church and the adjacent excavation with the secant pile anchored retaining wall

2. Survey results

An important part of the strengthening design was the initial gathering of information about both the structure and the reasons for its condition. The scope of the survey followed the guidelines given in Eurocode 8, part 3.

2.1. Structural survey

The main task of the in situ measurements was to identify the structural system and its load bearing members. The crack distribution was thoroughly studied in order to produce a detailed picture of the damaged building and a summary is presented in Fig. 2. Photos of some of the biggest cracks are shown in Fig. 3. The strength parameters of the masonry materials were obtained by laboratory and in situ tests, as shown in Fig. 4. More details about the survey could be found in [3].



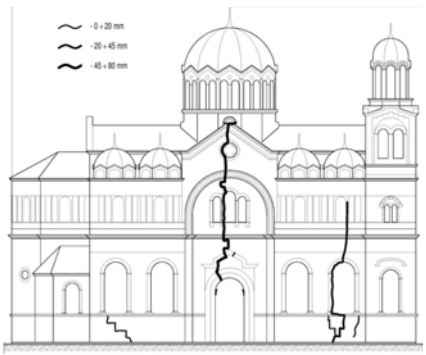


Fig. 2. Crack distribution in plan and on the north façade



a) b)

Fig. 3. Cracks wide up to 80 mm in the a) western barrel vault; b) north façade wall



Fig. 4. Test of the materials a) in situ test of masonry compressive strength; b) stone adhesion laboratory test

2.2. Geotechnical investigation

Seven boreholes with depth of 20 m. were studied. Dynamic penetration tests were performed in all of them. Plan of the boreholes and DPTs is shown in Fig. 5. One of the main conclusions was the considerable change of the underground water level because of the dewatering activities during the construction of the parking. That led to change of the soil deformation and strength parameters. Additionally, it was found, that a slip surface had occurred in the borehole closest to the excavation.

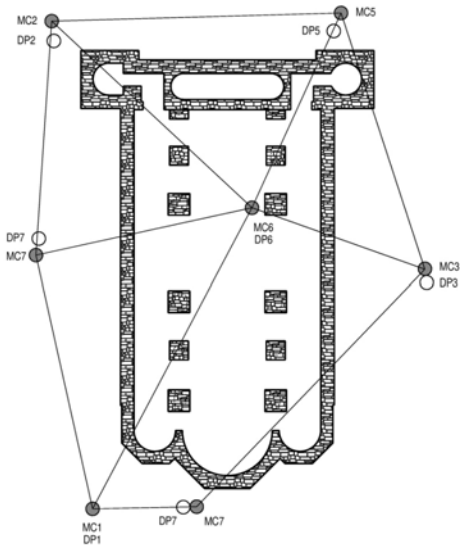


Fig. 5. Geotechnical survey boreholes and dynamic penetrations

2.3. Geodetic measurements

The results of the measurements that took place monthly for over a year showed that the cap of the secant pile wall moved about 200 mm. This led to major settlements and horizontal displacements of up to 16 mm of the north bell tower, inducing critical damage in the load bearing structure.

3. Strengthening strategy

3.1. General description

The presented bellow strengthening approach is proposed in the design project for repairing and strengthening of the church. All design examples given in [2] were studied before selection of strengthening strategy. The main activities of this approach are summarized as follows:

- Strengthening of the soil and foundation;
- Repairing of the cracks;
- Strengthening of the brick masonry barrel vaults and domes by CFRP;
- Strengthening of all arches by ties;
- Pre-stressing of masonry walls on several levels;

Finally, a system for monitoring the strengthened structure was developed and executed.

Each of the listed above measures is briefly described.

3.2. Strengthening of the soil and foundation

The underlying soil base was strengthened by execution of soil-cement "jet-grouting" columns, as shown on Fig. 6. This technique was selected because of the severely damaged soil beneath the church and also because it posed no threat for the anchors of the excavation, as well as the micropiles, previously executed as an emergency strengthening measure. During the execution, due to the damaged soil base the grouting parameters, such as pressure, speed and water-cement ratio had to be adjusted several times in order to achieve proper diameter and strength of the soil-cement columns. A specialized small portable rig was used for execution of the inner rows.

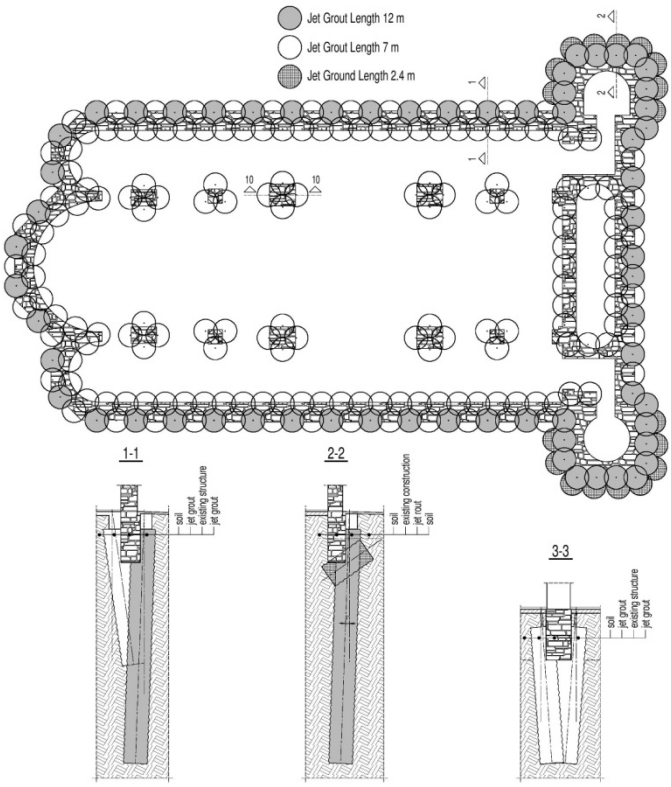


Fig. 6. Strengthening of the soil base by Jet Grouting

The connection between the existing foundation and the jet-grouting elements is accomplished by execution of new RC foundation beams (in Fig. 7), which are continuously anchored to the stone strip foundations by means of steel rods. Additionally, steel I profiles are embedded for better connection between the RC beams and jet-grouting columns.



Fig. 7. New foundation beams with prestressing

3.3. Repair of the cracks

The cracks in the structural members were repaired by injection of different types of expanding grout, depending on their width. Additionally, as shown in Fig. 8, the cracks in the walls were "sewn" with CFRP rods, bonded to the structure with epoxy resin. Those measures are aimed to reestablish the integrity of the structural elements and the connection between them. Photos during execution are presented in Fig. 9.

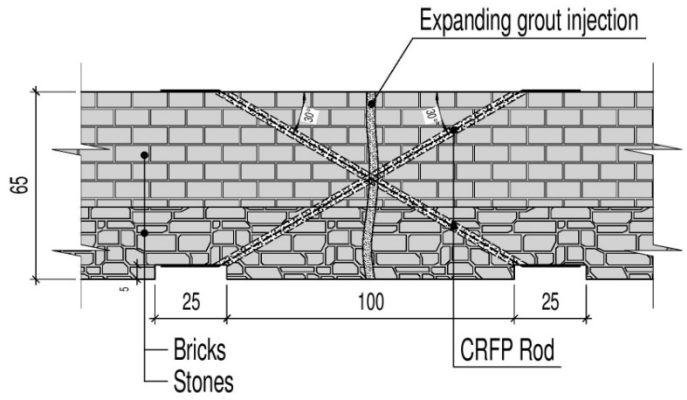


Fig. 8. Repair and strengthening of cracked façade wall – detail



Fig. 9. Repair and strengthening of cracked façade wall – execution

3.4. Strengthening of the brick masonry barrel vaults, domes and arches by CFRP

Externally bonded CFRP is used for strengthening of all damaged brick masonry barrel vaults, domes and arches (Fig. 10 and Fig. 11). All calculation are with accordance with Eurocode 8 part 3 and [1]. The extrados concept is selected for CFRP strengthening of barrel vaults because of art painting on the intrados side. The same concept is used for the brick domes. This method was chosen due to the following benefits:

- No weight is added to the structure;
- It allows for the iconography on the inner side to be kept;
- The CFRP acts as a tension reinforcement and prevents collapse mechanisms, in case of future support movement;

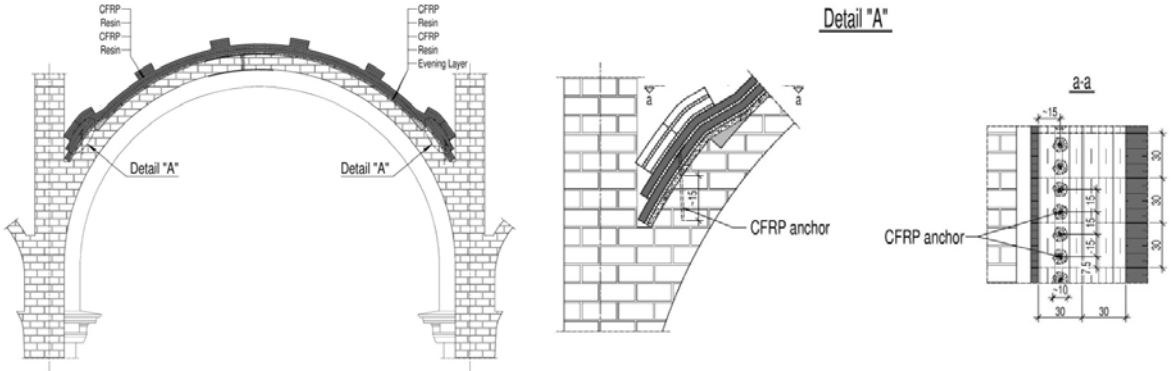


Fig. 10. Strengthening of brick masonry barrel vault with CFRP – design



Fig. 11. Strengthened barrel vault

3.5. Strengthening of all arches with ties

New tie elements are introduced at the base of the masonry arches at two levels and in two orthogonal directions (Fig. 12). The main purpose is to ensure the global stability of the structural system both for vertical and seismic loads. They are connected to the existing masonry structure in a way that reduces the stress concentration and prevents local disintegration of the masonry due to cyclic dynamic reverse excitations occurring during an earthquake. The tie elements prevent the formation of collapse mechanism in the arches, in case of horizontal displacements in the column tops or foundation settlements. View with the executed tie elements is shown in Fig. 13.

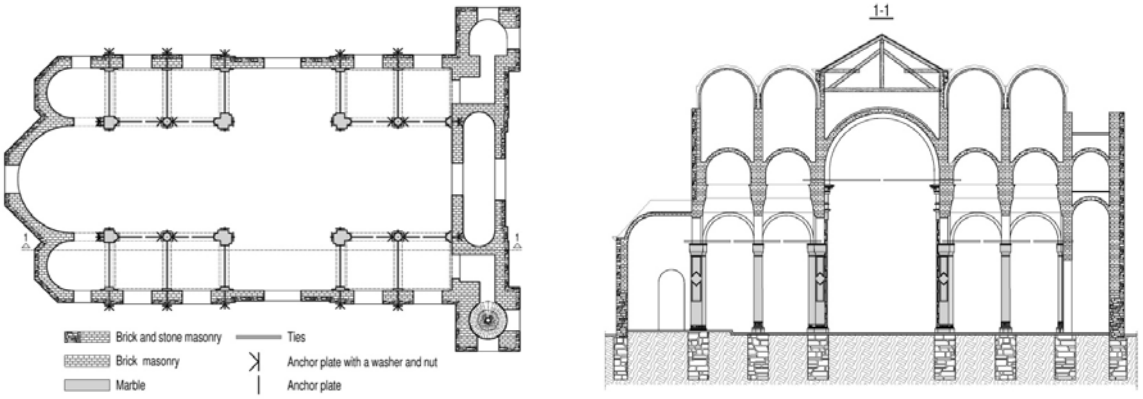


Fig. 12. Plan and section of the added tie elements – design



Fig. 13. View from inside with the added tie elements

3.6. Prestressing on several levels

Three closed contours of prestressed elements were introduced at different levels, binding together the separate parts of the structure and ensuring its uniform behaviour under seismic loading. Due to the geometric imperfections of the existing structure the paths of the tendons had to be further modified during execution. Initially chosen bars had to be substituted with high carbon steel strands which allow for a more flexible positioning and simpler anchoring devices (Fig. 15).

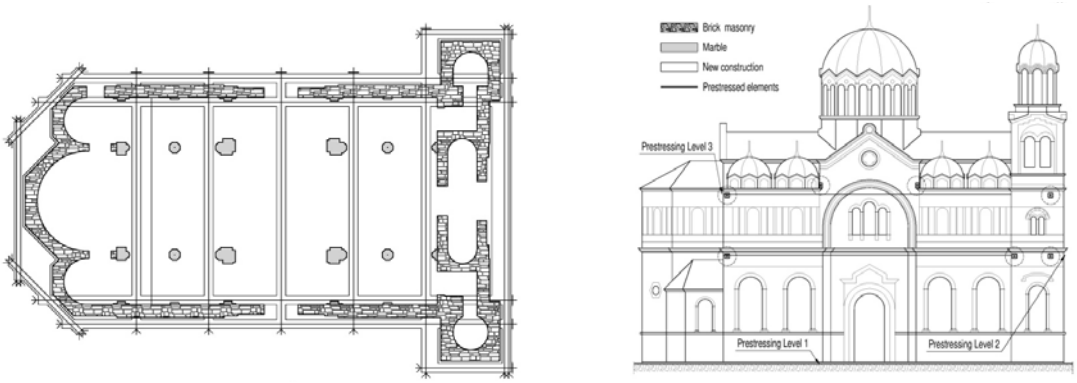


Fig. 14. Plan and elevation of the prestressing levels



Fig. 15. Prestressing at Level 2

4. System for geotechnical monitoring

The monitoring system is implemented so that the behaviour of the strengthened structure and its foundation can be evaluated and analysed. For acquiring sufficient volume and types of relevant data the following instruments were installed:

- Digital tiltmeters were placed on the structure for monitoring horizontal displacements resulting from possible uneven settlements (Fig. 16 a));
- Digital biaxial inclinometers were used for following the in-depth horizontal displacements of the soil underneath the church (Fig. 16 b));
- Digital piezometers were installed in boreholes for monitoring the ground water table and the pore pressure (Fig. 16 c));

The instruments are wired in such a way, that allows them to be connected to a single multichannel data logger (in Fig. 17). It can be accessed remotely via Internet and data can be read in real time.



a) b) c)

Fig. 16. Installation of instruments for geotechnical monitoring a) tiltmeter; b) inclinometer; c) piezometer



Fig. 17. Installed and connected multichannel data logger

4. Conclusion

Based on the practical study presented herein the following main conclusions could be drawn:

- The church St. St. Cyril and Methodius in Burgas which is a part of Bulgarian national heritage was severely damaged during the construction of the new underground parking nearby and needed urgent retrofit;

- The strengthening strategy was developed and executed on the basis of performed survey;
- This strengthening approach aimed to solve all structural problems of the temple for both vertical loading and seismic excitation;
- Some of the technical solutions had to be adapted to better fit the actual execution;
- Constant interaction between the designers and the contractors was required in order to adjust design and technical solutions.

Acknowledgement

Our team of "Yoda" Ltd. would like to express deep gratitude to our "sensei" – Prof. Jordan Milev who is the main reason for the success of the presented complicated project. We lost Jordan too soon – sooner than we all wanted to. We believe that the rescue of this temple would guide the soul of this magnificent person to heaven. Sensei, you will always be missed!

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RECOVERY AND RECONSTRUCTION OF THE CHURCH SV. ELIJAH IN VILAGE SIRČA NEAR KRALJEVO, DAMAGED BY AN EARTHQUAKE IN NOVEMBER OF 2010

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Abstract: *Church St. Elias in Sirča is built nave building length 15.50 m1, m1 width by 8.80 height to the top of the wreath is 6.65 m1, and the entrance wall is 11.70 m1. Since the earthquake in November 2010. have created significant damage to walls, ceilings, final facade ornaments, the main arches and arches not be used. Made the project of rehabilitation and reconstruction of the building: Strengthening existing spherical vault, making a new wreath - reinforced concrete ring beam at the top of the facade walls and the wall of the side arches, metal ties on the altar apse, choir lateral, longitudinal facades and wall where the entrance and rehabilitation and injection of cracks at the very structure. Works on rehabilitation and reconstruction were executed during 2012 and the church was in usable entire time during repairs.*

Key words: *recovery, vaults, arches, wreaths, cracks*

1. Introduction

Church St. Elias in Sirča was significantly devastated in earthquake which hit this holy place at November 2010 yr. and it's closed. Rehabilitation and reconstruction design has been done according to structure preview at 30 of April 2011 year and archi sketches. Institute for the Culture Monuments Protection city of Kraljevo, and existing condition record right after earthquake.

Church was built as nave structure base length, with three ornamental brick work areas divided by pilasters and half circle altar apse and choir lateral. It's length is 15.50 m1, m1 width 7.10m1 and at 8.80 of choir lateral. Height to the top of the wreath is 6.65 m1, and west the entrance wall is 11.70 m1. In church there are two pairs of pilasters with arches whose together with arches leaning by the facade longitude walls caries round dome. Wooden roof construction is with curved web members supported on cross arches between pilasters and at external facade walls. Roof cover is done as a sheet metal over wooden base. It was built at 1876, and roof was renovated 2010 right before earthquake strikes.

Church was crafted by bricks in lime mortar, with facade walls whose width vary from 0.70 m1 do 1.10 m1, with cross pilasters 0.65 m1 wide with cross arches, Ornaments

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at the top of façade walls are made of bricks. It is supposition that church is based on foundation made of bricks or made of stone in mortar and for now without visible cracks.

2. Condition at the moment

Structure is with significant damages at the very walls, arches, done facade ornaments, main arches, and window arches(Fig.1). At church were done some repairing works recently: existing façade has been redone over existing with film over existing, without removing old mortar ; at existing handmade roofing construction are added timber beams 5/8cm to reinforce and correct the flaw of chords to provide easier sub layer of roof and roofing cover is brand new. All this works are done before earthquake, and there is no technical or photo documentation of done work to be seen what and how it was done.

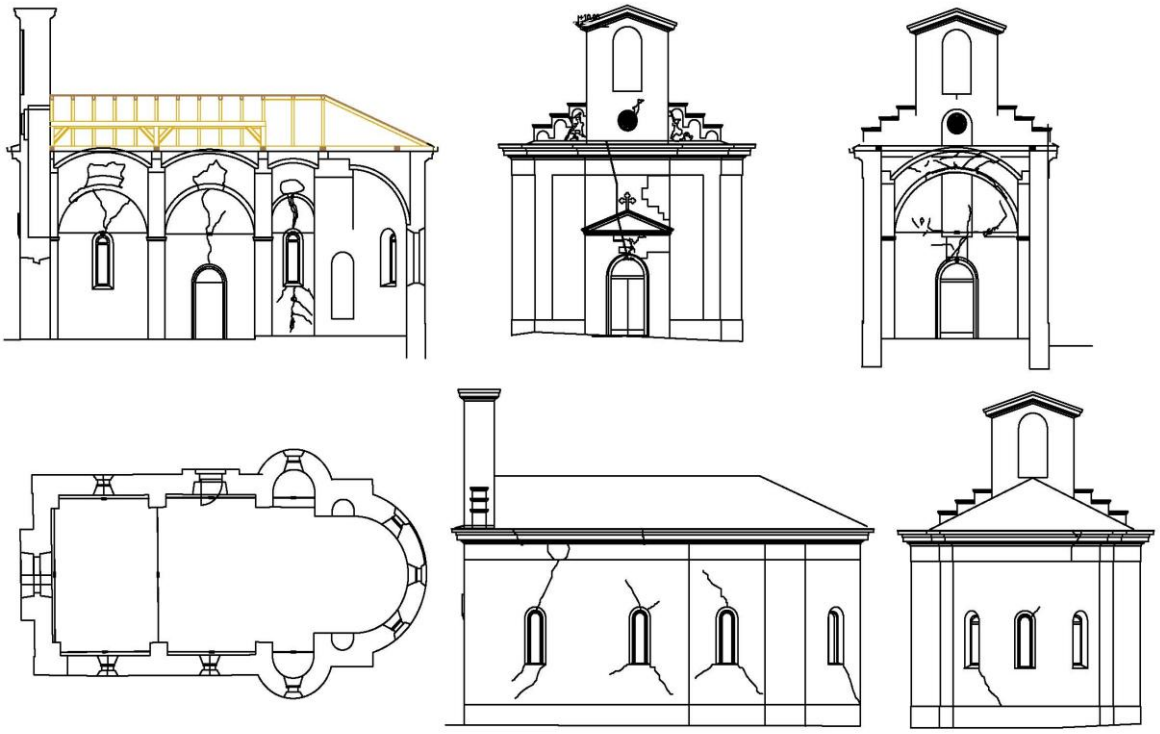


Fig. 1. Schematic view of the church with disabilities

The most vulnerable is part between entrance to church and first pair of pilasters, where ,,Czech" shallow round arch attend to fall because it contains great deformations, and in leaning arches by the façade walls top bricks are facing outside and some of them have fallout because construction disconnection in dome level such ass disconnection of final ornaments into one unit. Damages of this scale hit the surface because stability of dome has been disturbed because of forming hole at dome by the chimney of candle area, and leaving that same hole open after removing chimney from the church which additionally disordered construction stability of dome; non-existing connection between domes and non-existence concrete connection beam in arch and dome level and very first dome has the lowest rigidness because it is first in a row and single leaning arch by the west wall is rigid, and it isn't connected with very west wall because there is hatch in a west wall for passage at the ,,attic", i.e (Fig.a-g). Area under the roof construction. All this damages and moving affect at complete non-stability of object, disabling building usage

and it is necessary to do sanation and reconstruction of the mentioned. Facility is measured to detail, and static calculation of wooden construction is done, in program package Tower 6 has done model of object and analysed.

Impacts from constant load and influence of seismic force at VIII seismic grade, control has been done of seismic stability of structure and calculation of load which apply on foundations of entire structure. Based on everything, there are foreseen necessary measures of straightening construction.

a)  b)  c)  d) 

e  f)  g) 

Fig. 2a-g. Photos of damage to the church of the earthquake

3. Measures of construction strengthening

Removing existing roof construction and bringing it back with necessary straightening and replacements upon necessary domes and crafting of new wreath made of reinforced concrete[1].

Through static calculation roof construction it is being realizing that (on site, ei.) Existing chords with big deflections and it is needed to be replaced with a new ones)(Fig.3).

Existing roof construction is old hand crafted with, 2 by 4"on the top chords because of deflection. Construction, except chords which have big deflection, because of dimensions are not enough spans, at relatively good condition, and it is necessary to bring back the old one after interventions, except chords which has to be replaced with new beams with 14/18cm dimensions.

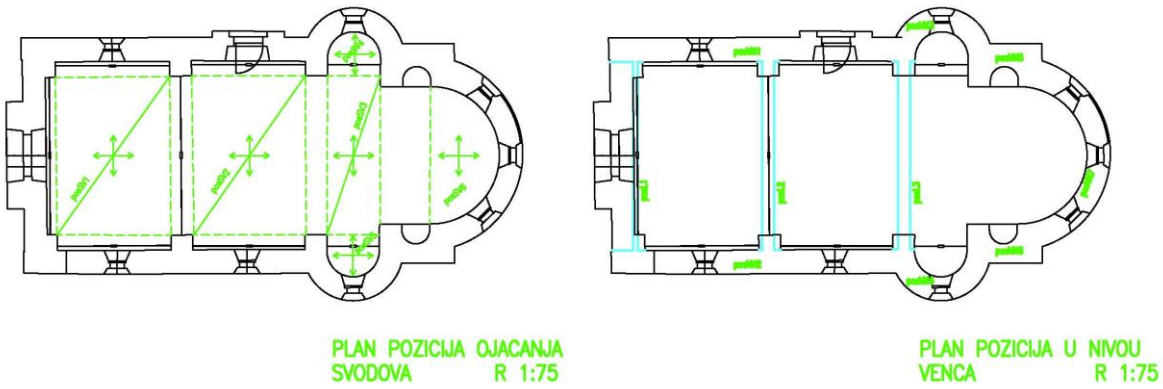


Fig.3. Static scheme position reinforced arches and wreaths

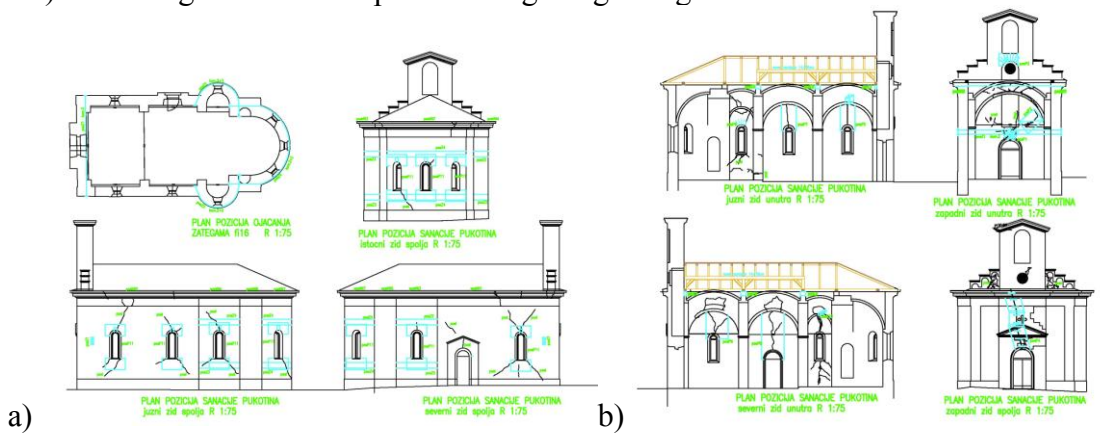
Straightening of existing sfaire domes above the church:

Existing sfaire domes are crafted with full brick 15cm thick at max span at 4,80 m1. Because of disconnection and new showing deformations made at the time of earthquake impact it is necessary to do additional church dome stiffening. It can be done, only from above and requires executing removing of existing roof construction. First dome facing to the entrance between west church wall and first arch between pilasters has to completely redo a brick work, because some parts of dome are moved from originally level of dome. All of this will require development of formwork and supporters under domes.

Domes stiffening are designed by making light reinforced concrete shell (position SV) from upper side with 5cm thickness reinforced with ø8/10mm bars which carry in both directions, and do concreting with small fine-grained aggregate Concrete Grade 30 (MB30). With grains max volume up to 16mm. During construction taking care of protection layer of concrete. It's necessary to previously remove mortar from upper side, do a contacts cleaning from lose mortar and deepen to prepare it for a better connection of new reinforced concrete shell and existing shell made of bricks. To make a better connection with external walls i.e. Arches close to dome end is doings as reinforced concrete beam which anchoring for domes and it's done with min 4Rø10, anchored with Rø14/80cm into arches on pilasters and blind arches by facade walls. Anchor bars needs to be placed into in front prepared holes in which are anchoring liquid poured. Uzengines of beams are designed to bee overlapped with dome bars min 50cm. In graphic documentation is given reinforcement plan which will be harmonized with real condition at the very object.

Make of new Reinforced Concrete ornaments-beams at the top of facade walls and beams at side wall arches: Existing ornaments at the top of facade walls have the most of damage and deformed, so that on a south facade wreath aments is totally devastated – fallout(Fig.4a,b). It is necessary for existing ornament made of bricks completely replace with new reinforced concrete beams (position VN) at three sides of church, except at west entrance wall where will existing ornament be singly enhanced and sanation cracks. New reinforced concrete ornaments are designed with height of 26cm, so that follow existing profilation of ornament from outer side and by need increase of decrease it to make it look like it's following existing and facade is not changing a bit. Beam need to reinforce with 10Rø14mm and with two uzengines Uø8/20cm which are overlapping at middle of a beam. Uzengines overlapping is designed so that width which vary on this way compensate so that beam can do stiffening at the middle because of its great length. Beam need to be anchored into facade wall with Rø14/40 placed into zig - zag lining. During placing of

anchoring hole need to be pre-cleaned, blow out and fill with anchoring liquid by instruction from factory. Reinforced concrete ornament need to be anchored with Rø14 for both corners of west entrance walls. On existing walls of side arches do a reinforce concrete beams (position L), for additional stiffen, width the same as wall 35cm (15cm) with height so as ornament (wreath) 26cm.Do a beams reinforcement with +-3Rø14 (+-2Rø14) and uzengines Uø14/50 placed in zig - zag lining**.**



a) b)

Fig.4a,b. Static scheme ties and repair cracks: a) outside; b) inside

Metal clamps at altar apse; side place for choir, longitudinal facades and entrance wall:

It is predicted constructing (position Z) two at each upper sides and two from bottom side of a window because there are very large damages at the dome above church doors and windows, and there are expressed cracks on a slope under the window. It is necessary extend horizontal connections for metal clamp to pass from inside or outside prepare bases for half inch wider for anchor tiles 200x200mm made of metal sheet 15mm thick.

Clamps are designed from reinforced metal bars ø16mm (GA240/360). Area around clamps is necessary to be injected in order to make wall „work" with clamp at the same time.

Bearings and holes fill out with cement mortar so that additionally protect clamp with that layer of protection and save it from corrosion and provide correct laying at wall surface.

Rehabilitation and injection of cracks on construction:

Now there are visible cracks above and around windows, cracks at arches laid up to façade walls, at the arch above entrance doors and arches at the choir spots. For now it's not possible to spot the cracks which are going all the way down to the foundations and if after removing completely façade and collie from foundations (which will probably be removed to provide drying walls from moisture) show that exist, than their rehabilitation had to be done as well. Cracks (position P) on carrying arches on whose are missing parts of entire bricks in top of arch has to be filled with bar basket and concreting with fine grain concrete MB 3, with necessary frame work and supporters following details from project design. At areas where cracks are larger than 10mm on vertical walls, do straightening with reinforce net bars Q131 with anchors Rø10/40 in both directions and spray with cement mortar. All bigger cracks at arches straightening with star bars Rø10/10 and net over and then spray with mortar. Walls have to be previously cleaned with open fugues. For smaller cracks (position I) do injection with premixed injection masses based on cement emulsion. All cracks previously clean, from wall face cracks secure with cement mortar and on every 40cm put tubes for injection liquid. Injecting do with over pressure max 0.50bar. All larger cracks additionally secure with reinforce bars.

These kind of sacral structures are pretty tall and not steady from influence from earthquakes in level of domes. Here, dome was damaged even before earthquake with ventilation hole for candles fireplace. Epicentre closeness of earthquake was causing to numerous of damaged walls, domes and arches of this structure. Facility was rehabilitated in summer 2012 year and constructor gave photos during and after work was done(Fig.5a,b,c,d,e,f and 6a,b,c,d). On work beginning redo brick work at domes and done thin reinforced concrete shells over them, than reinforced concrete beams of wreaths façade walls and inside arches , rehabilitated cracks by inside and outside walls.

a),  b) 

Fig.5a,b. Recovery sphere arches with thin reinforce plate

c)  d) 

Fig.5c,d. Recovery sphere arches with thin reinforce plate

Fig.5e,f. Chords has to be replaced with new beams with 14/18cm dimensions.

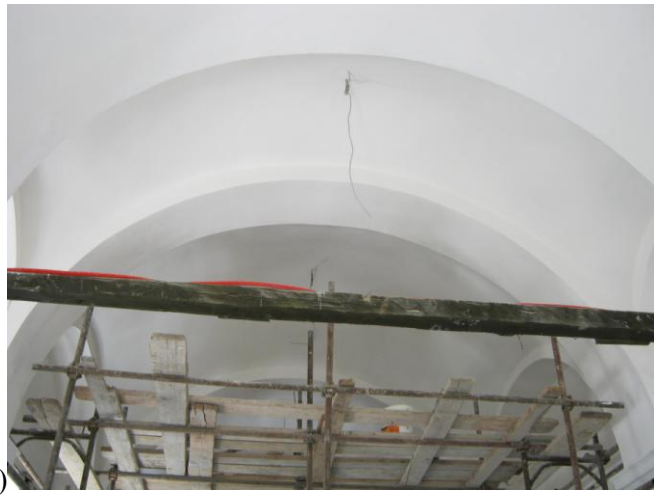
a)  b) 

Fig.6a,b. Replacing wooden purlin with new truss and recovery inside

c)  d) 

Fig.6c,d. Replacing wooden purlin with new truss and recovery inside

a)  b) 

Fig.7a,b. Church after recovery and reconstruction

4. Conclusion

Building was damaged very much from earthquake impact whose epicentre was nearby. Rehabilitation and reconstruction required wide range of work on execution reinforce concrete shells and beam wreaths, as well and fill out of major cracks in arches above windows and walls of construction , replacement and reinforcement of roof construction. Now facility is operational(Fig.7,a,b).

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NON-DESTRUCTIVE EVALUATION AND MONITORING OF TIMBER STRUCTURES

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***Abstract**: Design assumptions for historic structural members based on contemporary codes may not adequately reflect the strength and quality of the material, leading to unnecessary, intrusive, and often expensive interventions or replacement of culturally valuable material. Therefore, direct assessment of the material characteristics in situ is a more suitable approach. The presented research deals in a complex way with the investigation of the use of non-destructive testing methods for the evaluation of structural timber and for determination of the performance of structural timber elements. The paper focuses also on the effect of differences in the device structure on results of measurement of timber properties. Final part of paper deals with system of opto-fiber sensors for monitoring and diagnostics of mechanical stress of timber beams. The correct function of the proposed system was verified by mechanical tests.*

Key words: *Non-destructive testing techniques, accuracy, monitoring of structures*

1. Introduction

Non-destructive testing (NDT) techniques have been developed to convey the quality and condition of wood while minimizing impact on existing fabric. The ability to implement these techniques in the field, with minimal damage imposed on the structure, and without the need to transport fragile material to a laboratory for testing renders them desirable and effective for assessing historic timber structures.

The non-destructive methods for testing wood members described below offer advantages and disadvantages. They supply valuable information regarding the condition of the material tested, but are limited and may not provide accurate mechanical properties necessary for a thorough analysis of the member structural capacity. X-ray is primarily qualitative and exposes the user to small levels of radiation. The radial core drilling method establishes values for compressive strength and the modulus of elasticity, but requires fragile cores be removed and taken to a laboratory for testing. Wood load-jacking provides values directly in situ and has shown good correlations compared with results from radial core drilling. GPR is used successfully for concrete and masonry, but has had limited application to wood, mainly due to difficulty interpreting GPR data and because acquired information is primarily qualitative.

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Table 1 Description of established NDT techniques used for historic timber structures

NDT Method	*Description*
Ultrasonic and Impact Sound Wave Velocity	An ultrasonic or impact generator sends a stress wave through a member. Transducers placed at a fixed distance transmit a wave through the member, recording the time traveled. Wave travel time corresponds the elastic modulus where density is known.
Radiography (X-ray) Imaging	A radiation source emits a beam of radiation directed towards a member. Embedded objects and material variations will have varying absorption rates based on the material density and thickness. Penetrating radiation depicts the internal structure.
Penetration Test	A metal pin is shot into a member at a constant energy by means of a spring, providing information on the material density near the surface.
Resistance Drilling	A very long and thin drill bit penetrates a member and the resistance against drilling action is measured and recorded by a Resistograph. The output generated is a profile of energy consumption shown as a series of peaks and valleys along a graph.
Radial Core Drilling	Small diameter core samples are removed from a member and then taken to a laboratory for testing of material properties.

2. Correlations and differences between NDT methods

This part of paper is focused on comparing the results of assessment of mechanical properties of structural timber by visual grading, machine grading and NDT evaluation in-situ. Structural timber has been evaluated with application of two different grading machines, three devices for NDT evaluation and destructive tests. The most important result is comparison of static and dynamic modulus of elasticity of timber, and further comparison of strength classes obtained by different measuring methods.

Norway spruce (*Picea abies (L.) Karst*) laths with cross section of 35/70 mm and length 2,7 m were used for experiments. At first, laths were evaluated visually and sorted in strength classes according to standard ČSN 73 2824-1(identical with DIN 4074-1) [4]. Then they were mechanically graded (Goldeneye 702 and Metriguard HLT 7200). Three portable devices (Sylvatest, Fakopp and Timber Grader MTG) were used for measuring the speed of propagation of acoustic signal. The destructive tests were carried out according to standard ČSN EN 408 [3] for comparison with non-destructive tests. Modulus of elasticity and bending strength were determined during the destructive tests.

2.1. Machine grading

The first machine was Goldeneye 702 by Microtec which uses X-rays. The output values are the density of the timber element, the dimensions of element and static modulus of elasticity. Modulus of elasticity is measured over the entire length of the element.

The second machine was Metriguard HLT 7200 (MGP10) which uses real bending deformation of timber element for calculating static modulus of elasticity. The measurement is continuous for all cross-sections at least 600 mm away from the edges of the timber element. Average and minimum value of modulus of elasticity in bending were used within each element for the next data processing. Achieved outputs are strength class, average and minimum recorded value of modulus of elasticity in bending.

2.2. Non-destructive evaluation

The first device used was Sylvatest. Sound waves propagation through material is good and proven method with a clear theoretical background. The speed of the wave is also used for the determination of internal damage to wood, where a lower speed indicates rot or other internal damage [5]. Nowadays, this method is used by machines for sorting cut

timber and mobile devices determining the mechanical properties of inbuilt elements. Measurement of modulus of elasticity using Sylvatest device is based on the ultrasonic wave propagation between the measuring probes. When defining wood species (spruce, pine), cross section (squared, timber), element length and moisture content, the device classifies the element into strength classes (S7, S10 and S13) and also determines the value of the modulus of elasticity in bending. For the comparison of methods for determining modulus of elasticity, the time of ultrasonic wave propagation was recorded. Based on this time and density it is possible to calculate a dynamic modulus of elasticity. In this case, the density is not determined by device but is obtained by weighting. The modulus of elasticity in the direction parallel to grain is then determined by a simple relation:

$$E = \rho \cdot v^2 \tag{1}$$

where E is modulus of elasticity *[Pa],*
ρ is density *[kg/m^3],*
v is speed of ultrasonic signal *[m/s]*.

Second device was Fakopp Microsecond Timber. The Fakopp device measures the time of acoustic wave propagation through timber element, including the length of coaxial connections of probes. Therefore, it is necessary to subtract time of the propagation of acoustic signal through the probes connecting from measured time. The acoustic signal is created by a hammer strike on the acoustic probe. The calculation of the dynamic modulus of elasticity is the same as in the case of Sylvatest. The values of dynamic modulus of elasticity obtained from the two devices are very similar.

Last device for detecting the mechanical properties used was Timber Grader made in the Netherlands. This device is able to determine the strength class according to standard ČSN EN 338 [2] and static modulus of elasticity. The input variables are mechanically created shock waves, sample size, moisture content and a mass of sample. The disadvantage is the necessity of measurement on the faces of the sample. It is often impossible with inbuilt samples.

2.3. Destructive tests

For the destructive tests the samples were adjusted according to the requirements of standard ČSN EN 408 [3]. The loading of the timber sample was controlled by deformation. The progress of force and deformation was recorded for each sample. The aim of destructive tests was to obtain static modulus of elasticity and bending strength of the timber laths. The part of working diagram between 10 and 40% of maximum power achieved at failure was used to calculate the modulus of elasticity.

2.4. Description of achieved results

The main output of machine grading, measurement by non-destructive methods and destructive tests is the modulus of elasticity. Machine grading, destructive tests and device Timber Grader MTG give a static modulus of elasticity. Acoustic non-destructive methods give a dynamic elastic modulus. The Timber Grader device also classifies elements into strength classes according to standard ČSN EN 338 [2]. The output of the visual grading is classification to the visual classes according to standard ČSN 73 2824-1 [4] and subsequent transfer to the strength classes according to standard ČSN EN 1912 [1]. From destructive tests was also obtained the bending strength according to standard ČSN EN 408 [3] besides modulus of elasticity. These results were compared and evaluated. In all cases, it is a linear regression with one variable.

The correlation between modulus of elasticity corresponding to strength classes and modulus of elasticity from the destructive tests is 0,70. The lowest values of bending strength from destructive test were found for samples that were visually classified as class C18. This confirms the good practical use of visual sorting.

The most interesting result is the correlation between the MoE obtained by grading machine or devices for non-destructive evaluation in-situ and MoE from the destructive tests. The highest degree of correlation was found for the Timber Grader device (Tab. 2). Other acoustic devices Sylvatest and Fakopp also show a high degree of dependence. However, we can not overlook the fact that all three of these methods require the knowledge of density of timber element. Due to the high speed sorting and null input data, the correlations between values from grading machines and values from devices for non-destructive evaluation are comparable.

Table 2 The coefficients of correlation between the modulus of elasticity from destructive tests and from other procedures

	Goldeneye 702	Metriguard HLT 7200	Sylvatest	Fakopp	Timber grader
Coefficients of correlation R	0,80	0,94	0,94	0,95	0,97

Dynamic modulus of elasticity determined by acoustic methods (Sylvatest and Fakopp) came out about 31% and 27% higher than the static modulus of elasticity from the bending test. This confirms the results from literature for spruce and other coniferous or deciduous timber.

The degree of correlation between density and bending strength reaches a value of 0,57. This is average value in comparison with the means of other papers. The modulus of elasticity has proven as the better predictor of strength. For the static modulus of elasticity the correlation is in the interval 0,76 – 0,86. The coefficient of correlation between strength and dynamic modulus of elasticity came out as 0,80 (Sylvatest) and as 0,81 (Fakopp). This confirms the findings from the literature that the static modulus of elasticity is a better indicator of strength than the dynamic.

3. Influence of the device structure on results

One of the semi-destructive methods of timber assessment in structures is based on measurement of material resistance against penetration of an indenter. The damage of a timber structural member is small after the tests therefore the device is often used also in health assessment of historical structures. One of the well-known devices is Pilodyn 6J Forest that penetrates the surface layers of timber element by a short pin with the help of an internal spring. It is possible to determine wood density and even the related mechanical properties with the input of the depth of the pin penetration. The method is strictly local and the device is also used to detect biotic damage of wood in timber structures, especially rot attacks.

The disadvantage of the method is its device-dependency which makes adequate comparison of tests results measured by different but same type tools impossible. The next part of paper focuses on the effect of differences in the structure of the tool on results of measurement of timber properties.

3.1. Estimation of density and mechanical properties of wood

Wood density is of key importance value of timber. Very important also for the use of other NDT methods. Wood density is a complex character, determined by several factors, e.g. cell diameter and cell wall thickness, proportions of earlywood and latewood, cellulose and lignin content etc. Density has a significant influence on wood mechanical properties and the effect of a non-destructive parameter is frequently associated with the

density [6]. The depth to which the pin penetrates is indicated on the instrument and is inversely proportional to the density of the wood. [7]

The equations below are used for estimation of wood density ρ_{12} in the paper. The calculations were developed to be used for in situ measurement of timber quality in the existing buildings [8] and [9]. They are based on moisture content (MC) measurement, which is an integral part of each testing of wood, and the depth of penetration t_p.

$$\rho_{12} = - 0{,}027102 \cdot t_{p,12} + 0{,}727987 \quad (2)$$

$$t_{p,12} = t_p \ (1 - 0{,}007 \ \Delta w) \quad (3)$$

$$\Delta w = w - 12 \quad (4)$$

where ρ_{12} is density of wood with MC of 12 % [g.cm^{-3}],
$t_{p,12}$ the depth of penetration into wood with MC of 12 % [mm],
t_p the depth of penetration into wood with measured MC[mm],
w measured MC [%].

Not only the strength f_m, but also the modulus of elasticity E can be estimated with help of penetration depth t_p or more precisely density ρ_{12}. The relationships between the quantities with specific constants empirically determined resulted from the research [8],

$$f_m = - 36{,}13 + 0{,}156743 \cdot \rho_{12} \quad (5)$$

$$E_{stat,05} = - 14441{,}49 + 55,\ 7834 \cdot \rho_{12} \quad (6)$$

3.2. Accuracy of measurement

To compare the accuracy of measurement with semi-destructive device Pilodyn two tests of four devices were performed. Devices were tested for stiffness of the spring. Pilodyn with the stiffest spring (CET) was supposed to have the higher values for penetration depth t_p. It is observed only in three cases. VSB, the second stiffest spring, had the deepest penetration in six cases. Nearly same spring of ITAM had comparable results, almost always lower value of penetration depth. CTU with the less stiff spring had mostly the lowest values of penetration depth.

The best comparable depths within one lamella were observed in lamellae where the measurement direction of was allowed to be almost radial; e.g. lamella no. 5, 6 and 8. They show the same order of devices based on depth penetration and their comparison is the most proper.

Penetration depth $t_{p,12}$ was used to assess wood density ρ_{12} according to (2). As the relationship between density and mechanical material property of wood exists, strength f_m and static modulus of elasticity $E_{stat,05}$ were calculated with help of (5) and (6) for this test also. The results for weakest and the strongest lamellae are summarized below for better comparison (Tab. 3).

As shown in the Tab. 3, any two devices are not the same and the difference between the measured values can be significant. Only comparison of measurements performed by the same device can be relevant. It is not possible to compare results from various studies carried out by different but same type tools thoughtlessly; a calibration is necessary.

Table 3 Results for all four devices, evaluated quantities are density of wood, strength and modulus of elasticity

		CET	VSB	ITAM	CTU
(strongest) lamellae no. 3	penetration $t_{p,12}$ [mm]	10.0	9.0	8.5	9.8
	density ρ_{12} [kg.m^{-3}]	457	484	498	464
	strength f_m [MPa]	35	40	42	37
	$E_{stat,05}$[MPa]	11 050	12 562	13 317	11 428
(weakest) lamellae no. 7	penetration $t_{p,12}$ [mm]	13.0	13.0	12.5	12.0
	density ρ_{12} [kg.m^{-3}]	376	376	389	403
	strength f_m [MPa]	23	23	25	27
	$E_{stat,05}$[MPa]	6 514	6 514	7 270	8 026

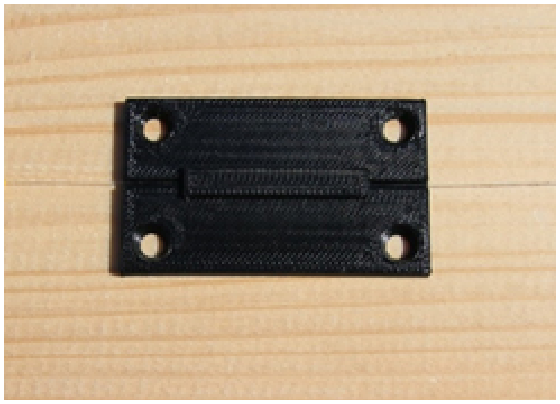
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4. Mechanical stress monitoring of wooden structure by 3D printed structure with embedded fibre Bragg grating

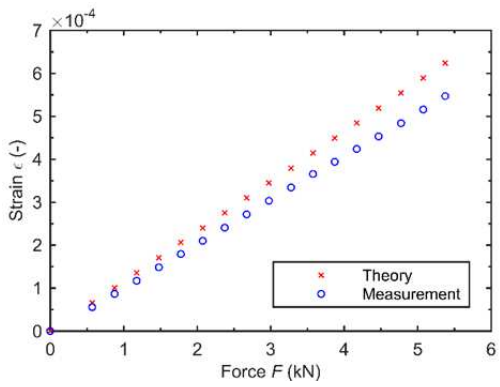
Safe operation of the construction should be the highest priority. Fibre Bragg grids can increase safe operation due to their sensing ability, low weight and cable size, which can be incorporated into various types of materials. Such built-in sensors are able to monitor structural health in critical locations that are not available to conventional sensors requiring additional mounting. However, in some cases, the sensor implementation into constructions during construction may has a negative effect on structural integrity and may be restricted.

In recent years, a lot of attention has been directed towards the fabricating of smart structures with embedded optical sensors providing in-situ, non-destructive and real time or on-demand information about the structural health. Fibre Bragg grating is used as the sensing element and embedded during the 3D print of the structure. A prototype structure was printed in order to verify that a fiber sensor embedded by 3D printing is able to measure mechanical strain. The response of a fiber Bragg grating is tested and evaluated. Prototype is designed to be thin and flat in order to provide high elasticity being necessary for easier straining. The basic size of the structure is 50x30x3 mm. At the edge of the structure, a small groove is created to show the center. This helps not only to place the grating to the center during the 3D printing, but also to stick or screw the structure correctly on the surface being monitored. Polycarbonate-acrylonitrile butadiene styrene (PC-ABS) is used for 3D printing for the low superior strength, flexibility, as well as good impact and heat resistance. The response of the inscribed FBG sensor is measured in a common way. The infrared radiation is pushed from the broadband laser source into one side of the optical fiber. In the middle of the structure, the part of the spectrum is reflected back from the grating inscribed. A circulator separates the reflected spectrum from the input signal to irradiate a spectrum analyzer. The measured spectrum is then processed in a computer by the centroid detection algorithm to calculate central wavelength β dependent on the applied strain. Strain ε is given by (7).

$$\varepsilon = \frac{\Delta\beta}{\beta \cdot (1 - \rho_e)} \tag{7}$$



a)



b)

Fig. 1. a) strain measuring structure glued on the wooden beam; b) measured and calculated strain ε for applied force F

where $\Delta\beta$ is difference between the measured and initial wavelength, ρ_e is photoelastic coefficient that is equal to 0.78 at the wavelength of 1550 nm. The grating used in this experiment is inscribed into the fibre to reflect at this wavelength.

In order to verify the sensing functionality, the proposed structure has been glued on the wooden beam, as shown in Fig. 1 a). The wooden beam is strained concurrently with the FBG confined inside the proposed structure. A 4-point flexural test [3] is employed ensuring that the grating is strained uniformly with the timber sample along the propagation axis.

In the test, the applied force from a hydraulic cylinder is increasing by 20 N·s-1 to reach the maximum at about F = 5.5 kN. The maximal force was estimated in order to satisfy that the prototype is not damaged during the testing. The maximal force under which the FBG is able to operate was determined by measurement is about 7 kN. Above this value, the fiber sensor is broken away from the glue and 3D printed material.

During the test, the applied force, the wood deflection and the reflected wavelength used for strain calculation is recorded. The results from the testing are shown in Fig. 1 b). At the maximal force, the wood deflection about 3.75 mm is detected corresponding to the mechanical strain about 0.65 mm·m-1. For this strain value, the wavelength shift about 1 nm is measured.

As one can observe in Fig. 1 b), the curves are almost linear, but the slopes differs from each other. Since the (7) is linearly dependent on the photoelastic coefficient, the coefficient can be varied to meet the both slopes. The photoelastic coefficient is found to be 0.802 for the both slopes being met. This implies the change by about 8 %. Therefore, the calibration of the FBG sensor can be necessary.

The temperature and humidity influence on the measured values is neglected, because of the nearly constant environment in which the measurement is performed. However, for a long term measurement, where the temperature changes are higher than about ±5 °C, an external or internal temperature compensation is required. In addition, using the sensor in different temperature conditions, can require recalibration. Contrary, the sensor is almost immune to humidity changes. More detailed information can be found in [11].

5. Conclusions

Several aspects for future application of non-destructive techniques in building survey and restoration shall be considered. This may include enhancement of NDT

methods accounting for specific applications in the structural assessment of existing buildings in the current state (diagnosis), the extension of application range of existing methods to species besides spruce and pine, especially to hardwood species commonly used for historic buildings (relevance), comparative calibration of new and existing methods with numerical analysis (validation), and preparation of draft proposals for technical guidelines in the field of building inspection by means of NDT methods (implementation).

Acknowledgement

This work has been supported by Technology Agency of the Czech Republic, project TE02000077 Smart Regions - Buildings and Settlements - Information Modelling, Technology and Infrastructure for Sustainable Development.

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IMPLEMENTATION OF EXPERIMENTALLY DEVELOPED METHODOLOGY FOR SEISMIC STRENGTHENING AND REPAIR IN HISTORIC MONUMENTS

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***Abstract**: To develop appropriate approaches for retrofitting of Byzantine churches, in general, and particularly churches located within Macedonia, research projects on seismic retrofitting, including seismic isolation of Byzantine Churches in Republic of Macedonia were realized in the period 1990-2000. To experimentally verify the methodologies, a model of the church of St. Nikita, selected as a representative prototype church in Macedonia, was constructed to a scale of 1:2.75 and tested on the seismic shaking table in the Dynamic Testing Laboratory of IZIIS. The project program included shaking table testing of the unaltered church model, the church model strengthened by using "ties and injection", as well as the base isolated church model. This paper presents the implementation of originally developed and experimentally verified strengthening methodology in the process of reconstruction of three very important cultural-historic monuments.*

***Key words**: historic monuments, earthquake protection, shaking table testing,*

1. Introduction

The problem of earthquake protection of historic structures and monuments is radically different from that of other structures, due to the priority given to preservation of aesthetic, architectonic and historic values instead of keeping the structure operational. In providing the protection of these structures in a manner that requires the least intervention and the greatest care to preserve authenticity, the experts are permanently challenged by the fast development and the improved performance of new materials and techniques. However, the implementation of particular strengthening methodology depends on the extent it has been investigated. The delicate problem of proving the effectiveness of the selected consolidation system can be successfully overcome by using the methodology of design assisted by testing.

2. Development of Methodology for Seismic Retrofitting of Byzantine Churches

Within the frameworks of the scientific research projects realized at the Institute of Earthquake Engineering and Engineering Seismology, IZIIS, Skopje in the period 1990-2000 for the purpose of development of appropriate methods for seismic retrofitting of the Byzantine monuments in general, and particularly the Byzantine churches located within

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Macedonia, shaking table testing of a church model in a realistic geometrical scale was performed for the first time in the world. 1:2.75 scaled model of St. Nikita church was constructed and tested on the seismic shaking table in the IZIIS laboratory in its original state, strengthened state by use of "ties and injection", and as a base isolated model. The knowledge gained in this way is unique and incomparable and hence necessary for seismic strengthening of individual important cultural-historic structures where it is important to have an insight into the effect of the interventions upon the authenticity of the monument.

2.1. Seismic Strengthening by use of "ties and injection"

Experimental and analytical investigations were performed to verify an original methodology that was developed for the repair and seismic strengthening of Byzantine churches. This work was part of the long-term research project entitled "Study for Seismic Strengthening, Conservation and Restoration of Churches Dating from the Byzantine period (9th-14th century) in the Republic Macedonia realized jointly by the Institute of Earthquake Engineering and Engineering Seismology (IZIIS) - Skopje, the Republic Institute for Protection of Cultural Monuments (RZZSK) - Skopje and the Getty Conservation Institute (GCI) -Los Angeles, California, [1,2].

The churches dating from the Byzantine period located in Macedonia are important architectural structures and contain extraordinary collections of highly important frescoes. Based on certain criteria (typology, existing state, interventions and authenticity), four representative churches, were selected for investigations which involve field and analytical tests for definition of seismic parameters, main dynamic characteristics and analysis of seismic resistance. The church of St. Nikita in the village of Banjani has been selected as a prototype church representative of the Byzantine churches in Macedonia, (Fig. 1).

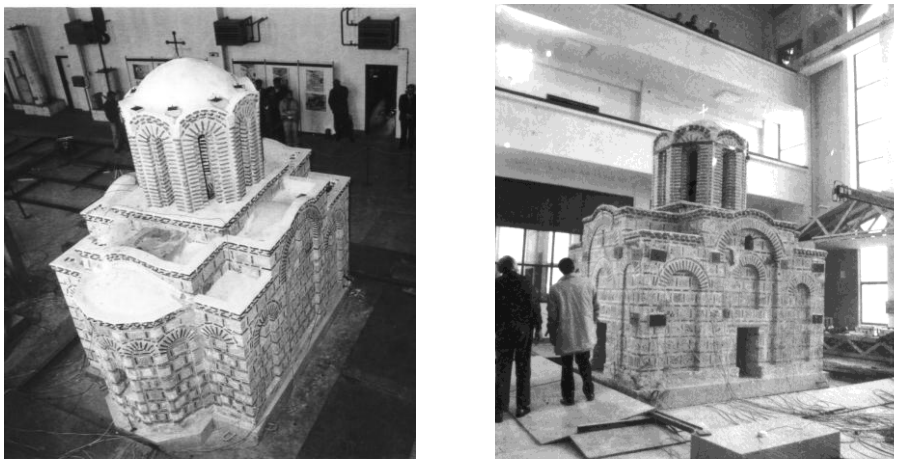


Fig. 1. The church of St. Nikita, v. Banjani

Ample field and analytical studies, in situ and laboratory tests were performed for the existing structure of the St. Nikita church for the purpose of defining the physical-mechanic and chemical characteristics of the built-in materials, the dynamic characteristics of the structure and the seismicity of the terrain. A preliminary analysis of the seismic stability of the existing structure points to nonsufficient ultimate bearing and deformability capacity according to the design criteria on seismic safety. The proposed concept of strengthening that thoroughly complies with the main principles for conservation and protection of historic monuments consists of incorporation of horizontal and vertical steel ties and filling the area around them with a corresponding material for the purpose of increasing the bearing capacity and the deformability of the existing structure.

To experimentally verify the methodology for repair and seismic strengthening of Byzantine churches, a model of the existing structure of the prototype church of St. Nikita (M-SN-EXIST) was constructed and tested on the seismic shaking table in the Dynamic Testing Laboratory of IZIIS, (Fig.2). The geometrical scale of the model church was selected on the basis of the characteristics of the seismic shaking table and the precisely defined objectives of testing, i.e., on the basis of the following criteria:

K1 - proportions of the shaking table (4.5 m x 4.5 m)
K2 - allowed total height of the model (10m)
K3 - allowed total weight of the model (400 kN)
K4 - realistic reproduction of nonlinear behaviour
K5 - realistic reproduction of the failure mechanisms



M-SN-EXIST M-SN-STR

Fig. 2. Models on shaking table

Satisfying these criteria and adopting the “"gravity forces neglected" modeling principle, the following three main scales were adopted: (geometrical scale Lr = 1:2.75, scale for the bulk density of the material ρr = 1, scale for the stresses Er = 1). The main targets of the experimental tests performed on the model-church M-SN-EXIST that simulated the existing state of the church of St. Nikita can be summarized into two groups: (i) assessment of the vulnerability of structures of interest, and (ii) selection of most appropriate procedures for repair and strengthening of damaged structures in post-earthquake protection. To that effect, a programme of experimental tests was adopted by gradual increase of the intensity of input earthquake excitations aimed at monitoring the progressive development of cracks and the failure mechanism, the modification of the dynamic characteristics as well as the phases of dynamic behaviour of the model, i.e., defining the elasticity limit (occurrence of the first cracks).

The investigation was performed by simulating two main types of an earthquake: the Friuli (Italy) earthquake of 1976 (Breginj record) as a local earthquake and the 1979 Montenegro (Petrovac record) and 1940 El Centro earthquakes as earthquakes from distant foci. From the general behaviour of the model, it was concluded that it behaved as a rigid body in the elastic range, while at occurrence of the first larger cracks, there was separation of the bearing walls and development of damages up to a state close to complete failure. This was proved by the decrease in natural frequency from 11. - 6.6 Hz.

The damaged model was structurally strengthened in accordance with the proposed methodology and the main conservation and model analysis principles, (Fig.3). Strengthening elements anticipated with the strengthening method were the horizontal and

the vertical belt courses that were formed by incorporation of steel reinforcement into the wall mass and filling the area around them by an appropriate injection mixture that enabled contact with the surrounding existing masonry. The repaired and strengthened model (M-SN-STR, Fig. 4), was subjected to the same series of dynamic tests for the purpose of direct comparison of the quantities of interest and proving the efficiency of the applied method of strengthening. However, due to the high resistance of the strengthened model, the tests were continued under higher intensities. The response of the strengthened model was considerably different from that of the original model. Characteristic was the increased elasticity limit and reduction of displacements at the top. Although there was a considerable deterioration in bearing capacity and stiffness under maximum seismic effect, the complete stability of the model structure was not disturbed due to the presence of ductile elements, while the damage was such that it was repairable. The structure of the strengthened model was in a state of deep nonlinearity but still far from failure.

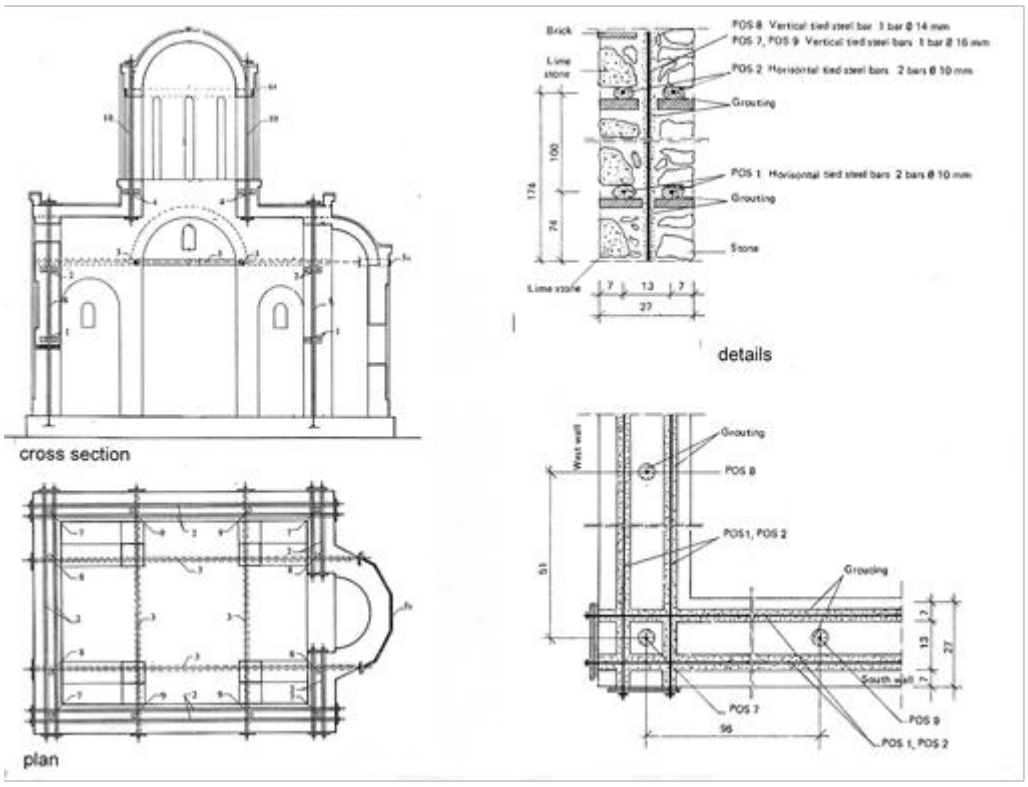


Fig. 3. Applied method for strengthening

Comparative analysis of results obtained from experimental tests of both models, (Table 1), shows that:

• The response of the structures to the acceleration of up to a_{max} = 0.20 g shows first nonlinear cracks in original model, whereas the strengthened model remains evidently completely elastic.

• Under the $a_{max} \approx 0.40g$, the original model suffers severe nonlinear damage close to failure, whereas the same level is the elasticity limit and beginning of nonlinearity for the M-SN-STR.

• The tests on both models evidently show high amplification of acceleration at the top of the dome.

• A qualitatively different type of failure mechanism has also been observed. The strengthened model does not suffer separation of bearing walls and vertical crack, but the failure is transferred in the lower zone and results in occurrence of diagonal cracks.

• The applied methodology for repair and strengthening that is in compliance with the principle of "minimum interventions - maximum protection" increases the bearing capacity and deformability of the structure up to the level of the designed protection which is proved by comparison of the results from the experimental tests on the original and the strengthened model.

2.2. Earthquake Protection of Byzantine Churches Using Seismic Isolation

As a continuation of previous activities, experimental and analytical investigations as well as shaking table testing of isolated church model were performed to develop a methodology for application of seismic isolation as a way of seismic protection of a large number of similar cultural monuments. These investigations were realized at IZIIS within the framework of the joint US-Macedonia research project and PHARE Cultural development program. [3, 4].

To experimentally verify the methodologies for seismic isolation of Byzantine churches, a base isolated model of the church of St. Nikita was tested on the seismic shaking table. For the needs of testing the isolated church model, 8 seismic isolators of the type of rubber bearings were specially designed and produced in the Republic of Macedonia, (Figure 4). The schematic presentation of the system clearly distinguishes three main elements:

A. Laminated rubber bearing element for receipt and transfer of vertical gravity forces and limited displacement (insulation) in horizontal direction (element 1)

B. Steel plate damper (element 2) which has the role of a damper in the form of hysteretic behaviour but only after the linear behaviour of the laminated rubber bearings is exhausted. This element is different from the other systems by being not activated in the first phase of linear behaviour of the laminated bearing isolators but in the second phase under higher excitation expected to induce large displacements.

C. In conditions of specific behaviour and requirements for historic monuments regarding their protection in seismic conditions, one of the elements is "limited displacement". This criterion is satisfied by design of the third element of our system - "stopper element".

In this way, the main requirements are satisfied as to linear behaviour of the system (meaning without damage and deformations) under slight and moderate earthquakes and its limited but controlled displacements by activation of elements 2 and 3 (hysteretic dampers and stopper elements) under catastrophic earthquakes. This enables complete protection of the structure and the valuable objects within the structure like frescos and other elements.

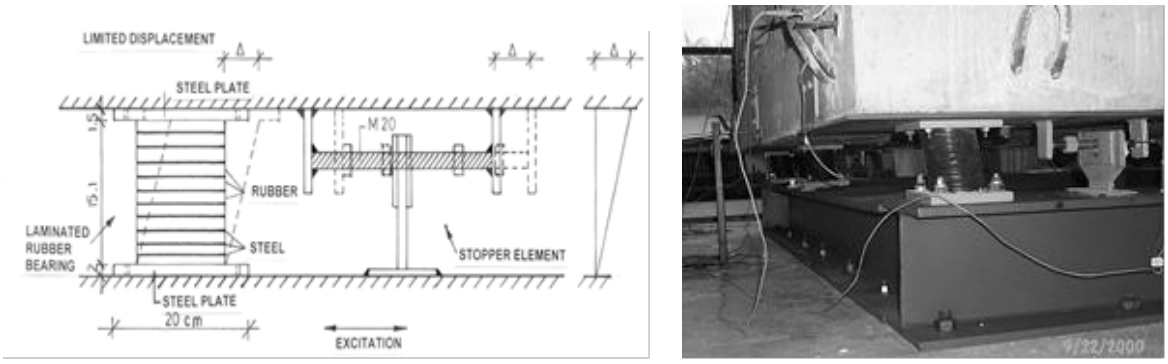


Fig. 4. Laminated rubber bearing for seismic isolation

For such a selected isolation system, detailed analytical investigations have been performed for different levels of input acceleration and for different earthquake records. Designed and constructed for the optimal system of a total of 8 isolators has been a steel

structure for connection of the isolators with the church model structure and the seismic shaking table (Fig. 5). The base isolated model, labeled M-SN-BIC, was then placed on a shaking table on a specially designed steel structure for connection with the isolators an the shaking table.

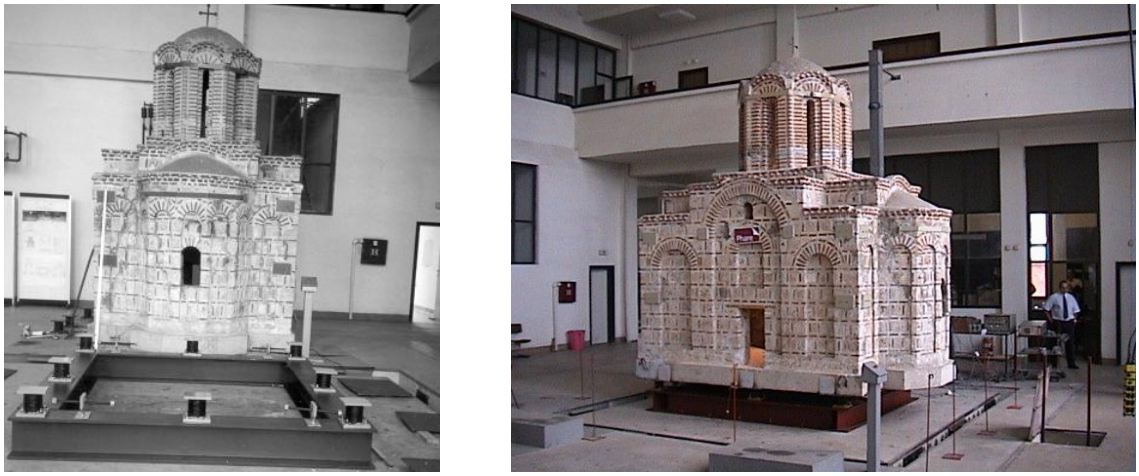


Fig. 5. Base isolated church model on shaking table

Table 1 shows the comparison between experimentally obtained results for all three models, i.e. between previously tested original, non-retrofitted model (M-SN-EXIST) and strengthened model by use of "ties and injection" (M-SN-STR), and the base isolated model (M-SN-BIC). From the comparison of the experimental results the following can be concluded:

Table 1. Comparison between the experimental results of three models

		Output acceleration (in g) for the church models					
Earth-quake	Input acc (g)	M-SN-EXIST		M-SN-STR		M-SN-BIC	
		level 1	level 2	level 1	level 2	level 1	level 2
El Centro	0.17	0.29	0.55	0.20	0.47	0.07	0.10
El Centro	0.30	-	-	0.65	1.10	0.10	0.14
El Centro	0.49	-	-	0.91	1.59	0.23	0.31
El Centro	0.54	-	-	0.77	1.41	0.35	0.68
El Centro	0.63	-	-	-	-	0.42	0.82
Petrovac	0.19	0.39	0.76	0.27	0.48	0.09	0.15
Petrovac	0.40	-	-	0.77	1.36	0.15	0.28
Breginj	0.17	0.22	0.52	0.30	0.55	0.07	0.10
Breginj	0.28	-	-	0.20	0.40	0.10	0.16
Breginj	0.38	-	-	0.34	0.79	0.14	0.23

• The results from these tests, especially compared with the results from the previously tested strengthened model by use of "ties and injection", pointed to a decrease of input acceleration in the model structure for 50-60%.
• It was evident that the output displacement and acceleration through the height of the base isolated model were considerably smaller than that for the strengthened model.
• There is also significant difference in the amplification of the dynamic response of the dome structure, in the case of M-SN-BIC it was negligible.

• The failure mechanism of the base isolated model is completely different from the previous models. For all testing up to input acceleration of 0.60g, the base isolated model behaves as a rigid body without any visible cracks. When the cracks formed they were in a pattern similar to those of the original model.
• The base isolated model did not suffer damage under low and moderate earthquake intensities and the damages under maximum expected accelerations with a return period of 1000 years, (a_{max}=0.54g) were minimal and absolutely allowable and repairable.
• Finally, to conceive the efficiency of the applied seismic isolation system, the model was tested under earthquake intensity greater than the expected one, (El Centro, a_{max}=0.63g). Under this intensity, the monument was evidently stable and safe, with the exception of some minor damage

The main result of the project is demonstration of the efficiency, as well as technical, economical and conservatory justification of the applied technique of base-isolation on the structure of the church model. The tests undoubtedly proved that the new technology of seismic base isolation of historic monuments offers absolute safety and protection and that its application should become an imperative in earthquake protection of historic monuments in future.

3. IZIIS' integrated approach in seismic protection of historic monuments

Within the frames of the IZIIS' research activities, in addition to seismic design and protection of modern structures, particularly noteworthy is also the experience gathered in the field of protection of structures pertaining to the cultural historic heritage. During a period of more than 30 years of activities in this field, the Institute has realized important scientific research projects involving experimental and analytical research, field surveys of historic structures and application of knowledge during earthquake protection of important cultural historic structures and monuments.

Extensive research activities have been performed by IZIIS for the purpose of evaluation of a procedure for repair and strengthening of valuable historic monuments. Such a procedure is based on conventional understanding of retrofitting, although, in our concepts, there are also techniques, which are based on the idea of structural control. As a result of several decades of gathering of experience, it can be said that an integral approach to seismic protection of extraordinarily important cultural historic structures has been adopted by the Institute. This approach, first of all, complies with all the restoration and conservation requirements set in a number of international documents and declarations, as well as procedures and legislative regulations for high category structures. This integrated approach to repair and seismic strengthening of historic monuments encompasses the following:

- Definition of expected seismic hazard;
- Definition of soil conditions and dynamic behaviour of soil media;
- Determination of structural characteristics along with the bearing and deformability capacity of existing structures;
- Definition of design criteria and development of a concept for repair and/or strengthening;
- Design of structural methods, techniques, materials and types of excitation;
- Determination of the response of repaired and/or strengthened structures and verification of their seismic stability;
- Definition of field works, execution and inspection.

Although the above stated seems to be the "normal procedure", it is the only way of providing high quality in protection of cultural heritage. This task is certainly much more than simply listing of what is to be done since it requires a lot of knowledge and efforts.

The knowledge gained through the above mentioned investigations is used in the individual phases of the analyses for the purpose of obtaining more accurate input parameters and carrying out a realistic analysis of the structures. Generally, the procedure consists from: (i) elastic – static analysis carried out on a 3D mathematical model by application of the finite element model, (ii) analysis of elements up to ultimate state of strength, deformability and ability of the bearing elements and the system as a whole to dissipate seismic energy, and (iii) analysis of the dynamic response of the system for actual seismic effects with intensity and frequency content that are expected on the considered location.

4. Reconstruction of monuments implementing the developed methodology

After the realization of the project on Byzantine churches and gained unique and incomparable knowledge, IZIIS became partner of the National Conservation Centre in R. Macedonia, which enabled direct application of the gained knowledge in actual conditions and for specific historic monuments. Presented further are the three most characteristic examples of application of the developed methodology in the process of reconstruction and seismic upgrading of monuments.

4.1 Reconstruction and Seismic Strengthening of St. Athanasius Church

On August 21, 2001, during the armed conflict in R. Macedonia, the monastic church of St. Athanasius in Leshok experienced strong detonation, which resulted in its almost complete demolition, (Fig. 6). From structural aspects, there have been two approaches taken in the attempt to renovate and reconstruct the structure. Based on the performed detailed analysis of the structure, (i) solution for repair and strengthening of the existing damaged part of the monastic church and (ii) solution for seismic strengthening of the ruined part of the church to be reconstructed were made, [5].



Fig. 6. The Church after detonation (left) and after reconstruction (right)

1. The solution for *repair and structural strengthening* of damaged existing part of the structure anticipates (i) injection of all the cracks and (ii) incorporation of strengthening elements (vertical RC jackets along the inner side of the walls of the staircase core and the columns of the tambours, horizontal RC belt courses at the level of the floor structure and at the base of the domes, RC slab below the floor level, as well as steel ties besides the timber beams in the floor structure of the gallery.
2. For the demolished part of the structure, a *concept of complete reconstruction* by maximum possible use of selected material has been adopted, whereat elements for structural strengthening for providing the necessary level of seismic safety have also been anticipated: (i) incorporation of RC belt course below the floor level, in the existing foundation walls, below the massive walls for the purpose of connection with

the vertical strengthening elements, (ii) incorporation of vertical strengthening steel elements at the necessary height, at the ends of the massive walls and around the openings, (iii) incorporation of vertical strengthening steel elements into the tambour columns composed of deformed reinforcement, (iv) incorporation of horizontal steel elements along the massive walls, in the base of the tambour and in the base of the dome

3. Due to the different treatment of the structural units constituting the integral structure, an *expansion joint* between them is anticipated to be constructed and dictate the concentration of damages during future earthquakes.

The church was reconstructed according to the designed methodology in 2002-2004. The results from the analysis show that both structural units constituting the integral structure possess a sufficient bearing and deformability capacity up to the designed level of seismic protection.

4.2. Consolidation and reconstruction of the church St. Pantelymon in Ohrid

In the process of conservation and rebuilding of the St. Panteleymon Church in Ohrid, having in mind the importance and specific nature of the structure representing a historic monument classified in the first category and a structure of a particular national interest, it was necessary to design a building structure that will satisfy the stability conditions in the process of application of the conservation principles regarding shape, system and identification of materials, [6]. Seismic strengthening was provided in accordance with the previously developed and verified methodology. On the basis of performed analyses, the structural system of the church was defined, the horizontal and vertical steel ties were proportioned and a solution for the foundation of the structure was given**.** The principal structural system of the church consists of massive stone and brick masonry with incorporated horizontal and vertical steel ties, (Fig 20). The stone and brick masonry were constructed in lime mortar with characteristics selected through static and dynamic analysis. The church was reconstructed in 2001-2002, (Fig.7).



Fig. 7. The church during (left) and after reconstruction (right)

4.3. Reconstruction and seismic strengthening of the blown up church of the Holy Trinity in Mostar

The cathedral church of the Holy Trinity in Mostar built in the period 1863 to 1873 was shelled on June 7, 1992, while on the 15th June, the belfry was torn down and the church was put on fire and finally blown up. The remains of the church were cleared in 2005 (Fig. 8). Later, a decision on renovation of the church involving full reconstruction and maximum possible use of the existing preserved material has been made. Based on this decision, the Main Project on Renovation of the Cathedral Church in Mostar (architecture

and structure) has been elaborated. In this project, the bearing structure is designed to be constructed of massive stone masonry in cement lime mortar, [7].



Fig. 8. The original church and view of the torn down church

Upon getting evidence on the necessity of structural strengthening, variant solutions of strengthening have been proposed and analysed. Following the selection of the most adequate (from the aspect of stability and economy) possible solution, the stability of the strengthened structure under gravity and seismic effects has been analysed. Three general states of the bearing structure have been treated: (PS) – designed structure constructed of plain stone masonry; (HE) – strengthened structure by horizontal steel elements and (OS) – strengthened structure by horizontal and vertical steel strengthening elements, (confined masonry). The analysis has shown that, with its strength, stiffness and deformability characteristics, the designed structure (PS) as well as the structure strengthened by horizontal elements only (HE) do not satisfy the seismic safety criteria and are not in compliance with the most recent knowledge on behaviour of masonry structures exposed to gravity and seismic effects. Strengthening by use of horizontal and vertical steel elements (OS) turns the structural system constructed of plain masonry into a confined masonry system that exhibits a considerably more favourable behaviour under dynamic effects. All the performed analyses have shown that the designed strengthening system which complies with the conservation demands enables the necessary integrity of the structure at the characteristic levels and increases its strength, bearing and deformability capacity to the designed level of seismic protection.

The reconstruction of the cathedral church with the bearing system constructed of confined masonry started in May 2011 (Fig. 9) and nowadays is in its final stage, (Fig.10).



Fig. 9. Beginning of the church reconstruction in 2011



Fig. 10. Almost finished reconstruction of the church in 2017

5. Conclusion

Shaking table tests on the models in a realistic geometric scale are excellent method for research investigation of dynamic properties and bearing characteristics, verification of technology and method for repair and strengthening as well as for investigation of new technologies in historical monuments. The experimental evidence is helpful in proving the suitability of the design choices and the effectiveness of the applied consolidation system. In this way the original methodology using "ties and injection" for repair and strengthening of Byzantine churches was developed and verified through ample experimental and analytical investigation.

The developed and experimentally verified methodology was successfully implemented in the process of reconstruction and seismic strengthening of structures of the real historic monuments. The design process, which has been followed as shown in this paper, should emphasize the suitable way to follow when facing the complex problem of protection and conservation of a cultural heritage construction.

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STRUCTURAL REHABILITATION AND STABILIZATION OF SMEDEREVO FORTRESS TOWER 11

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Abstract: *This paper presents the structural rehabilitation and stabilization of Tower 11 of Smederevo fortress. The tower is tilted for about 1.8 m towards its outer edge, that is, towards the surrounding vertical walls that it was connected to. As a part of the structural rehabilitation and stabilization of Tower 11, it is planned to improve the soil properties with jet grouting columns and to secure tower's foundations, as well as to rehabilitate its parts above the ground. After completion of these works, conservation and restoration of the tower can begin.*

Key words: *Fortress, tower, damage, rehabilitation, rehabilitation projects, jet grouting.*

1. Introduction

The Smederevo fortress was one of the largest Middle Ages fortresses in Europe. The core was the triangular so-called Small town, at the top of the Big, again triangular town. This Big town was built right after the Small town, which was finished in 1430. The construction of Smederevo fortress began in 1428, ordered by Despot Đurđa Branković, and it was completed around 1439. The Small town had 6, and the Big town had 19 more, giving a total of 25 large towers, square shaped, over 20 meters high. Between the towers, there were walls about 4 m thick and about 15m high (Fig. 1). Since it also had a trench around the Great town, the Fortress belonged to a type of so-called water fortresses, although its foundations were not in the water. During its long history, the Smederevo fortress suffered significant damages in the world wars, from bombing and explosions. During the First World War and the crossing of the Austrians over the Danube, the fortress was bombarded several times and severely damaged. The last major destruction came in the Second World War due to an explosion of ammunition on June 5, 1941, when the Varoška gates, Tower 15 and walls towards Towers 14 and 16 were completely destroyed (Fig. 2.). Towards the end of the Second World War, the fortress was bombarded by Great Britain forces in the Allied bombing (Fig. 3.).

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Fig. 1. Smederevo fortress with Small town and Tower 11



Fig. 2. Destroyed parts of Smederevo fortress during ammunition explosion on June 5, 1941

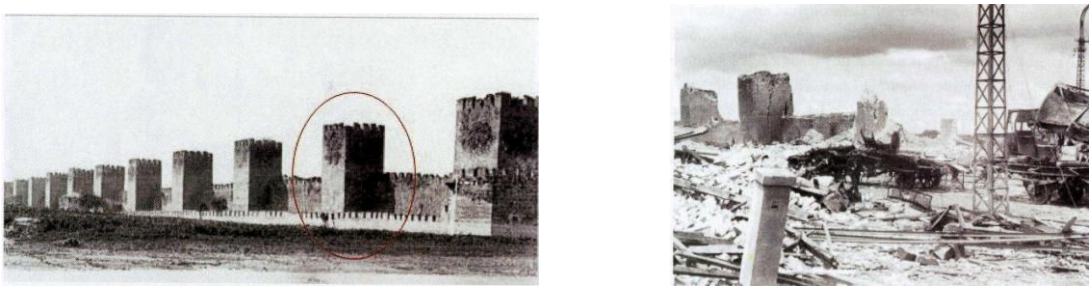


Fig. 3. Tower 11, before and after the Allied bombing on June 11, 1944

2. Current State of Smederovo Fortres and Tower 11

Various rehabilitation interventions have been carried out on the fortress over the past thirty years: the entire city was renovated, a timber bridge connecting the Small Town with other parts of the fortress, as well as some of the big towers. At the entrance to the fortress, around the so-called Varoška Gate (part between Towers 14 and 16) no rehabilitation interventions were performed, except for investigative archaeological works. In addition, no rehabilitation interventions were performed either on existing towers towards the city, nor on the walls between them. Tower 11 is located within the southern walls of the Big town, and it has the shape of the Cyrillic letter P. The dimensions of the outer base are 10.67 x 7.5 m. The height of the tower is 20.50 m in relation to the current ground level. The thickness of the tower walls goes from 3.75 to 4.07 m. In the foundation zone, the thickness of the tower walls is somewhat higher due to the counterforts. Foundations are at a depth of about 5.5 meters from the current ground level. The tower itself is in a very poor condition, and this primarily relates to its rotation towards the outside, which is about 1.8 m (Fig. 4.). What exactly caused this rotation of the tower is unknown. One of the possible causes is the explosion that occurred near the tower on the south side. Rotation (tilting) of the tower led to the separation of the tower walls from the walls of the ramparts, which resulted in a partial collapse of the rampart sidewalls. Apart from the mentioned tower tilting, there are also noticeable vertical cracks, as well as degradation of the walls, that is, damage to the elements from which they were built (stone, brick, mortar). These damages are a result of time - the influence of moisture, rain, frost, temperature change, traffic, etc. (Fig. 5.)



Fig. 4. Rotation (tilting) of Tower 11



Fig. 5. Degradation of building elements of Tower 11

3. Measures and Methods of Rehabilitation and Stabilization

Since Tower 11 of Smederevo fortress is a part of a cultural monument of exceptional archaeological and historical value, this implies that methods of strengthening this structure must be based, first of all, on achieving stability and security against the effects of external loads, but also these methods must be consistent with cultural and historical demands. When choosing a technical solution for rehabilitation of such structures, the value of the cultural property must be respected, as well as preservation of original material properties, as much as possible. In order to develop a rehabilitation and stabilization project of Tower 11, previous research was carried out, which included detailed inspection of the tower's current state, as well as geotechnical characteristics of the soil. Previous research included:

a) Detailed visual-macroscopic examination of the tower in order to determine the type and degree of damage to its walls;

b) Setting geodetic markers and measuring the inclination of the tower and its settlement;

c) Taking out the required number of samples of tower material in order to determine the type and quality of the materials used for building the tower and associated part of the rampart;

d) Preparation of the geotechnical study in order to determine the soil composition and groundwater level.

The geotechnical study was done based on an investigation borehole (diameter 146 mm and depth 10.2 m), as well as an experiment of cone penetration (CPT) with a penetrometer capacity of 200 kN to the exhaustion of the penetration force (Fig. 6).



Fig. 6. In-situ investigation of Tower 11 foundation soil (left: CPT, right: borehole drilling)

Based on all previous research, two opposite solutions of rehabilitation and stabilization methods have been analysed:

1. Correcting the tilting of Tower 11 and bringing it into a vertical position;
2. Keeping the tower in an inclined position and securing it from further inclination.

Detailed analysis of the possible rehabilitation solutions included all previous activities that were conducted in order to find the optimal solution and consultation with the representatives of The Institute for the Protection of Cultural Monuments of Serbia. Based on this, it was decided that the rehabilitation and stabilization project be done with Tower 11 kept in an inclined position and secured from further inclination.

This solution involved developing two parts of the project, one relating to the soil stabilization in the tower's foundation area and the other relating to the structural rehabilitation of the tower above the ground level.

3.1. Technical solution for soil stabilization (improvement)

According to geotechnical field investigations to the depth of 10m, the terrain profile consists of the following layers (Fig. 7.):

- 0,0 m - 4,3 m embankment,
- 4.3 m - 7.0 m dust clay,
- 7.0 m - 10.0 m fine to medium sand.

At the time of the survey, the groundwater level at the depth of 2.8 m from the ground was registered in the field.

For the estimated foundation depth, foundation loading, and the characteristics of the terrain at the site of the tower, it was concluded that it is necessary to repair the soil below the foundation of the tower in order to reduce settlement of the structure. After examining the problem, and in order to increase the load-carrying capacity of the soil, a method for improving soil strength using a technology known as jet grouting was adopted.

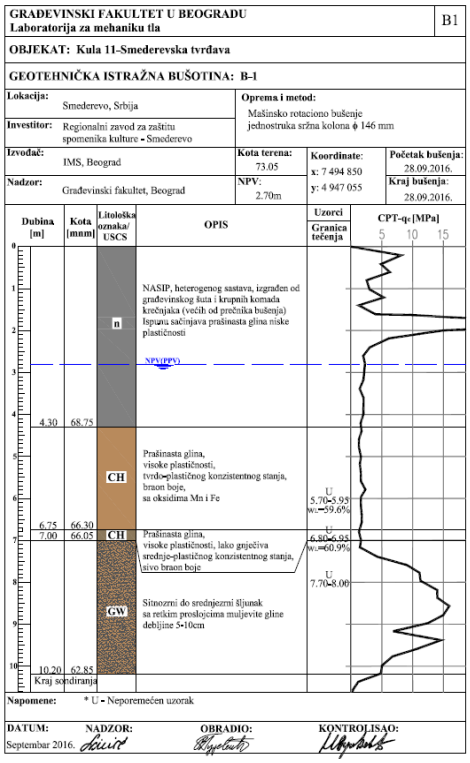


Fig. 7. Geotechnical investigation borehole

Jet grouting is a modern technology of geotechnics that from the very beginning of development was used exclusively in the function of improving the mechanical characteristics of the soil, that is, soil reinforcement. Jet grouting can be applied in all soil types, from clay to pebbles, independent of granulometric composition and porosity. Jet grouting includes drilling to the required depth and high-pressure injecting of cement milk (suspension), which is mixed with degraded soil (due to high pressure) and soil particles (aggregate), thus forming a column whose strength and/or permeability are improved compared to the existing surrounding soil. The strength of these columns depends on the type of existing soil, as well as on the parameters under which the work is carried out - jetting. In this case, the ground level is 73.12, and the level of tower's foundation bottom (according to the current state project) is 66.94, therefore it is planned to extend the foundations to the level of 69.81 by jetting, in the form of counterforts. Jet grouting is to begin with smaller diameter columns Ø50cm by mono-fluid technology at an axial distance of 50 cm, with a depth of 9.50 m, around the front of the tower (this includes works on the front and on the sides of the tower). Columns are made with a distance of at least 2.0 m between consecutive ones. In each jet grouting column, an anchor rod T76 is inserted (Fig. 8.). Anchors are type SDA, so that during installation the tube gets injected also in order to achieve better connection with the existing cement structure in the jet itself. After finishing of the columns, the reinforced concrete capping beam 50 x 50cm is formed, in order to tie in the columns. Then, two-fluid jetting of columns with a diameter Ø100 cm is performed at an angle of approximately 10°, so that already constructed counterforts are not damaged. These columns are only constructed below the foundations of the fortress, so that the soil characteristics are improved without any soil excavation and any interventions that could damage already weakened structure. From the rear (inner) side of the fortress, there is not enough space for the machine which makes the dual-fluid columns, therefore columns with mono-fluid technology and Ø60 cm diameter, performed at an angle, are provided here (Fig. 8.).

3.2. Structural rehabilitation of Tower 11 above ground level

As a part of the structural rehabilitation and stabilization of Tower 11, besides improving the soil and securing its foundations, the rehabilitation of the part above ground is planned, so that upon completion of these works, based on specially designed technical documentation,

conservation and restoration works on the tower can begin. Within the tower, both on its front and on the sides, there are cracks spreading from the ground and practically to the upper levels of the walls to the left and to the right of the tower. Observed cracks, which are predominantly vertical, but some are also inclined, exist on all sides of the letter "П" - shape of the tower's cross-section (Fig. 5.). Considering the described state of the cracks on the vertical walls of the tower, the intervention on it also means that within the letter "П", at four height levels on each of its vertical walls, horizontal boreholes Ø80 mm are made and that through these boreholes tension elements Ø56 mm with appropriate threads at the ends are positioned. The positions of tension elements, as well as the details related to their construction, are shown in Fig. 9. As it can be seen, the ends of the tension elements in question will be supported on concrete "pillows" with 20 cm thickness (MB 20). The "pillows" need to be reinforced with reinforcing meshes Q 335 throughout their thickness both on the front and towards the stone walls.

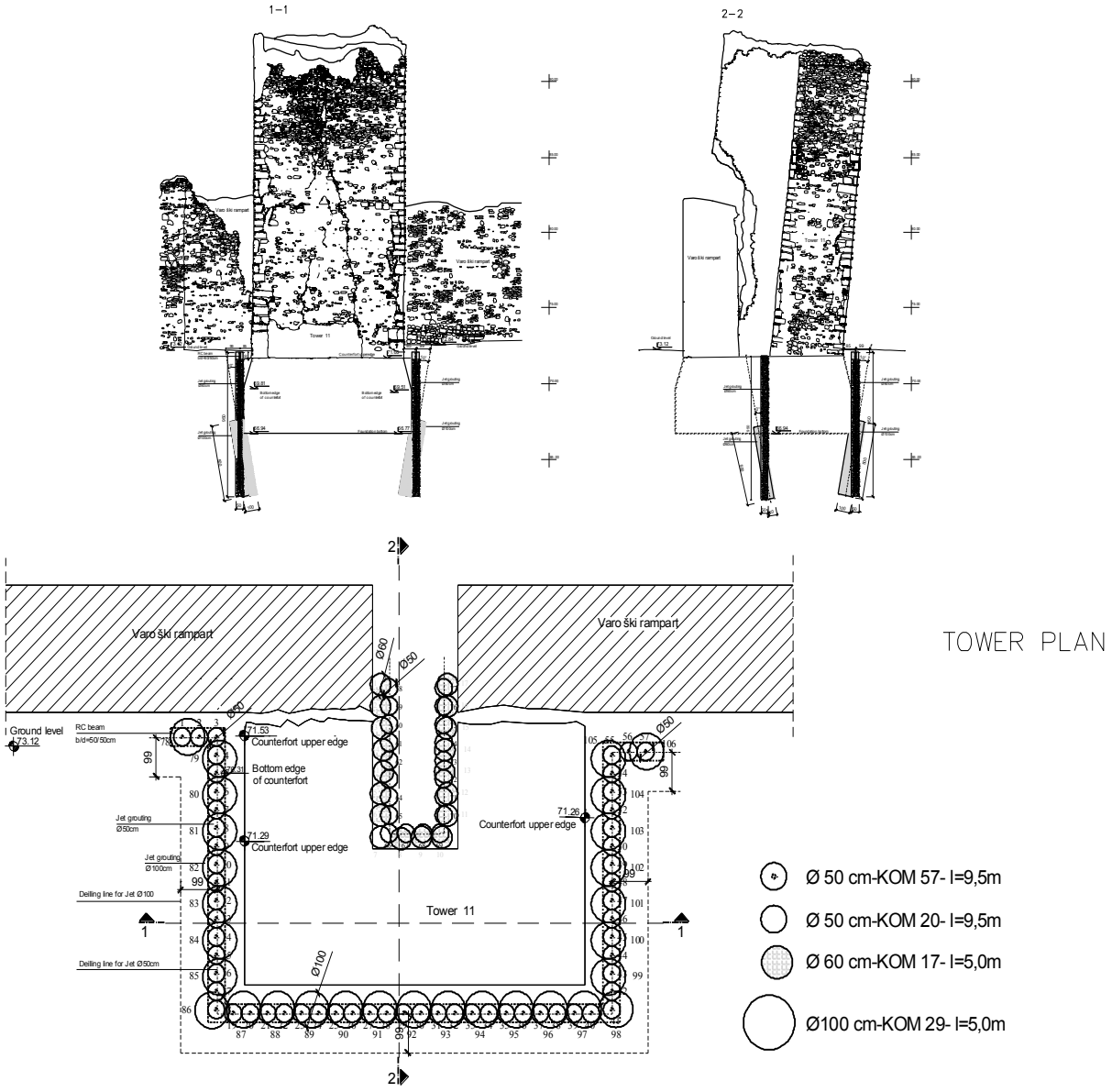


Fig. 8. Rehabilitation plan for Tower 11 foundations using jet grouting

Afterwards, in the free space between the tower and the ramparts, it is necessary to build a RC wall with a thickness of 30 cm, which should be double reinforced with reinforcing meshes Q 335. For double-sided covering of this wall with stones, ribbed rebars Ø8 mm should be embedded in 50 x 50 cm raster. These rebars should be anchored in stone walls that will be added later, at least 25 cm. Concrete and stone coating walls should be constructed, first of all for aesthetic reasons, so that the tension elements would not be visible, but also for protection from atmospheric influences, i.e. corrosion. Described elements should be included throughout the height of the walls from ground to the upper levels, both on the left and right. In addition, on the side walls of the tower, as a part of overhangs that resulted from separation of the tower from the walls on the left and right, two additional tension elements Ø56 mm should be added. With these elements, global stability of overhangs will be ensured. As part of structural rehabilitation and stabilization of Tower 11, and on the proposal of the representatives of the Investor, the steel bridge at the level of walkway on the rampart was also designed. The bridge was based on a similar structure carried out in the towers of the Manasija Monastery, as Investor proposed. The load-carrying structure of the bridge is designed from a box steel profile in the form of two longitudinal girders 140 x 80 x 4 mm and three cross girders 100 x 80 x 4 mm. On top of these, a walkway of steel gratings with dimensions of 40 x 40 mm is positioned. The load- carrying structure of the bridge fence is also designed from box steel profiles 40 x 40 x 3 mm, while the fence web is made of round steel with a diameter of 12 mm. The entire construction of the bridge, including the fence, should be protected against corrosion by hot-dip galvanizing prior to installation.

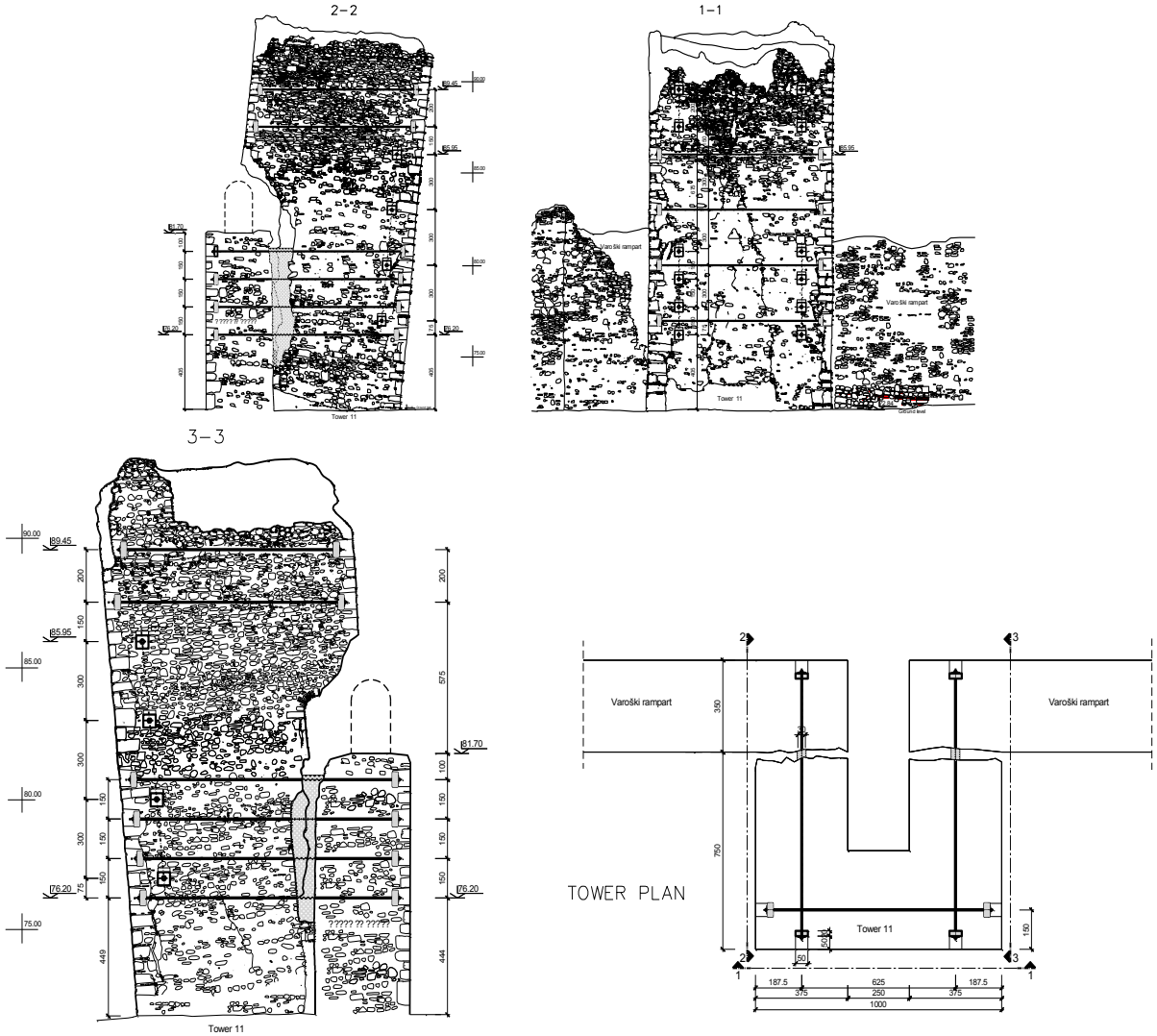


Fig. 9. Tension elements for strengthening of Tower 11

THE MEDIAEVAL CHURCH OF SAINT ATHANASIUS, BOBOSHEVO INTERDISCIPLINARY INVESTIGATION, CONSERVATION, RESTORATION AND EXHIBITION, 2017

F-DEKO LIMITED, KYUSTENDIL

HristoGanchev, architect; Rumen Spassov, archaeologist; Ivan Geshanov, PhD(Eng); IliyaBorisov, restorer; VladislavKolarov, architect; YuliyFarkov, architect

ABSTRACT

The late-mediaeval church of St. Athanasius is located on a terrace to the south overlooking the town of Boboshevo. According to the ktetoric (founder's) inscription on the south wall, the church was built in 1590 over the remains of late-Roman (Byzantine) buildings. It has been declared a monument of architecture and construction, and a monument of architecture and artistry, of national significance. The "biography" of this cultural treasure reveals several building periods. For many years, no support or restoration works have been carried out; the frescoes of the church are in a poor state, covered in a thick layer of soot and dust.

For this reason, the Board of the church retained F-DEKO Limited in 2016 to prepare an investment project for: Reconstruction and repair to a building of religious significance, and improvement of the surrounding spaces of 5the cemetery church of St. Athanasius. Boboshevo. The project was agreed and approved by the Ministry of Culture.

The client – the Church Board – is making an application under the European programme for development of rural areas 2014-2020, Measure 7 "Primary services and regeneration of villages and rural areas", sub-measure 7.6 "Surveys and investments connected with maintenance, restoration and improvements to cultural and natural heritage in rural areas".

The investment project was assessed and approved for financing under the programme. Work was commenced on implementing it in July 2017.

The present article is the work of an interdisciplinary team and deals with the issues of archaeological, architectural, constructional and art investigation, the conservation and restoration work begun, and socialisation.

The SkrinoGorge of the ancient river called the god Strimon (today "Struma") is one of the five rocky gorges mentioned in Ancient Greek myths and legends in connection with one of the Twelve Labours of Hercules: the scattered Cattle of Geryon demanded by Eurystheus. With the fury of a demigod, Hercules vengefully squeezed the river god with enormous boulders between Mount Skombros (Vitosha) and the Aegean Sea. The small Bulgarian town of Boboshevo is sited at the southern end of the gorge, which is filled with late-Roman-era (Byzantine) and mediaeval settlements, forts, and ecclesiastical buildings, forming part of the Holy Mountain of Velbazhd.

- Skrino Village – the mediaeval monastery of the Holy Father, the birthplace of the patron saint of the Bulgarian people, Saint Ivan of Rila
- Pastuh Village – a late Roman-era settlement, mediaeval churches: Saint George the Martyr, Holy Virgin, Saint Ivan, Holy Trinity, Saint Demetrius

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- Vukovo Village – a late-Roman and mediaeval settlement with the ruins of the churches of St. George the Martyr, Archangel Michael, Holy Virgin, and the church of the Ascension of Christ (known in Bulgarian as Holy Spass)
- Marvodol Village – the mediaeval church of Saints Cosma and Damian
- Town of Boboshevo – the mediaeval monastery of St. Theodore, the church of St. Athanasius, the church of the Holy Virgin

A little piece of the gleaming nature of the Valley of the Holy River and the Road overflowing with relicts of the cultural heritage of Bulgaria and the Christian model of civilisation over the course of fifteen centuries.

These introductory notes are a prelude to the main subject of this study: **The mediaeval church of St. Athanasius in Boboshevo – an interdisciplinary investigation, conservation, restoration and presentation**.

The late-mediaeval church of Saint Athanasius the Great,Patriarch of Alexandria in Boboshevo was declared a monument of architecture and construction of local significance (State Gazette issue 77/1972), and a monument of architecture and artistry in the National Significance category (State Gazette issue 38/1972).

The ecclesiastical ensemble is built on a terrace above the town to the south in the so-called GornaMahala. According to the ktetoric (founder's) inscription on the south wall, the church was built in 1590 over the remains of late-Roman buildings.

The following building stages are evident in the biography of the cultural treasure of the church of St. Athanasius:

- **Single-nave, single-apse church**
- The church is located on a wide terrace in the town cemetery of the so-called GornaMahala part of Boboshevo. A small, single-nave church with a semi-circular apse with a recessed front façade.

According to the partially legible ktetoric inscription on the south wall, the church was built in 1590 or 1600. Up to 29 October 1862, it was the main church for the village. Its length is 9.16 m, width 4.90 m (external dimensions). To the left of the apse there is a votive niche to the patron, and to the right – a diaconicon. On the north wall there is a small, semi-circular niche – a font. The walls are stone, 0.70 m thick. The church was entirely frescoed inside, and the western façade was frescoed on the exterior in accordance with established canon, and it was here that people from Boboshevo and neighbouring villages would attend the liturgy. Following the construction of the church of the Holy Virgin, the church of St. Athanasius was turned into a cemetery church.

The church frescoes are in a poor state, covered with a thick layer of soot and dust. No stabilising or restoration work has been done.

The church bears the name of Saint Athanasius, or Saint Athanasius I the Great, Patriarch of Alexandria (293/7 to 373 CE) – an Egyptian theologian and church father, and Patriarch of Alexandria from 328 to 373 CE. The Church celebrates the saint's day on 18 January and 2 May.

- **Single-space service building**, built to the west of the church in 1792
- **Narthex(porch),** built in the nineteenth century against the west façade of the church
- **Church school**built against the north wall of the church and the narthex
- **Refectory (exonarthex, or memorial hall)**built in 1926-1927



St. Athanasius Church. East wall and apse

The archaeological investigationwithin the site of the ensemble (church, porch, refectory, and the surrounding terrain north and south of the church) by boring according to type and covering a total area of 89 sq.m., of which: 40 sq.m. within the church; 28 sq.m. in the area of the refectory; and 7.3 sq.m. in Borehole 1 (south) and 13.7 sq.m. in Borehole 2 (north-west). The maximum thickness of the layer showing site usage is 2.60 m.

1. The information from the research shows that the terrain on which the late-Mediaeval church was built had been inhabited as early as the **late-Roman (Byzantine)period (4th to 5th centuries CE).**During this period, an architectural complex was built, which covered an area greater than 100 sq.m. The schematic of its layout describes a storage area, whose foundations were built of broken stone and earth with a width of 0.55 m, which are preserved to a height of 1.04 m. The western and eastern walls of the room (no other walls are preserved within the boundaries of the investigated terrain) are 7.20 m apart. The room contains *dolia* for storage of food and products, two of which are preserved in situ. One of them is 1 m tall with a maximum diameter of 1.12 m, and the other: height 1.70, diameter 0.82 (fig. 1). A large part of the building falls outside the boundaries of the investigated terrain. This fact, together with the fact that the part discovered is not sufficiently informative, does not give grounds to judge the layout of the complex and its type differentiation.
   Among the materials found during the investigation of the Byzantine layer, two bronze coins of the emperors Constantine the Great (306-337 CE) and Valens (364-368 CE), small fragments of clay pottery, and a large quantity of fragmented construction ceramics.
2. **The church of Saint Athanasius**is situated within the Byzantine building, but its construction does not conform to the layout of the earlier building work. Three reconstruction phases of the church have been identified (synchronous in the naos and altar). They are accompanied by three phases of elevation of the

floor levels (fig. 2) – from its initial construction up to 1926, when the last major repair works were carried out on the church and its attached buildings.

*First floor level*is marked by a thin edge of white mortar stamped down with tiny stones. The frescoing of the church starts from this level. It is located at a depth of 0.45 m compared to the threshold of the western entrance.

*Second floor level*is formed in clay, bricks and flagstones. A coin dating from 1782 was discovered in the clay rendering. The flagstones have the following dimensions: 58 x 45 x 4 cm; 80 x 40 x 6 cm; 39 x 26 x 4,5 cm; 47 x 47 x 2 cmetc.

*Third floor level*has white mortar, with thickness of the rendering 2,5 – 3 cm. The mortar contains small stones and organic additions. The stratigraphic position of this level compared to the previous one gives grounds to conclude that it was formed no earlier than 1782.

*Modern floor level,*cement rendering, dating to 1926.

It was noted that, in the case of all the floor levels, there is a slightly apparent inequality in the level of the naos along the west-east axis reaching 9 cm. The floor level of the altar is slightly raised (3 cm) compared to the naos at its eastern end.

The original*threshold of the western end* of the church was formed by a massive stone block with a thickness of 0.26 m and a length of about 1.42 m. At this stage of the investigation, its width cannot be ascertained.The entrance had one step. The step, located 0.17 m lower than the threshold, was built up with large, medium, and small-sized stones held together with clay.

3. At a distance of 8.35 m west of the churchthere is a**site of a room dug into the ground (a separate building?) with a hearth.**A small part of it has been preserved (fig. 3). Two walls of this room have been cleared: the western and the southern. The remainder were destroyed during the construction of the refectory in 1926. The western wall has a length of 3.10 m, and the southern has been preserved to a length of 1.10 m. They are constructed of large and medium-sized rough stone held together by earth. Their width is 0.57 m and they have been preserved to a height up to 0.90 m. The construction is single-faced. In the middle of the western wall, a niche has been formed with a width of 0.80 m and a depth of 0.34 m., which is part of a hearth with overall dimensions of 0.80 (west-south) x 0.72 m.

The small preserved part of this room, and the lack of typifying discoveries in its filling, do not permit a judgement to be made about its layout or its functions. The situation of the newly-discovered building also cannot be ascertained chronologically at this stage of analysis of the terrain data.

4.The archaeological investigated provided an opportunity to locate the**church school**(information about this is very scarce), and to establish its layout and dimensions. It comprises a rectangular building with interior dimensions 12.50 m (west-east) x 3.15 m, attached to the north wall of the church and the porch. The walls were constructed in stone and earth with a width of 0.63 m, and have been preserved to a height of 0.35 m in the superstructure. Their interior faces have been plastered twice with mortar. In order to create a link between the church and the school, an entrance was opened in the north wall of the church forming a doorway with a width of 1 m. The difference of 0.58 m between the floor levels of the school and the church (third floor level with white mortar) leads to the assumption that this is a one-step entrance.

An analysis of the terrain data gives grounds to assume that the school was built no earlier than 1782.

5. Necropolis. West of the church, below the refectory built in 1926, are sited four graves dating from the eighteenth to nineteenth century (fig. 4). The burials comprise bodies laid supine in ordinary grave pits oriented west-east. There are no remaining grave goods. The graves are located at a depth of 2.01-2.14 m compared to the modern ground

surface west of the church. The presence of these graves is an indication of the presence of a Christian necropolis dating to that time. Its area cannot be ascertained, due to the fact that graves are still being dug around the church today.

The archaeological excavations in and around the church of St. Athanasius in Boboshevo during 2017 are the first investigations of their kind of a site in the Ruen/Skrino Gorge. Together with the interesting information that this investigation provides about the particular monument, it also leads to the belief that a similar approach needs to be taken in socialising the remaining mediaeval and late-mediaeval sites in the area too.



Late-Roman/Byzantine building: floor levelsin the naos of the church.
S.R. Spassov



Stratigraphic position of the floordolia in the storage room
S.R. Spassov



Fragment of a single-space church
S.R. Spassov.



Graves in the necropolis west of the church; Building with hearth west of the church.
S.R. Spassov.

Architectural investigation

The church of St. Athanasius is a typical example of the archetype of ecclesiastical construction in the sixteenth to seventeenth century. The church has a single nave with a vaulted roof. This type is widespread in the Bulgarian lands during the Ottoman period.



View towards the apse



Refectory with a gallery facing into the church

- Interventions

During the nineteenth and twentieth century, the composition was extended westwards with an open porch (narthex), and a refectory (exonarthex), with a gallery facing into the church.

The following conclusions may be drawn from the analysis with regard to the sources of information in connection with the preserved authenticity of the existing structures of the monument:

- materials and substance – the main structure of the church has been preserved, and a two-layer covering has been built over the vault of the church; the entire roof of the church, the porch and the refectory is covered in Marseilles roof tiles; heat-insulation has been mounted on the façades of the entire church, the porch and the refectory using mineral-based rendering. Everywhere the flooring is modern cement;



Two-layer covering over the vault and heat-insulation with mineral rendering



Preserved original stone cornice.

- Tradition and techniques – due to the radical interventions carried out during recent years, a precise characterisation of the original decoration/shape of the façades and roof cannot be made;

- Symbolic/metaphorical sources of information: floors – bricks and flagstones; seventeenth-century church façades – stonewalling with flush pointing (the exact characteristics will be ascertained following removal of the heat-insulation);

- Condition and pathologies

Technical condition

- seventeenth century church – the structure of the walls and the vault are in a relatively stable condition, although a crack has been noted in the keystone of the vault; capillary-action (rising) damp in the walls has been noted due to a cement-rendered floor having been laid;



Cracking of the vault keystone.

- refectory–the non-masonry construction of the walls and roof is in a critical state



Refectory

Theoretical grounds for the methodological approach

- Regarding the church

The accepted methodological approach is regulated in the Cultural Heritage Act as "reconstruction in an authentic way" (§ 4, p. 15 Supplmental provisions of the CHA), namely: "partial rebuilding on the basis of graphic, textual, photographic documentation and other sources that provide information about the form and design, the materials and the substance, the use and function, tradition and techniques, location and situation, spirit and mood/feel and other internal and external factors". The proposal is in accordance with the directives of the Venice Charter (1964-1965), more precisely: "[the] aim [of restoration] is to preserve and reveal the aesthetic and historic value of the monument and is based on respect for original material and authentic documents", and is in complete synchronicity with the philosophy and definitions of the Nara Document on Authenticity (1994), namely: "Depending on the nature of the cultural heritage, its cultural context, and its evolution through time, authenticity judgements may be linked to the worth of a great variety of sources of information. Aspects of the sources may include form and design, materials and substance, use and function, traditions and techniques, location and setting, and spirit and feeling, and other internal and external factors. The use of these sources permits elaboration
of the specific artistic, historic, social, and scientific dimensions of the cultural heritage being examined".
This document defines sources of information as "all material, written, oral and figurative sources which make it possible to know the nature, specifications, meaning and history of the cultural heritage."

- Regarding the surrounding area

The principle of "integrated conservation" is accepted, which "includes various measures aimed at enshrining cultural heritage as part of a corresponding environment created by Man and Nature, where the usage and adaptation of the heritage sites are for the needs of society (§ 4, p. 10 Supplmental provisions of the CHA). This regulation is in the context of the doctrine of sustainable development within the spirit of the following documents: European Charter of the Architectural Heritage; The Declaration of Amsterdam; Congress on the European Architectural Heritage; Resolution 76 (28) of the Committee of Ministers; the Grenada Convention; Guiding principles for development of legislation and management systems for cultural heritage; Leading principles for sustainable territorial

development of the European continent, 2000, Recommendation (2000) 1 of the Committee of Ministers.

Proposal for interventions in immovable cultural treasures.

On the basis of the quoted theoretical, methodological and legal regulations, the proposed project envisages the following:

- Regarding the church: removal of the thermal insulation cladding from the façades; restoration of the authentic height of the roof and coverage in stone tiles; restoration of the original formation of the façades – stone walling with flush pointing; restoration of the authentic late-mediaeval flooring of bricks and flagstones; restoring the authentic dimensions of the windows, walling up of other openings;
- Regarding the porch and the refectory (narthex and exonarthex): reconstruction of the collapsing non-masonry structures, walls and bell-tower, presentation of the structure and architecture of the masonry walls; improvement of the conditions for usage; an architectural solution to ensure optimal opportunities for presentation of the seventeenth-century church, and enrichment of its impact on the sacred space.
- Regarding socialisation:
  The project offers an optimal socio-cultural realisation through restoring the ritual functions and presentation of the artistic and architectural treasures.
- On the surroundings
  The landscaping solution for the surroundings is aimed at restoring the general urban nature of the church, and also to provide an opportunity for visits by specialists and satisfying the needs of tourist interest. In this sense, all the landscaping works must be carried out under the observation of an archaeologist.
- On the structural policy and integrated conservation
  The project envisages various measures aimed at showcasing the qualities of the cultural heritage site, the archeological remains, and the significance of the cemetery for the town. The restoration of the functional meaning of the church gives an opportunity for organic connection of the heritage with their restored social, spiritual and ritual meaning as sustainable elements of the regional community.
  In connection with the planned landscaping actions including beneath ground level in the interior, monitoring on the part of an archaeologist during implementation of the building works must be ensured.
  The area of the property is 25,982 sq.m; enclosed area 143.56 sq.m; real enclosed area 143.56 sq.m

Structural Part

1. Altereddesign solution

1.1. Justification for changes

The alterations in the design documentation are based on the results of the archaeological, architectural and artisanborings during demounting of narthex structural elements (porch) and exonarthex – including the memorial hall (refectory) built at the beginning of the 20th century, that covered part of the decorative system of the western façade of the late-mediaevalchurch with the patron niche and biblical scenes that include rare paintings. During the dismantling, irreversible destruction of the fenced walls of the narthex works was found. They were originally built with stone masonry with a solution of

earth and plaster–double-faced, clay, whitewashed, without stone foundations which were found in the process of destruction.

This necessitates their rebuilding. In order to ensure complete perception of the frescoes on the western façade, a partial alteration of the roof structure of the narthex was undertaken in such a way that itdoes not obstruct the view towards the decorative system and the biblical narrative. This altered architectural and constructive solution makes it possible to appreciate the entire Western façade of the late-mediaeval church of St. Athanasius and the gallery in the hall, where it is planned that the museum chamber exhibition is shall be displayed–showing a short history of the mediaeval town of Lower Sateska (Boboshevo), the results of the archaeological investigation - from the late Roman period (Byzantine), artefacts from the necropolis, an art-history analysis of the iconography, the architecture of the church, and the conservation and restoration works, as well as a wall panel entitled "Memory" with photos of ancestorstaken from the personal archives of the people of Boboshevo..

During the archaeological inspection of the site of the exonarthex, it was established that the construction of the western wall of the memorial hall and the gallery above it,together with the laying of the groundwater drainage system, would affect new burial sites,and therefore a partial structural modification of the foundations and basement walls of the memorial hall will be required.

1.2. Changes in the design solution for the church

The changes in design documents are made due to technical information based on removal of the narthex and partial uncovering of the existing wooden two-storey secondary roof structure.



Fig. 1. Condition of the exposed roof.

The study of the exposed roof top at the point of support by the *voussoirs* of the supporting stone walls (at the base of the vault) shows that implementation of the projected reinforced concrete lintels in slots in the stone crosses is very risky with respect to the integrity of the supporting walls. The study of the revealed roof top at the footsteps on the

supporting stone walls (at the base of the vault) shows that the implementation of the planjned reinforced concrete belts mortises in the transverse stone walls is very risky with regard to the integrity of the supporting walls.Therefore, the solution decided upon is to anchor the new steel frame by means of end steel beams, bent directly towards the supporting stone walls and anchored by steel studs. For the same reason, the new tensile element will be anchored to the anchor holes, made specifically for the purpose at the sides of the longitudinal walls.

These changes are shown in Figure 2.

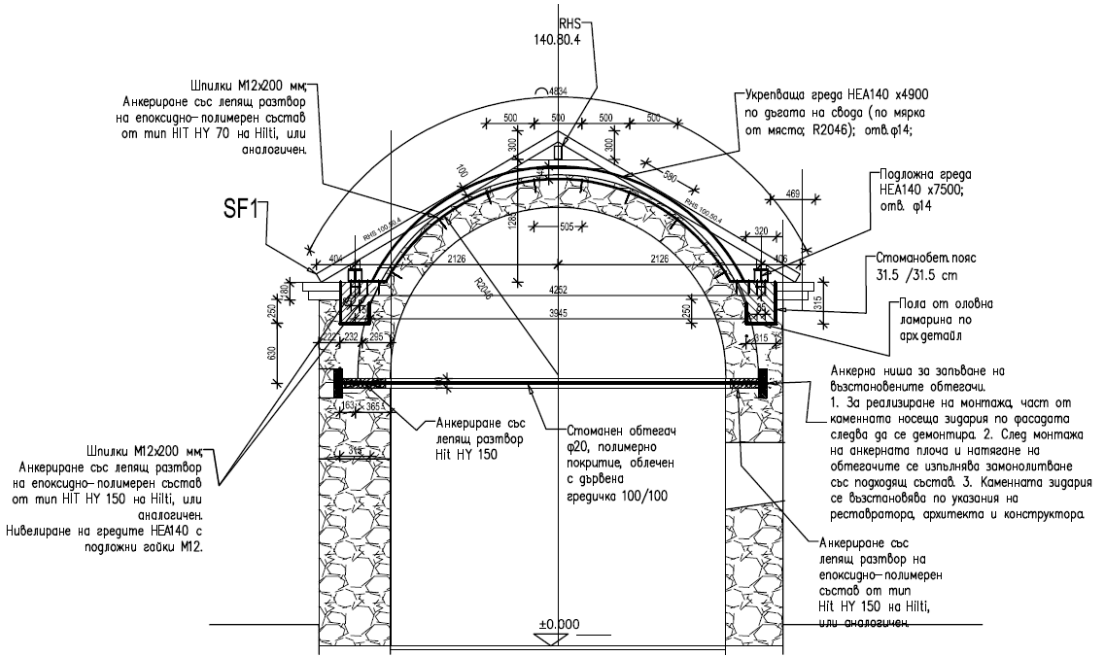


Fig.2. Changes in reinforcement of the new roof structure and anchoring of the tensile element.

1.3. Change in the design solution for the narthex

As mentioned above, the surrounding walls of the narthex, built in stone masonry and earth mortar, proved to be seriously damaged and at high risk of collapse, and these walls are in a virtual state of emergency. It was judged that they could not be deemed safe for exploitation in the future, which is why they were removed.

For their rebuilding,a load-bearing reinforced concrete wall will be built in the new walls of the narthex. The reinforced concrete wall will be supported on reinforced concrete foundations with sufficient width to ensure the seismic resistance of the newly-built narthex –including monolithic walls, a light wooden superstructure and a wooden roof. On both sides of the reinforced concrete walls, a stone lining with details, identical to the original wall, will be placed. A typical cross section is shown in Figure 3.

Close to the western façade of the mediaeval church, a specific solution was chosen to partially modify the roof structure of the narthex so as to "not obstruct the view of the decorative system and biblical narrative". It was decided that a complete unification of the side windows is needed, as well as the fitting of steel and wooden girders, and the steel elements will be made of ropes of stainless steel with 8 mm diameter. The design solution of the steel-wood girders is shown in Figure 2, and the typical cross-section is shown in Figure 4.

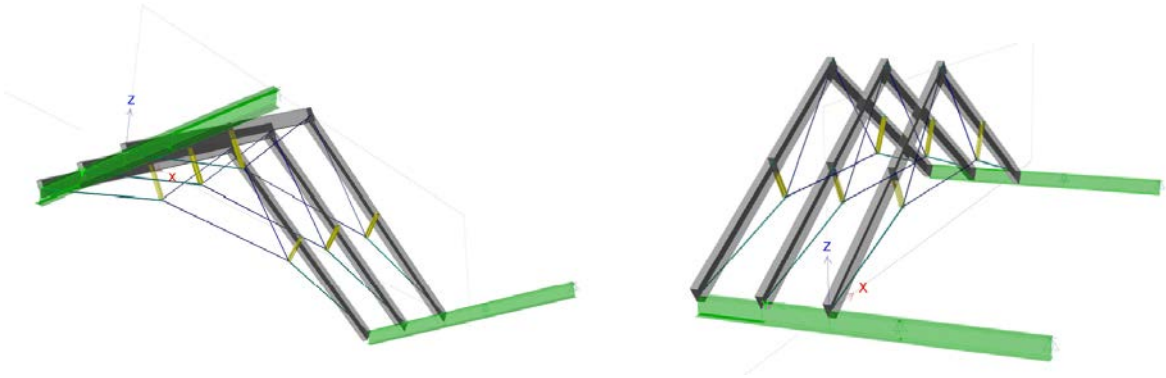


Fig. 2. Steel-Wood girders right next to the western façade.

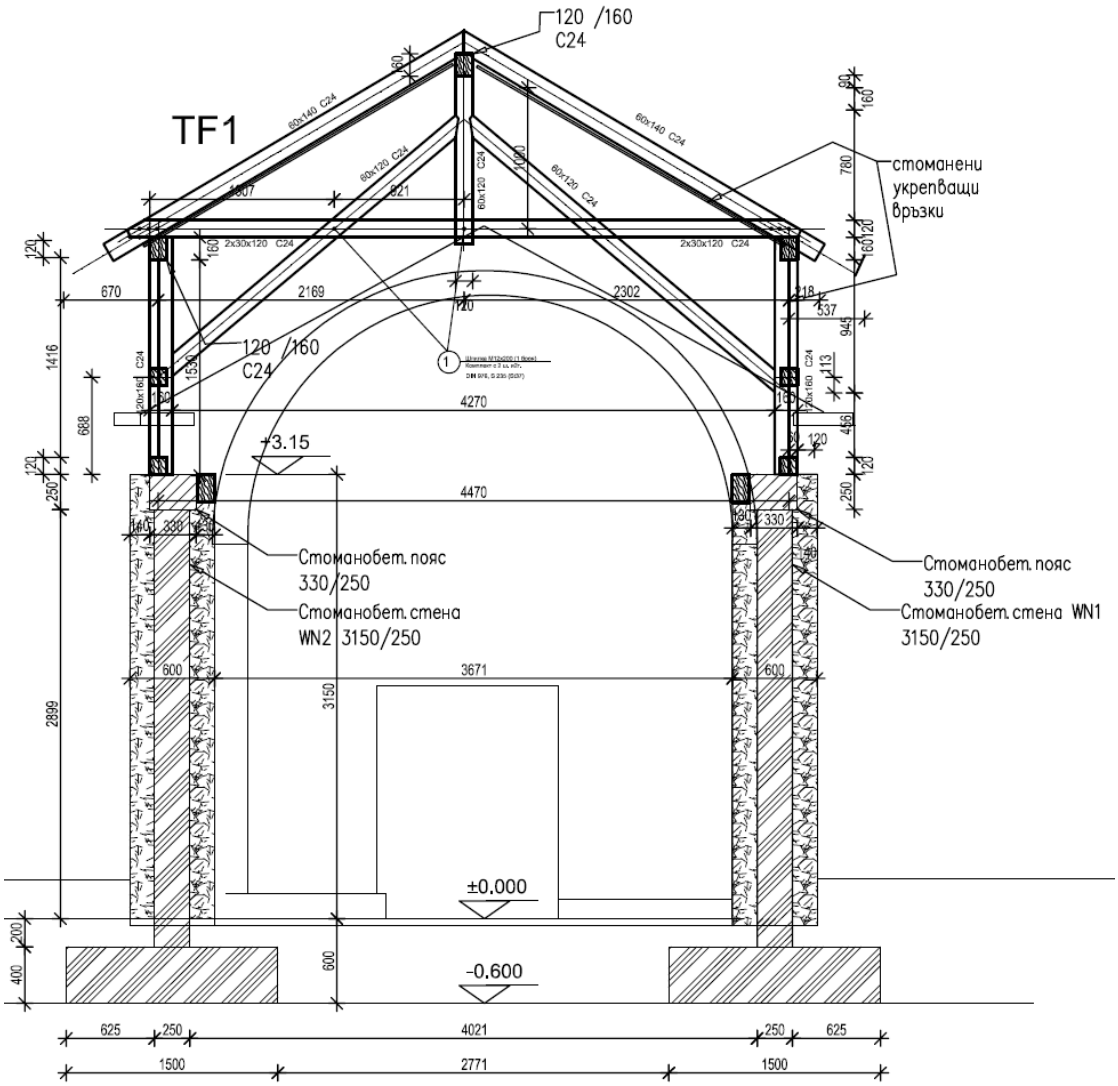


Fig.3. Typical section of the new narthex structure.

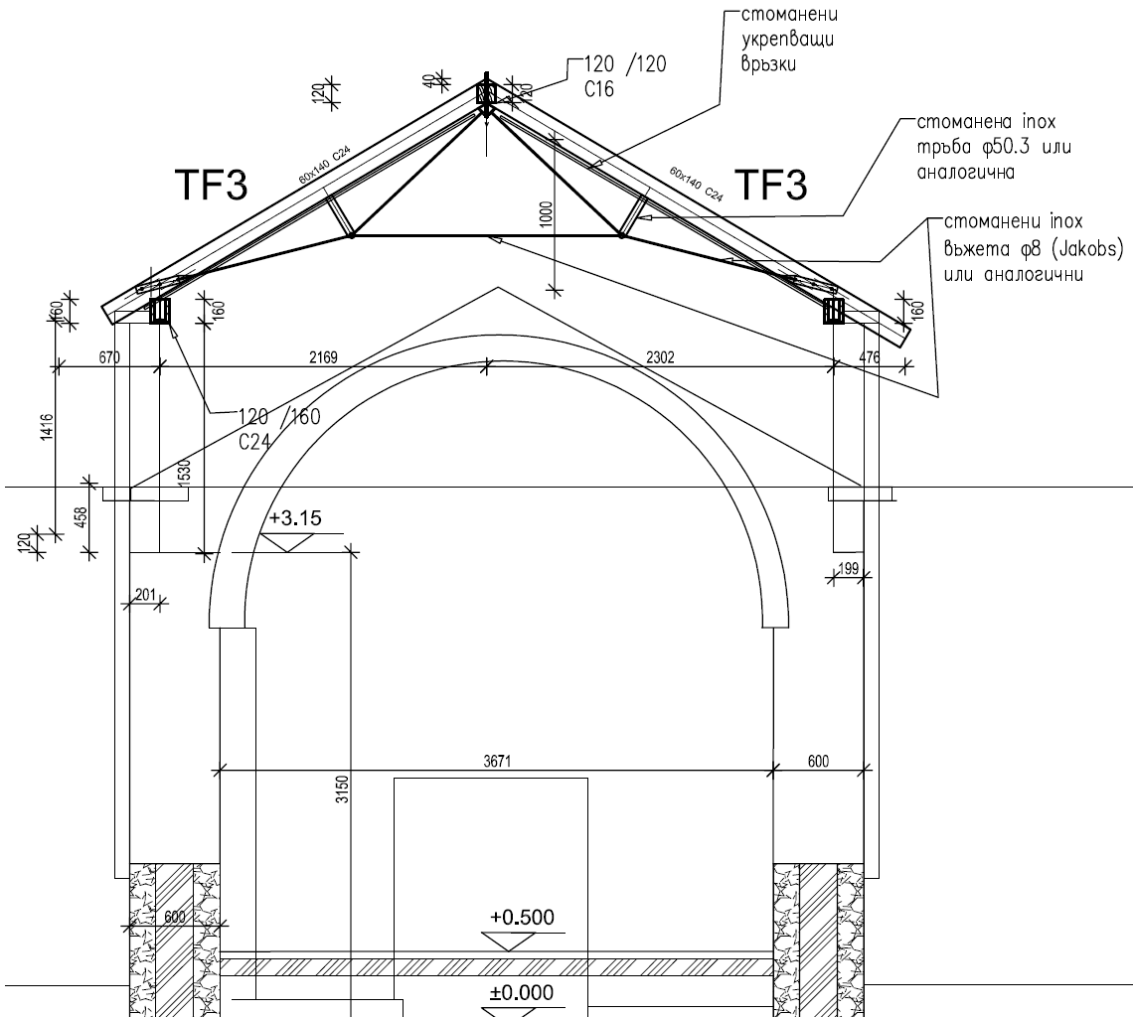


Fig. 4. Typical section of the new narthex structure on the western façade of the church.

Figure 5 shows a fragment of the changed design solution for the façades of the Narthex near the western façade of the Church.

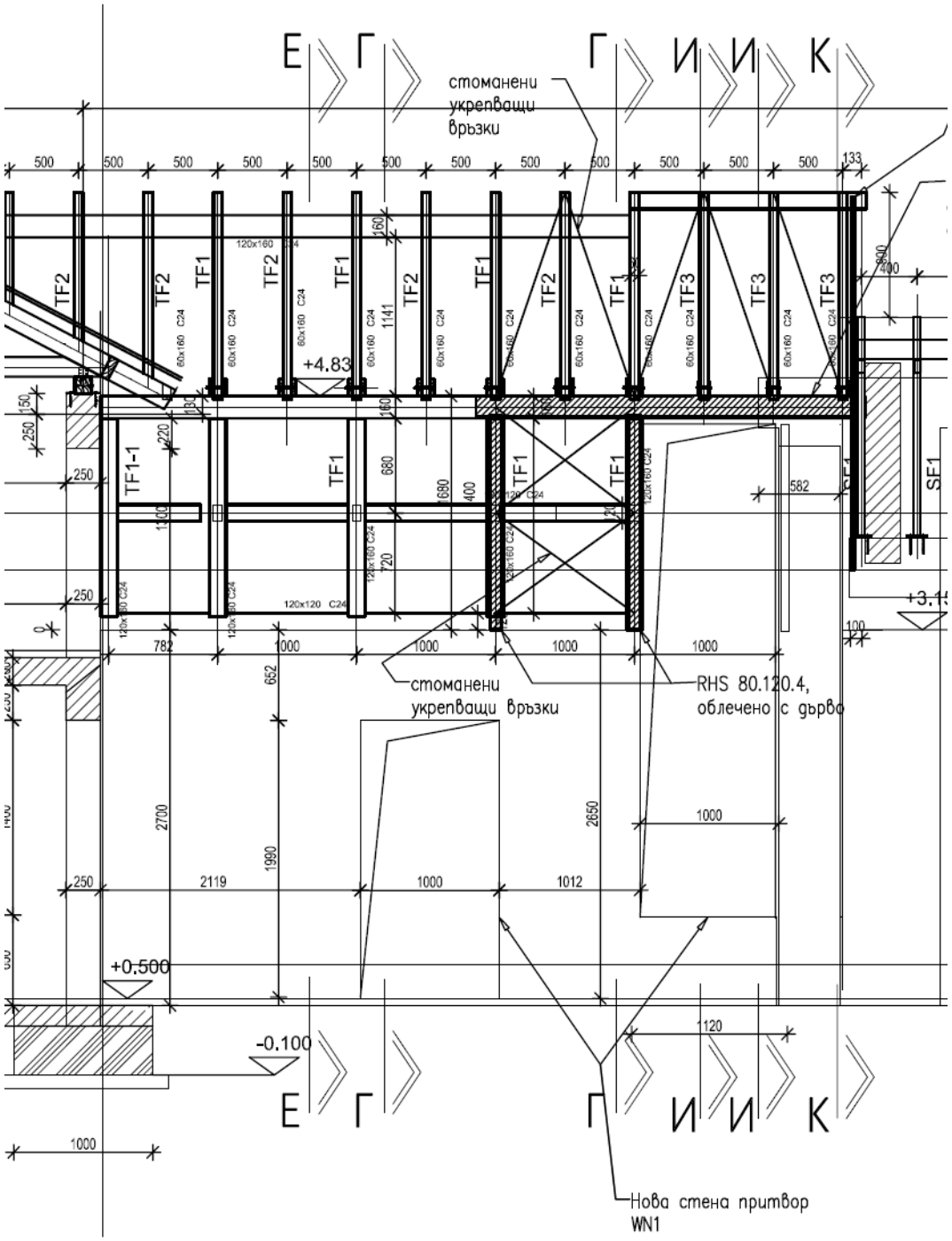


Fig. 5. A fragment of the southern façade of the narthex close to the western façade of the church.

1.4. Alteration to the design solution for the memorial hall

During the archaeological excavation of the site of the exonarthex (memorial hall), it was established that the construction of the new / old walls of the Memorial Hall and the gallery above it, and the laying of the drainage system for the groundwater,will affect the newly built burial sites. The solution is a partial structural modification of the foundations and basement walls of the Memorial Hall.

Excavation works will be done by forming resistant slopes or providing temporary support walls made of steel or timber.

A plan of the new basement walls positions is shown in Figure 6, and a typical cross-section through the Memorial Hall on the southern façade of the Narthex is shown in Figure 7.

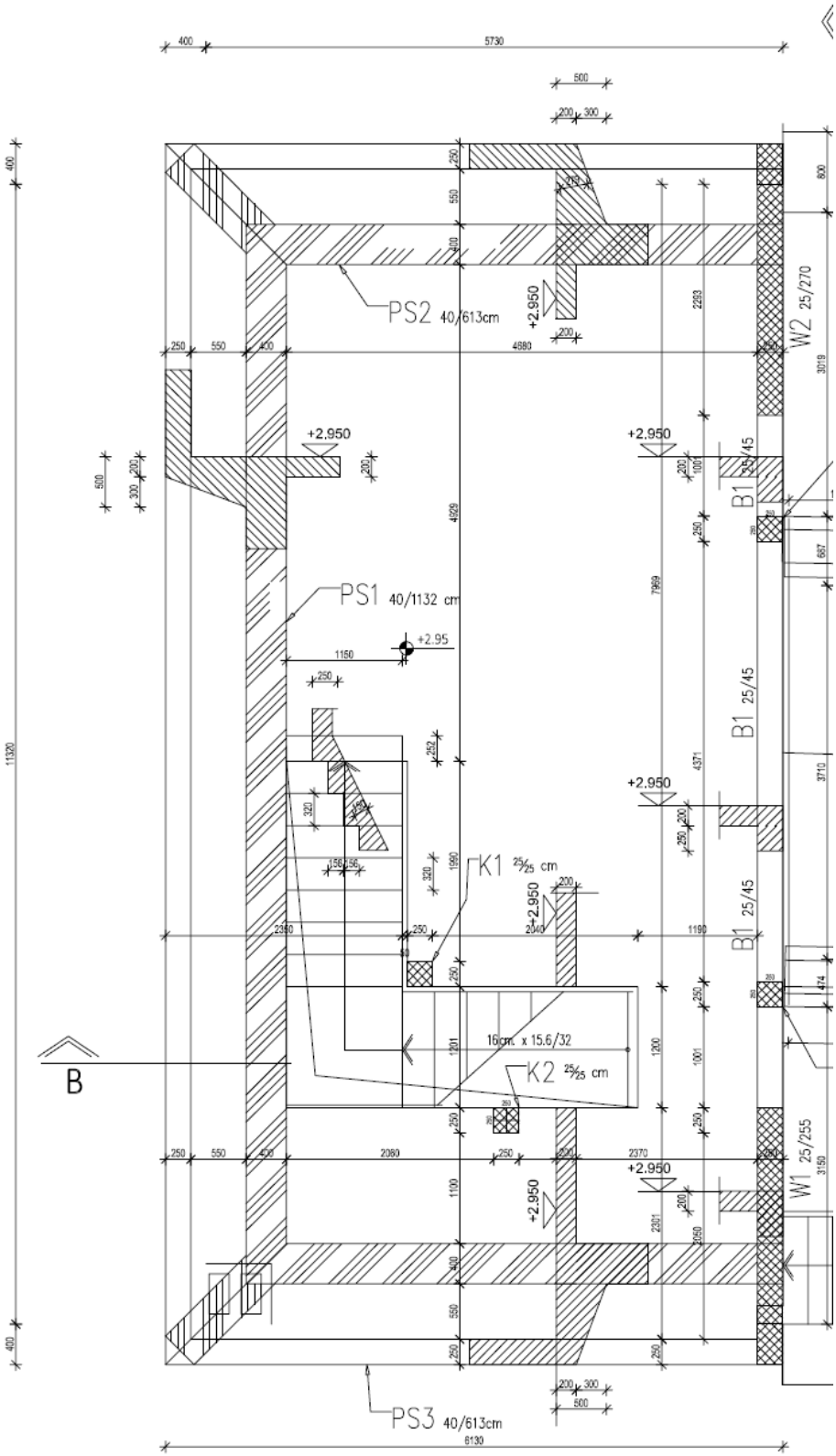


Fig. 6. Plan of the new solution for the basement walls of the Memorial Hall.

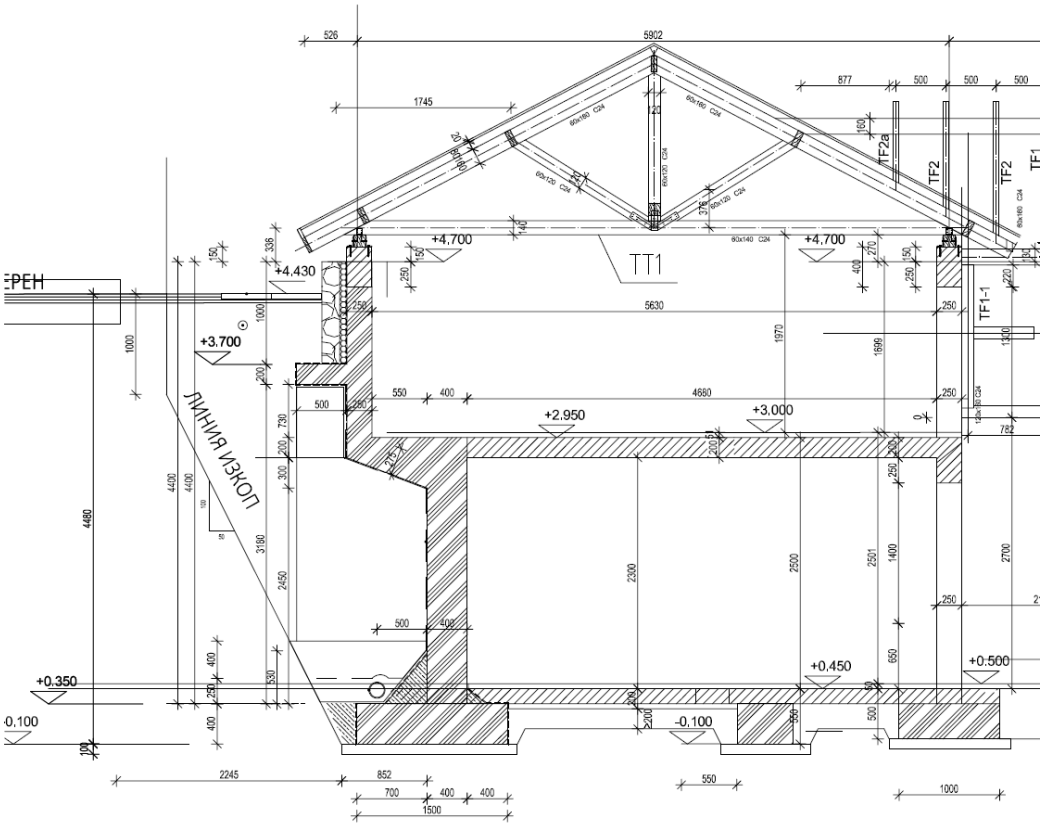


Fig. 7. Typical cross section through the Memorial Hall on the southern facade of the narthex.

1.5. Conclusion

The alterations to the design solutions for the church, the narthex and the memorial hall fully meet the requirements of archaeological, architectural and artistic solutions made during and after the dismantling of the structural elements of the narthex.

With the exhibition of the architectural ensemble of the church of St. Athanasius – acultural heritage monument of national importance – itwill become an accent of the cultural panorama of the town of Boboshevo, along with its churches and monasteries of exceptional artistic and architectural value.

Thoughts on the occasion of the survey and conservations works of the frescoes of thechurch.

And whether one member suffer, all the members suffer with it; or one member be honoured, all the members rejoice with it…Now ye are the body of Christ, and members in particular

Corinthians, Chapter 12 v. 26-27 (King James version)

Saint Athanasius is a cemetery church located in the GornaMahala part of the town. It has a single nave with a semi-circular apse belonging to the type of churches with a recessed western façade. Prof.AsenVasiliev was the first to mention a dating based on the preserved icons in the iconostasis. A closed narthex was added to the late-mediaeval building at the end of the eighteenth century, while in 1926-27 a new narthex and a refectory (exonarthex) were added. For many years, St. Athanasius was the main church of

Boboshevo. Following the building of the big church of the Holy Virgin, it was turned into a cemetery church. There are several possible years for the frescoing of the late-mediaeval church in Boboshevo. The earliest dating is given by VentsislavMaturski, who considers that the naos was built in 1590. SvetozarAngelov read a different year for the frescoing – 1600. Elena Floreva, in her monograph on the Boboshevo church of St. Elijah the Prophet, proposes a date of about 1678 for the frescoing of the church of St. Anathasius. Beneath the image of the Virgin on the dais where the iconostasis once stood, Prof.AsenVasiliev read a date for the frescoing of the frieze of the apostles – 1666 – and discovered a date of 1723 on the icon of the patron saint from the iconostasis by the Master of Sofia,Ioan, and also a second icon of St. Nicholas by the same artist. And according to SvetozarAngelov,regarding the graphic reconstruction by VentsislavMaturski of the wall inscription above the south window, "one cannot with absolute certainty state that this is the year the church was frescoed. The inscription itself is in a very poor state, and the reading of the year may also have other variants…"

As one of the main parish churches in Boboshevo, it has undergone rebuilding, and corrections and repairs have been carried out on it. It was only following construction of the big church of the Holy Virginthat St. Athanasius was turned into a cemetery church. According to Maturski, the naos was built in 1590. It was entirely frescoed inside, and the programme of fresoces in the church, according to art specialists, was usual for the period from the fifteenth to the seventeenth centuries.

The narthex was added in the seventeenth century, and the last repair was done in 1926-27. But this is not information taken from the latest expert assessements:investigations into the building substances, make-up of pigments and linkages for the site remain to be carried out.

An exceptionally valuable second repainting of the frescoes exists in the apse and it corresponds to one of the now missing icons. This second painting of the Four great priest-liturgists is located on a horizontal line below the Virgin Queen of Heaven. The Holy Mother is rendered in the old prayer position, the Orans. At the moment, she is covered in a thick black layer of soot and dirt. Judging from separate fragments, traces of a second repainting are also visible, which was probably the work of the Master of Bansko: they were painted by Dimitar Molerov in a characteristically expressive and colourful way and with a painterly technique.

The Master of Bansko was born in Bansko between 1780 and 1795 into the family of TomaVishanov. His works have been documented in Belgrade, Thessaloniki, Crete, Shkodër (Albania). For a while, he was the court painter to the Serbian Prince Milos Obrenovic.In 1824, the Prince ordered icons in Belgrade, the monastery church at Vracevsnica, the royal icons of the Virgin at Ripanj near Belgrade, Satornja, etc. BrankoVujovic, a specialist in Serbian art of the eighteenth and nineteenth centuries, spoke about this unnamed master: "In Kragujevac, for Princ Milos, worked a master, incomparably better than any other masters who worked at that time in Serbia". Vujovic mentioned the individualistic symbiosis of the traditionally organised iconic arts, and underlined the purely painterly way of drawing the carnation, adding to the baroque construction of the figures with a particularly rich and and expansive drapery, and added: "his are the greatest achievements in art…"

At this stage, it is difficult to date the second rendering of the frescos in St. Athanasius. The Moler family had worked in Boboshevo: here, the father – TomaVishanov – left dozens of icons in the churches in the town. With regard to the programme of frescoes and images in the church, serious scientific publications have been documented. The team dealing with the problems of conservation, at this stage, would not comment on the images in detail due to their poor, dilapidated condition at the moment. It is no coincidence that, in 1957-58, in his "Survey of representational art in several settlements along the Struma Valley" Prof.AsenVasiliev did not comment on the frescoes. It is only after we remove the darkened tatty clothing of the interior that we can sense of the palette of the masters with their iconographic subtleties, hints of a general, colourful tone and harmony with the painted iconostasis, voussoirs, and frescoes as a whole. The frescoes in the church are in a state of emergency: they are covered with dirt from dust, organic layers, and in some places a thick layer of soot.

We shall separate the conservation practices on the frecoes, the iconostasis wall, movable cultural treasures, and also the reconstructions in the final exhibition version as completed into various headings:

One accent of the conservational challenges is in the narthex. The condition of the mediaeval paintings is an emergency. The problems vary, but the most notable are the typical areas of destruction in the first register. The plaster ground has been eroding for many years. At the moment, the sections are covered in cement rendering.

1. Implementation of conservation-restoration practices set forth in the conservation programme for the interior wall frescoes; actions for preservation of the authentic iconostasis divider; mandatory reconstruction of the now non-existent iconostasis structure using layers of plaster and polychrome.
2. Problems with the plaster layers of the exterior, presentation; problems connected with the iconostasis divider; conservation problems and icons.
3. Movable cultural treasures in the naos, the altar, the narthex and the exonarthex to be exhibited and adapted to modern aesthetics in complete harmony.
4. Changes to the constant "historical climate" maintained over the years in the church and the narthex; recommendation, preventative measures for the materials; possible difficulties connected with preservation of the building, with the characteristic, unique frescowork and architecture; cultural monument, part of the cultural historical and artistic heritage of the area – principles.

1. The iconostasis of the church was covered in icon images, wings, doors and friezes, and up to the end of the 1980s it was one of the few completely preserved from the seventeenth century. Elements from the iconostasis and its royal icons are currently stored in different places. Two of them are in the National History Museum. The museum collection in Boboshevo contains some of the conserved icons from the church of St. Elijah the Prophet: the iconostasis divider is identical to the ones in St. Athanasius, St. Demeter, and Saint Petka (Vukovo Village), The last conservation work carried out on the icons from the cemetery church was done at the end of the 1980s by a team of restorers from the NIPK – Kyustendil.

Unfortunately, the interior furnishings are missing: some of them have been destroyed. The iconostasis divider as a silhouette tears the space into two parts. I am speaking of an iconostasis divider, because of the fact that there is no actual iconostasis. That which was documented in the 1980s was an eclectic iconostasis, put together from various periods, which "temporarily" fulfilled its function. During the second half of the twentieth century (1987-1989), the icons, the royal doors, the crown cornice and the wings were conserved in the studios of the NIPK in Kolusha, Kyustendil. At the end of the 1990s, during the greatest period of icon theft in our modern history, the conserved and restored icons were stolen, later recognised, and finally distributed among various museum stores. A survey shall be carried out on the authentic look of the oldest iconostasis in the church. The issues and actions are multiplying, following the latest archaeological surverys by Rumen Spassov – archaeologist and historian from the Regional History Museum, Kyustendil.

The final exhibition place of the iconostasis still needs to be determined.

- Administrative statute of the museum exhibits: the icons, crown cornice, royal doors, crucifix, wings, etc. are already in the "foundations" of museum collections. This presupposes another regimen of permits for a ***probable possibility*** of creating museum copies or probably exhibition in situ.
- It is assumed that the original crown cornice, crucifix, icons etc. cannot be returned to the church, and this will lead to their reconstruction. The reconstruction shall be determined in terms of to what level of execution it should be carried out: new modern versions, copies, replicas, gilding, etc.
- Conservation of the iconostasis wall and replacement of the compromised structural elements.
- Reading and determining an "optimal date" for the final display of the iconostasis. The presence of an iconic image different, both in terms of linear time and style, from the late mediaeval period (icon by the master Dimitar Vishanov, Bansko school of iconography), corresponding to the second rendition of the apse of the church, which will be conserved and presented in a display fashion. There are also imports – elements of Classicism and late Baroque.

VI. With the final display, we would achieve an interesting mix of styles. This controlled, mixed way of presentation would suit the final aesthetic solution. Following work on the original image of St. Athanasius of Alexandria, the portrait image will be displayed in the exonarthex of the church.

The darkened, in places burnt, painting brings out images which are going to surprise us. The reading of the texts and newly-discovered graffiti are also within sights of the team.

The movable cultural treasures also should not be examined outside the overall system of the church and its surroundings: at the moment, the subject of observation are the archaeological finds, *spolia*, the image of the patron saint in its niche, the metal chandelier, the wooden elements for censers nailed to the north wall, etc. The condition of the two objects of cultural treasure is critical and requires emergency measures. The recessed image of St. Athanasius of Alexandria was supported and transported to a studio in Sofia for conservation and *doublage*. The metal chandelier remains for conservation and restoration in the laboratory of the Regional History Museum, Blagoevgrad.



Archive photograph of the church iconostasis Documented in 1980s.



Patron saint icon, Stage of conservation. documented in 1980s, Personal archive



West façade before removal of the narthex and exonarthex



Holy throne



Apse



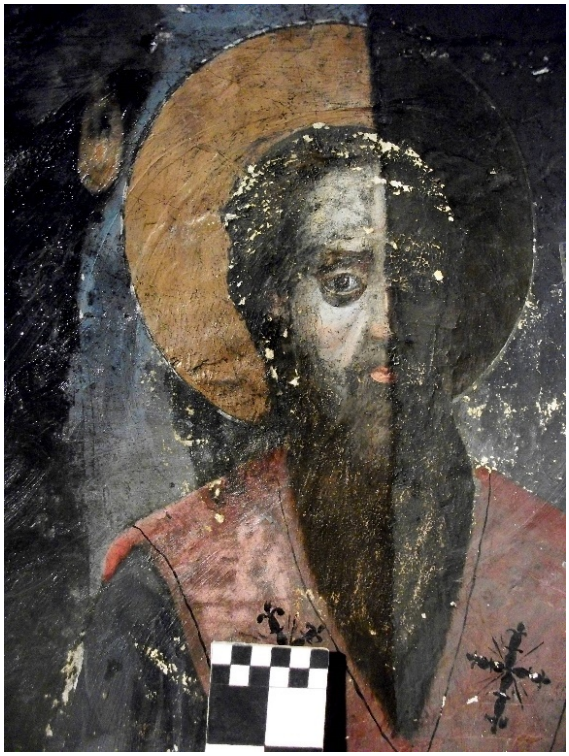
Portrait of an unknown saint, south wall



Stage of conservation. Portrait of St. Vlasius, south wall



Stage of conservation. Portrait of St. Spyridonsouth wall



Stage of removal of dirt from the second fresco layer



Partially removed dirt. Portrait of a saint – artist:Dimitar Molerov.

CONCLUSION:

The biography of this three-hundred-year-old ecclesiastical ensemble has different readings of the architectural text: Spirit of Time and Place; historical context; building periods; materials and techniques; building layout; development of the planar and spatial composition; synthesis of the arts; liturgical *mises-en-scènes* and *eucharistiaesacramentum*. The project for the survey, conservation, exhibition and socialisation – the work of an interdisciplinary team – is an exciting journey through the Past.

Using the postulates of professional ethics, let us restore a living antique of Bulgaria's cultural memory for the Present and the Future!

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ЗНАЧЕНИЕ НА ФАКТОРИТЕ НА СРЕДАТА ЗА ВИЗУАЛНАТА И ФУНКЦИОНАЛНАТА АТРАКТИВНОСТ НА ЛАНДШАФТА В ИСТОРИЧЕСКИ ПАРК „СОФИЙСКА СВЕТА ГОРА"

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IMPORTANCE OF SITE FACTORS FOR THE VISUAL AND FUNCTIONAL LANDSCAPE ATTRACTIVENESS IN THE HISTORICAL PARK "SOFIISKA SVETA GORA"

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University of Forestry - Sofia

***Резюме**: Настоящото изследване е насочено към установяване на значението на различни характеристики на двата основни природни фактора в обхвата на исторически парк „Софийска Света гора" – релеф и горскодървесна растителност, които имат значение за опазването му като рекреационна територия и за експониране на културно-историческо наследство в природна среда, но и с наличието на много нарушени участъци вследствие на рудодобив.*

***Ключови думи**: ландшафт, културно-историческото наследство, природна среда, горска територия*

Въведение

Въпросът за рекреационното използване на горския ландшафт в обекта на изследване напоследък става все по-актуален в практико-приложния си аспект, защото "Софийската Света гора" е тематичен културно-исторически парк, в който ОУП на София предвижда да се развива културно-познавателен туризъм. В резултат се очаква да се породят някои проблеми по отношение на функционално-планировъчната структура, пространствената организация на отдиха и опазването на горската територия от прекомерно рекреационно натоварване [1]. Настоящото изследване се насочва към установяване на ролята на релефа и растителността и тяхното значение за атрактивността на туристическата дейност в специфичната среда.

Материал и метод

София е град с много богато и древно културно наследство. Повечето недвижими културни ценности са концентрирани в центъра на града. Като отлична

алтернатива Исторически парк „Софийска Света гора" предоставя възможност културен и природен туризъм да се съчетаят в едно и то на съвсем кратко разстояние от центъра на града (фиг. 1).

Изследването е онагледено с анализиран графичен материал – топографски карти и сателитни изображения, взети от [2]. Изследването е допълнено с авторски фотографски материал от избрани участъци в района на исторически парк „Софийска Света гора", в които черквите и манастирите се намират в горска следа. Такива са Сеславският манастир „Св. Николай Мирликийски", Кремиковският манастир „Св. Георги" и Буховският манастир „Св. Архангел Михаил". Ролята на релефа и горскодървесната растителност, съвсем естествено, тук ще бъдат водещи за правилната организация на пространството и експониране на ценностите.



Фиг. 1. Общ изглед на разглежданата територия от югоизток (фотография – авторите)

Основни особености на релефа се очертават неговата *силна разчлененост*, както и наличието на множество *изкуствени насипи*, резултат от добив на изкопаеми в близкото минало. Друг много важен фактор, който прави изучавания район особено атрактивен, е преобладаващото *южно изложение*, което е причина за една отлична възможност за формиране на изгледи към София, една от най-ценните характеристики в крайградските лесопаркове [3].

Горскодървесните насаждения заемат голяма част от исторически парк „Софийска Света гора" и са основен ландшафтен компонент. Тяхната роля не може да се определи като такава с водеща културна значимост, тъй като не са открити данни да са носител на автентична информация за миналото [4]. По-скоро в това отношение те имат второстепенна роля, защото са създадени с цел подобряване на условията на средата, защита от ерозия или залесяване на табани и други нарушения в природната среда, резултат от добива на полезни изкопаеми. Горскодървесната растителност, обаче, е доминиращият елемент, който определя общия облик на територията, обемно-пространственото оформление, както и възможностите и качествата на визуалните връзки (фиг.2).

В разглежданата територия са представени всички видове пространства, определени в [5]. Напълно закритите горски пространства преобладават, но те се разполагат извън подходящите за рекреация места.

Видовият състав, основно от иглолистни с характерен представител черен бор, се е наложил поради установената вече необходимост от усвояване на табаните и

съседните обезлесени зони. Това пък спомага за целогодишното използване на територията – през зимата вечнозелената растителност предоставя възможност за ветрозащита. Тук ясно се очертава ролята на закритите горски пространства за регулирането на туристическия поток и за осигуряване на емоционалния комфорт на посетителите [4,6].



Фиг. 2. Горскодървесни масиви определящи облика на територията в района на Кремиковския манастир „Св. Георги" (фотография – авторите)

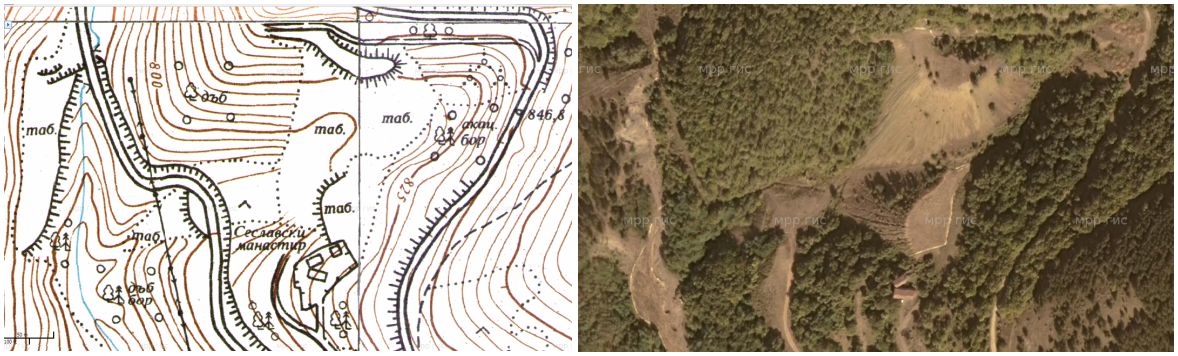
Полузакрити и полуоткрити, както и напълно открити участъци се срещат по продължението на автомобилните и пешеходните комуникации и формират непосредствената среда около тях. Затова те са в пряка връзка с качеството на отдиха, свързан с пешеходното движение (фиг.3).



Фиг. 3. Полуоткрито пространство около транспортна комуникация, предвидена в ОУП за пешеходна връзка към Буховския манастир „Св. Архангел Михаил" (фотография – авторите)

Безспорно най-голямо значение за отдиха в исторически парк „Софийска Света гора" имат откритите пространства. Тук те най-често представляват поляни, чиято периферия едностранно завършва с изкуствен насип, резултат от рудодобива. Тези табани частично или напълно са залесени.

Интерес представлява откритото пространство, в което е разположена единствената запазена сграда от Сеславския манастир. Това е поляна, формирана от няколко по-малки пространства. Пространството е с много сложна ландшафтна характеристика (фиг.4).



Фиг. 4. Ландшафтна характеристика на откритото пространство при Сеславския манастир (топографска карта и сателитно изображение – [2])

Силно раздвижения релеф, резултат от сложни релефни форми, табани и съответно различна пълнота и гъстота на насажденията са предизвикателство към ландшафтното му формиране за туристически цели. Същевременно черквата на манастира е паметник с висока културна значимост (фиг.4).

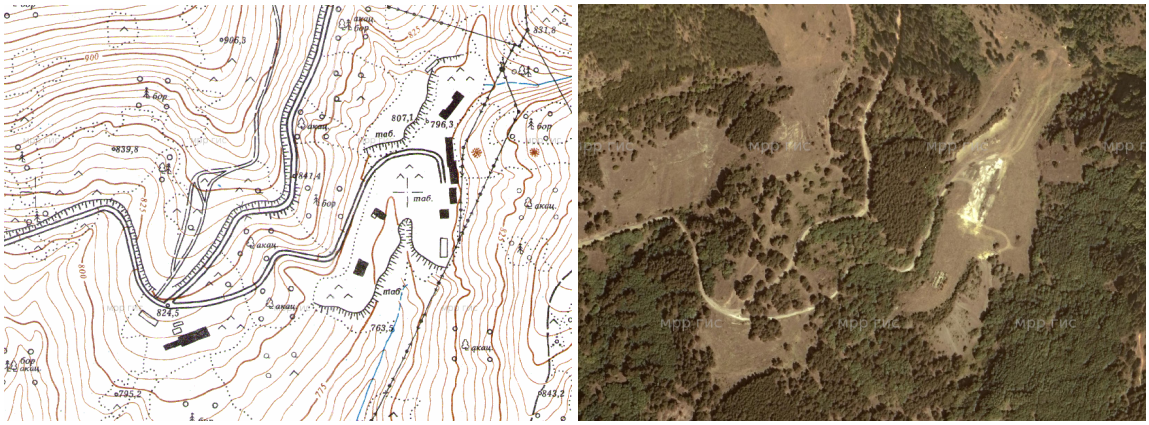


Фиг. 4. Черквата на Сеславския манастир и един от табаните зад нея (фотография – авторите)

С подобна характеристика наблизо са няколко поляни, тангиращи транспортна комуникация в североизточна посока. Откритите пространства са разположени отново от силно раздвижен релеф, изкуствени насипи и структурирани от разпръсната зеленина.

Съществено значение за формиране на някои от тези пространства са разположените промишлени сгради и съоръжения, повечето вече разрушени (фиг.5) .

И докато поляната около останките от манастира се характеризира с разкриващите се гледки към съседните атрактивни ландшафтни участъци, то пространства, подобни на илюстрираните на фигура 5, позволяват възпремане на много дълбоки перспективи и панорамни гледки, фокусът на които се явява Витоша или други планински участъци от южната страна на София. Безспорно тази ценна характеристика е резултат и от допълнителното моделиране на терена.



Фиг. 5. Ландшафтна характеристика на открити пространства в района, източно от Сеславския манастир (топографска карта и сателитно изображение – [2])

Изводи

В разглеждания обект съществува отлична условия за културен отдих и туризъм в природна среда. Това е резултат от комбинираното взаимодействие на следните фактори на релефа – преобладаващо южно изложение, силно раздвижени микрорелефни форми, изкуствени насипи от едва страна, и от друга – висока гъстота и пълнота на насажденията, сред които преобладаващи иглолистни насаждения. Тази комбинация от фактори позволява формиране на нови атрактивни поляни с широки панорамни гледки към град София и далечни перспективи с фокус – планински възвишения от южната страна на града.

Развитието на рудодобива в голяма степен е променило естествения ландшафт, но извършените намеси имат и благоприятно въздействие, най-вече за формирането на гореспоменатите пространства. Тази територия би получила най-пълноценно развитие едва след цялостно култивиране на всички нарушени терени - резултат и от въздействието на много други фактори – антропогенна намеса, биоразнообразие и други, които не са обект на настоящото изследване, но се нуждаят от изучаване.

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THE DEFINITION OF THE UNIQUE ROOF STRUCTURAL SYSTEM OF CENTRAL SOFIA MARKET HALL, TOGETHER WITH ITS ARCHITECTURAL DESIGN

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Abstract: *Central Sofia Market Hall is a very important landmark. Its architectural design as the cover of the building has three parts in architectural planning, as main entrance axis, with two side areas for stalls. Then it has two different portal frame solution, one for main entrance axis and other one is for two side area sof the entrance axis. In the first years of Market Hall, in the beginning of 20th century, it was designed as one floor over ground level, and later after the mid of the 20th century, the first floor over the ground floor has been constructed as the new floor area never touched to the present cover of the building. Its iron-steel roof structural system has a unique solution from the point of steel architecture.*
In the paper, the definition of the unique roof structural system of Central Sofia Market Hall, together with its architectural design will be explained through its main portal frame systems in two different design and solution, with circular columns in the mid of the space, truss beams and roof detailing with bars. Also its address in the city centre will be explained to define the importance of the market hall which is still in service through sustainability in function.

Key words: *Central Sofia Market Hall, Portal Frame Solution, Iron-Steel Roof Structural System*

1. Introduction



Fig. 1. The front facade of Central Sofia Market Hall
(photograph taken by Yesim Kamile Aktuglu, 07.06.2017)

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The front facade of Central Sofia Market Hall, which has an address as 24 Maria Louise Blvd. (Fig.1) just in the city centre, has the main entrance, with a clock at the apex. All four facades are having similar architectural design, having an entrance door in the id of the facade with several windows at both sides of the entrances. At the moment, there are only three doors are in service.



Fig. 2. The identity detail of Central Sofia Market Hall on the front façade (photograph taken by Yesim Kamile Aktuglu, 06.06.2012)

On the front facade, it is written that this building was Municipal Hali in 1909, and its architect was Naum Nikolov Torbov (Fig.2). From a photograph in 1961, it is understood that it had a single storey, as ground floor, with the same unique roof structural system. Now there are two stories as ground floor and first floor over the ground which affects the inner space, comparing the first design as a one storey inner space.

In the paper, the unique roof structural system will be examined, through its structural detailing advantages together with its architectural design.

2. Architectural features of central Sofia Market

Central Sofia Market Hall has three parts in floor plan. One is the entrance hall which is main axis and narrower than the other two parts which are at the both sides of the entrance hall. The entrance hall has the full height of the building, where the other two has one floor upstairs. Then the both floors are having the half height for each of them. In the mid of the plan, main axis is perpendicular with the other corridor axis for the doors at the sides.

At the ground level and at the first level there are shops and closed spaces as office areas. In the main axis there is a cafe, having all inner view at once, from ground to the top.

There is a perimeter wall, surrounding whole building complex, having 4 doors and totally, 40 windows, which are designed for one storey building and now gives service for light for two storeys. The roof design is as gable roof for main entrance hall and as hipped roof for the side parts. Three of them have clerestory for daylight.

There are 12 circular hollow section columns at the sides of entrance hall to get the loads coming from the roof levels. At the side ends of the building, the loads coming from the roof, are transferred to the perimeter wall.



Fig. 3. The main entrance door of Central Sofia Market Hall from inside (photograph taken by Yesim Kamile Aktuglu, 07.06.2017)



Fig. 4. The end of the main entrance hall of Central Sofia Market Hall, with stairs up (photograph taken by Yesim Kamile Aktuglu, 07.06.2017)

Towards to the main entrance door, from the end of the main entrance hall, the portal frames for the mid part of the building are defining the space and also letting the daylight in (Fig.3). At the end of the entrance hall, the stairs are letting the way to the upper level (Fig.4).

3. Structural features of unique roof design

The structural design of roof solutions has a very unique feature, as a tailor-made system, which may be called as a mixture of a virendeel beam and a castellated beam in the manner of a Polonceau roof truss, converted from a timber roof system, as with king post and queen posts by iron bars. And also both type of roof trusses over the halls are vector-active structures.

In the main entrance hall, there is a iron-steel truss, as king post by bars, to keep the both inclined lattice trusses, at the apex, connected with, in its first designed form. Not to let the end of the trusses over the columns move to be opened due to the vertical loads coming down from the roof level, horizontal iron bar is in service in tension and the vertical iron bar is keeping the horizontal one not to go down freely, due to the span. The

lattice beams are made with equal leg angles in double for the upper and lower flanges of the beam, and the inner part is constructed with metal sheets in strips and in forms to compose a tailor-made product to get the strength of the combination of vireendel beam and castellated beam. The lateral beams are I-beams to get the total stability. Over the third purlin the clerestory is present, having Windows inside a Viereendel beam, surrounding the lightning pool. In the apex, the lateral beams are similar to the portal beams, having a less height than the portal ones (Fig.5).



Fig. 5. The connection level of portal frames with trusses as king post and quen posts with iron bars
(photograph taken by Yesim Kamile Aktuglu, 07.06.2017)

The second roof solution has a similar frame with queen posts in iron-steel bars. In this type, the clerestory is above the fifth purlin's place as iron-steel lattice beam and has similar composition for roof top windows as inside the Virendeel beam. This iron-steel lattice beam is surrounding the perimeter of the daylight pool (Fig.6).

The pattern of all portal frame lattice beams are as ".....IXIXIXIXIXI....." where "X" is for castellated beam and where "I" is for Virendeel beam.

Over the columns, there are supporters as haunch knee bracings in four directions, and also there are haunches which there is no column under to transfer the loads down.

The connections are made by rivets or bolts.

At the other end of the portal frames with queen posts by iron-steel bars, the loads are being transferred through the perimeter wall with gussets or without, down to the foundations (Fig.7).



Fig. 6. The front facade of Central Sofia Market Hall
(photograph taken by Yesim Kamile Aktuglu, 07.06.2017)



Fig. 7. The front facade of Central Sofia Market Hall
(photograph taken by Yesim Kamile Aktuglu, 07.06.2017)

The deepest lattice beams are over the columns, the least lattice beams are around the daylight pool and at the apex of the portals of the main entrance hall. The inclined lattice beams of portals have the middle height in these three kind of lattice beams (Fig.5).

There are cross-bracings at the daylight pool level. In the apex level of the portal frame truss with queen posts with iron-steel bars, there are vertical cross bracings (Fig.8).



Fig. 8. The horizontal cross-bracings at the daylight pool level and vertical cross-bracings at the apex level
(photograph taken by Yesim Kamile Aktuglu, 07.06.2017)

4. Conclusion

With a date as 1909 as a Municipal Hali, the building has arrived today as a Central Market Hall in 2017with a great success by having an additional floor under the unique roof structural system.

The new floor has no real touch with the present structure, esp with the roof structure. It has a separate structural system having balconies around, can be reached with stairs at the end of the entrance hall (FIg.9) (FIg.10).

The design for getting daylight inside, from windows and also from clerestories, make the whole spaces be used easily at the best comfort level.
To be able to see every place from every point of the building also adds quality in service.

Central Sofia Market Hall has a total space, acts as one of the very important Landmarks of the Universe. Its architectural design is very functional. Its engineering design is very logical.

What is interesting is that, even though in past till 1909, there were so many similar structural design solutions with "IXIXIXIXIXI" as the beams of Cyristal Palace, as the portals of Palais des Machines, etc., the structural solution of Manhattan Bridge, in NYC, in 1909, has a large coincidence with Central Sofia Market Hall's unique roof structural system, in 1909. At least, to be aware of this, gives happiness in the manner of being present about Landmarks with similar structural solutions at the same time in the far edges of the Universe.



Fig. 9. From the first floor towards downstairs
(photograph taken by Yesim Kamile Aktuglu, 07.06.2017)



Fig. 10. From the stairs toward Ground Floor together with the first floor
(photograph taken by Yesim Kamile Aktuglu, 07.06.2017)

Acknowledgement

I am very thankful to Prof.Doncho Partov, PhD, who supported me to prepare a paper for this conference, **International Scientific Conference "Preservation of Cultural Heritage" BASA' 2017** from the first moment when i heard about the conference in the beginning of 2017.

Thank you very much Prof.Doncho Partov, to be able to prepare a paper for this conference.

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GOAT'S BRIDGE IN SARAJEVO, BOSNIA AND HERZEGOVINA

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***Abstract**: Goat's Bridge in Sarajevo was built mainly out of lime and tuff stone in XVI century as one of key links of the chain of, back then very important "Istanbul Road". The bridge was always a part of part of the daily life of the local community and that is why there are many legends, folk stories and songs about it and connected to its name, even though its founder is unknown. During the last four centuries there were a lot of attempts of repair works on the bridge, but most of them done more damages to the structure rather they helped to preserve it. Today, we can notice cracks and other damages on the bridge which are very dangerous for the structure of the bridge itself. These crakes and damages aren't dealt with professional care and the problem of them is becoming bigger over time. In modern times the bridge was left out of the modern communications and roads and it is now looked at as one of the touristic sites of city of Sarajevo.*

***Key words**: Goat's Bridge, one-span bridge, stone bridge, Sarajevo, Ottoman architecture, lime stone, restoration and repair works, current state*

1. Introduction – Historical Overview

At the east entrance to Sarajevo, 3 km from the center of the city, on Miljacka River, we can find beautiful stone bridge called Goat's Bridge. During the Ottoman period, this bridge was part of the "Istanbul Road", main communication leading to Istanbul, center of the Empire, connecting Bosnia with Serbia, Montenegro, Sandžak, Macedonia and entire East. This emphasizes the importance of this bridge – putting it in the same category with famous Mehmed Pasha Sokolovic's Bridge inVisegrad. The reason why the surrounding of this bridge is not densely inhabited is its position. The Bridge is situated in a narrow ravine. Another reason can be its close distance from center of Sarajevo. Welcomes and goodbyes characterized this bridge through history. Pilgrims on their way to Mecca were seen of or welcomed back on this spot.

This bridge was almost forgotten for more than a century. New communication built nearby, put this bridge in back plan. This was a two blades sword. On one hand the bridge

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was saved from damages caused by heavy traffic and urbanization, however on the other hand it was abandoned and neglected by authorities and public.



Fig. 1. Goat's Bridge in Sarajevo, current state

This bridge is one span stone structure and it belongs to the group of big-size constructions of this type. Its span is 17.60 m and height 10.30 m. The radius of release openings is from 3.01 to 3.15 m. Width of the bridge is 4.17 – 4.79 m in arch fragment, and 7.54 m in its maximum side range. Length of the bridge from one to another end of the parapets is 42.18 m in plane level, and 49 m measured by its pedestrian path.

It was settled on the solid rocks on both sides of the river. In order to equal the difference between the heights of two banks, they built big abutment wall on the right side (9 visible rows) and 2 to 3 rows wall on the rock at the left side. These walls were made of local lime stone. To underline the difference between abutment walls and load bearing arch, constructor narrowed the arch for 5 to 6 cm on each side. Abutment wall under the arch consists of 11 horizontal rows of stone block (one edge stone in the upstream side is also limestone, although the rest of the row is already made of tuff). Further on, it is obvious that the arch is made of tuff blocks, until the top of it, where three key stone rows are made of limestone.

Reduction openings are made of tuff stone, indented into the spandrel walls in same manner as the load bearing arch. For spandrel walls, cornice, parapets and pavement, ancient builders used local limestone. Cornice profile is simple yet still beautiful. Joints between the stone blocks were filled with lime mortar, and most of them were attached to each other with forged iron cramps. Three iron strain shafts are fixed into the spandrel walls from one side to another. They are clearly visible on the façade, two on the left and one on the right side. They probably date from one of the reconstructions.

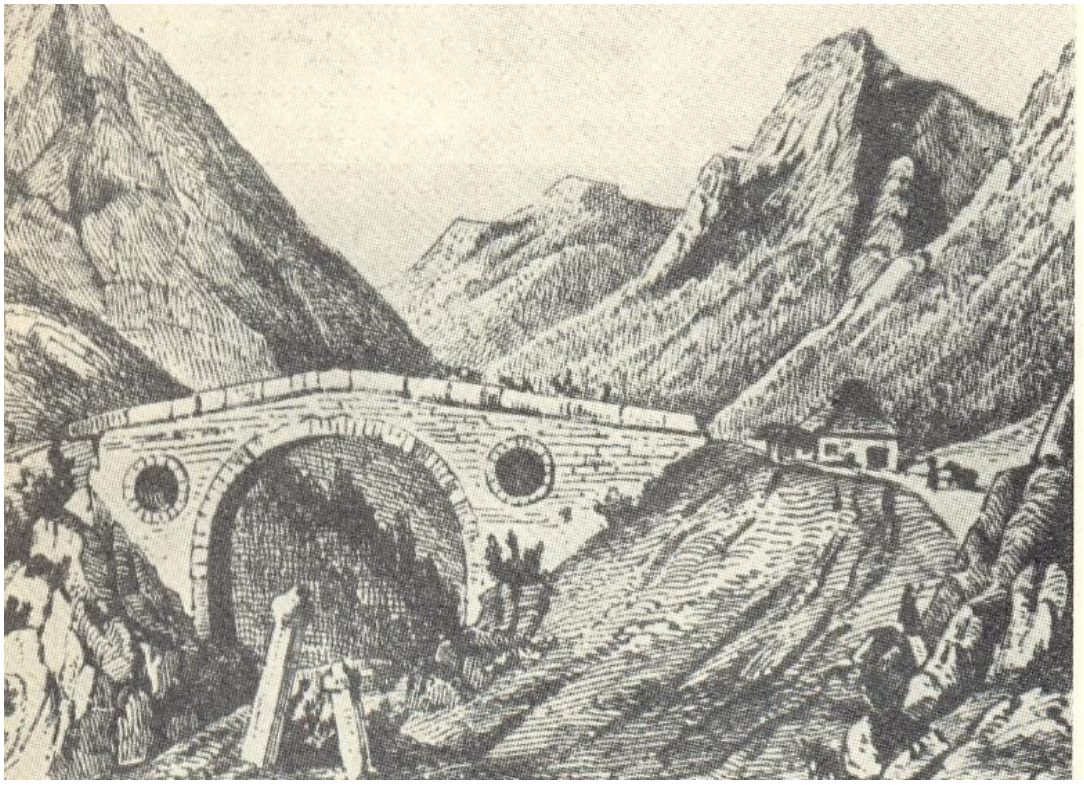


Fig. 2. Goat's Bridge in Sarajevo, historical drawing from 1873

1.2. Origins of the bridge

There are many tales about this bridge origins, but none of them seems to be authentic. One tells the story about an orphan, Meho, who lived with his neighbors and dreamt about Istanbul, Sultan and royal suite. One day when he was watching the goats, he noticed one goat digging earth on one spot. He went there and discovered many treasure pots. With that money he went to Istanbul, finished the best schools and joined Imperial service. Soon he became Pasha. His stepfather's house became a wealthy home, and as the memory of goats and found treasure he built this bridge.

The other one tells about two brothers Mehmed and Sinan who were also watching over goats and found lost treasure in a nearby cave. Sinan erected Kečedži Sinan's (keci – goat in Turkish) mosque and Mehmed built Goat's Bridge. According to Sejfudin Kemura, the mosque was built in 1544, but later research shows that it was built at least 30 years before, in 1515 and the name Kečedži Jusuf was related to it. What is most likely – Sinan and Jusuf are the same person, and keke was the hook for bending fruit trees branches. Last research done by the historian Hamdija Kresevljaković gives us the 1530 as the most possible time of mosque arising. What seems to be most probable is that square besides the bridge was used as live stock market – because nearby village is also called Jarčedol (Goats-dale).

One theory goes as far as to claim that legendary Meho is no-one else but Mehmed Pasha Sokolović (according to Robert Michel – Alte Brucken). This would be credible if there was not a sign on the kiosk by the bridge (no longer existing) which mentions years 1577.-1578., same time as when Višegrad bridge was built. Of course, it is possible that the kiosk was added later, in years mentioned above.

What is sure is that by its position, appearance and the period it was built in, this bridge was definitely important part of "Istanbul road" and of entire Ottoman communications network.

2. Historical interventions on the bridge

First accurate notes about this bridge come from XIIII and XIX century, and provides data about its repairs. Tax report from 1794 quote following means and costs necessary for bridge repairs:

chiseled stone for repair of the Goat's Bridge given to stone-mason Ahmed-basha - 87,5 grosz
carpenters 290 grosz and - 36 dimes
workers 117 grosz and - 14 dimes
timber and iron - 115 grosz
for material transportation to the commissioner himself - 15 grosz
to Hasan-Aga the headmen for works on the bridge - 40 grosz
for supervision on mentioned bridge - 40 grosz

From the quantity of materials, especially wood for the scaffolding and iron for connectors – we can tell that they needed a scaffolding. That means that damage was sizable and that the load bearing arch must have been damaged. M. Bašeskija registered immense floods on 15th November 1791 which destroys all except Šeher-Čehaja Bridge in the center of the city. Flood this big can be totally devastating for the construction in such a narrow ravine as the one in which the Goat's Bridge is situated. Perhaps it was lack of financial needs, but it was definitely lack of capable constructors and it was the reason why this reconstruction started three years after the flood. Bašeskija also says that when in 1697 Eugene of Savoy intruded in Bosnia and Herzegovina, repairs on facilities destroyed in that occasion lasted for next half of the century.

The same author registered another order of material for this bridge (11th April 1799), in fact – it was intended to use the other pedestrian wooden bridge as a temporary substitute for Goat's bridge while it was being repair. The works on Goat's bridge continued simultaneously, according to this author, on 22nd June the same year, the arch of the bridge with the scaffolding and five stone cutters fell down. One died in this accident and others were injured.

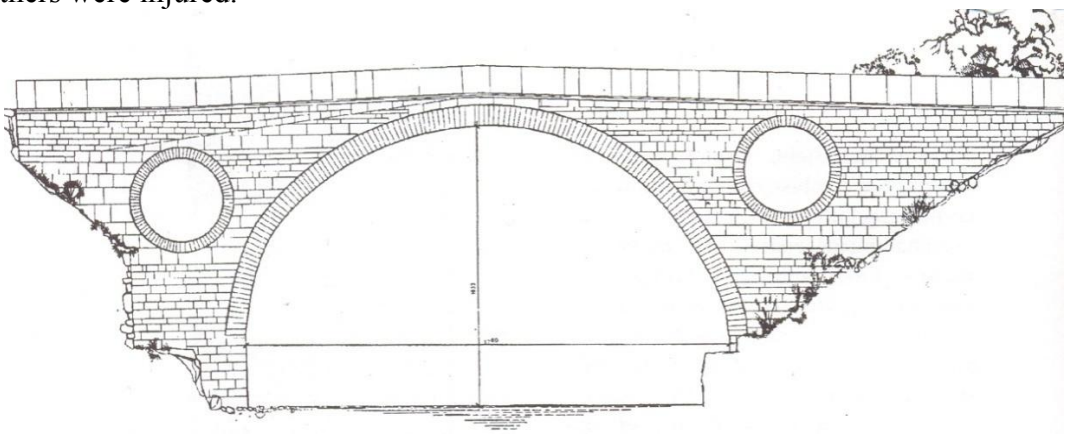
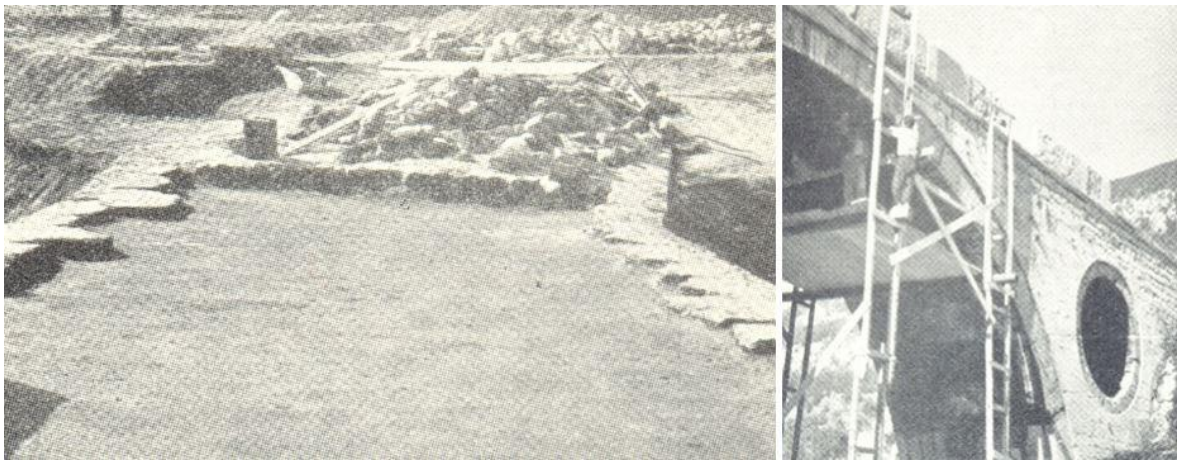


Fig. 3. Goat's Bridge in Sarajevo, Drawing of the upstream side

During the XVIII century, this bridge and the Ali-Pasha Bridge requested several large repairs. This fact can tell us much about the constructors and their skills, or in other words, their incompetence and lack of skills. The Ottoman Empire have already built what they needed in Bosnia and Herzegovina, and not only did they not find it necessary to have some of their good engineers in this area, but they also took away all skilled and talented builders to help them in engineering troops in north borders. Lack of available builders from Istanbul school caused many problems in damaged objects. Repairs done by local unknown builders with their staff from Herzegovina and Dalmatia were poor and mostly inadequate. We can find testimonies about this in folk songs on 19th century. They sang about hard work of incompetent builders and they became parts of the legends.

One note about the repair of Goat's Bridge was made on 20th September 1838, referring to the damaged road leading to the bridge. Last note was made in 1866 in "Bosnia" magazine, about "some" repairs on the bridge.



a) b)

Fig. 4. Goat's Bridge in Sarajevo: a) works on insulation on the pavement of the bridge; b) repair works on the bridge (1956-1957 restoration)

During the Austro-Hungarian occupation, a new road to Pale was made in the east, thus the bridge had to live through some changes. Access roads now were elevated to a new level, so the right side of the bridge had to be raised. Left side had only smaller interventions and the marks of its uplifting are barely visible. Spandrel walls on the right side were lifted up to equal with the new road. Works started on 1880 and finished in 1888 (recording to Sidžil).

In recent times, Goat's bridge had one major repair during 1956 -1957. Before this intervention, the bridge was in a bad condition mostly because of many previous attempts of reparation and the fact that different materials used on this bridge were in different condition.

2.1 Aftermath of inadequate repairs

The bridge was built on solid rock, and first rows of arch, made of limestone, were in satisfying condition. They only had a few microscopic cracks which did not influence the construction stability. The situation was worst with the rest of the arch made of tuff stone (Calcareous Sinter). Disintegration was obvious on left reduction opening, on its upstream side cornice was completely missing for its almost 3m of length and 30-50cm of thickness. Load barring arch was also damaged in 7 stone layers.



a) b)

Fig. 5. Goat's Bridge in Sarajevo: a) right abatement; b) left abatement

Mortar from the joints started to fall down, and took pieces of stone or even entire blocks with it. It was a perfect base for weeds and small bushes, and as the bridge wasn't maintained for years, their roots went deeper into construction. That led to even worst destruction of structure, greater loss of mortar and stones.



a) b)

Fig. 6. Goat's Bridge in Sarajevo: a) arch intrados; b) cracks on the arch intrados

As mentioned in text above – the pedestrian path on both sides of the bridge was raised in 1888. Right side had more than obvious uplifting – 2.76 meters above the initial level. Original slope of the bridge was much steeper, and its composition of layers and structure did not predict any water accumulation. This new situation on the surface of the

bridge caused serious problems for entire bearing structure. First problem was its new weight: the one gained from additional structure and the one out of layer of earth and stones washed away from the hill above. Almost entire pavement was covered with thick layer of floor deposit.

A more dangerous problem for this new shape of bridge was water. Original structure and materials used for it did not have problems with water accumulation on the pedestrian path. Slope was steep enough that every drop which fell on the surface immediately slid downhill. New shape of the bridge carried something completely different for the old structure. Water from the rain and snow was resting longer in deposited soil layers and puddles and slowly leaked trough the structure, washing away the mortar from the joints between stone blocks and harming mostly the load bearing arch and reduction openings made of tuff stone.



Fig. 7. Pedestrian path and parapets

Left side of the bridge was visibly recessed at one spot. Cornice became wave shaped above one of the two left side metal strain shafts. Interesting thing was that this phenomenon seemed not to disturb the construction of the load bearing arch or reduction opening. It is possible that this strange shape was result of some inadequate repair from 18th century. Bridge also lost most of the lime stone parapets. Before the 1956-1957 restoration, it had only 20 pieces of blocks on the upstream, and 13 on the downstream side. It is noticeable that although motor vehicle traffic over the bridge is prohibited, vehicles are using it.

Acording to the Commision to preserve national monuments of Bosnia and Herzegovina there was a study made in 2001 and 2004 of the damage to the bridge, during which it was discovered that in addition to the stone being dirty, the stone parapet of the bridge had tilted from the vertical on the south-eastern side of the bridge.

The following works were carried out:

1. the bridge was mechanically cleaned using water under high pressure;
2. the removal of salt residues by using an appropriate revetment and water under high pressure;
3. fungicidal and anti-bacteriological treatment;
4. protection of cut surfaces using appropriate chemical preparations;
5. the dismantling and subsequent replacement of the sections of the parapet that were out of true;
6. clearing the site of self-sown vegetation.
7. the site was cleared (2004);
8. the structure has been floodlit (2004).

This was carried out by the Cantonal Institute for the Protection of the Cultural, Historical and Natural Heritage of Sarajevo with the help of Stari Grad Sarajevo municipality.

4. Conclusion

Goat's Bridge in Sarajevo is one of the best examples of public Ottoman architecture in Bosnia and Herzegovina. The bridge used to be one of the key spots of "Istanbul Road" and as such it was very significant for the Sarajevo and its surroundings. Construction of new and modern roads left Goat's Bridge in the past and forgotten for years.

Goat's Bridge did not suffer much damage in Bosnian war 1991/95 which was the common case with other important monuments. At first sight we can even tell that it is in pretty good condition. But reality is completely different. Micro cracks on load bearing arch, and on its abutment part are numerous and very serious. They are most probably caused by the weight of injection material and insulation layers. What catches the eye are also very thick and inappropriate coloured joint between the tuff stones. Considering rough structure of tuff stone – it seems impossible to clean it without further damage of the load bearing arch.

In one word – our humble opinion is the following: although it seems that most of the monuments are at its edge of existence by staying unpreserved, the harm that can be done to the bridge by inadequate restoration is much bigger and in most cases – irreversible. Concrete layers are something that can't be removed without harming the stones in the construction. The style of an era should not be the guideline to destruction.

The lesson learned on this bridge is following: if we are not sure that we can provide adequate help – it is better to leave it as it is – future generations will definitely have better solutions.

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IN MEMORIAM
Prof. d-r eng. **Yordan Milev**,
(1960-2017)



On January 9, 2017 in Sofia Prof. Dr. Yordan Ivanov Milev suddenly left us at age of 57.
Born on February 15, 1960 in Sofia. He completed his secondary education in 1978 at the Sofia Mathematical High School. From 1980 to 1985 he stadied "Industrial and Civil Engineering" in the Higher Institute of Architecture and Construction (HIAC). In 1985-86 he held a post-graduate specialization at the Center for Applied Mathematics of the Higher Institute of Mechanical and Electrical Engineering. In 1986 he was chosen as Assistant Professor in the Department of Reinforced Concrete Structures and until 2004 he passes through Senior Assistant Professor and Chief Assistant Professor. In the years 1994, 1995 and 1996 he was consistently a postgraduate and a visiting researcher at the Tsukuba Science Research Institute, Yokohama University - Japan and the Institute of Seismic Engineering at the Tokyo University. In 1997 he was visiting researcher at the CITY University, London and the Technical University in Darmstadt, Germany. In 1998 he was a visiting researcher at the University of Naples, Italy.
In 1999 he defended his dissertation for acquiring the educational and scientific degree "doctor" on topic: "Modeling of reinforced concrete walls for seismic research of buildings with reinforced concrete structure" - under the guidance of Japanese professor Toshimi Kabeyasava. In 2000-2001 he was a visiting researcher and lecturer at Chalmers-Göteborg Technical University, Sweden. In 2001-2003 he conducted a postgraduate research at the Institute of Seismic Engineering at Tokyo University.
In 2004 Dr. Eng. Milev was elected as Associate Professor and in 2013 he was elected as Professor in the Department of Reinforced Concrete Structures at the University of Architecture, Civil Engineering and Geodesy (UACEG) - Sofia. He has three workshop certificates - fire safety from 2010, Eurocode 2 from Brussels, 2011 and Eurocode 8 from Lisbon, 2011. Prof. Milev was a member of CHAMBER OF ENGINEERS IN THE INVESTMENT DESIGN (CEID) where he was a lecturer in the Eurocode training courses. He had control functions in the design of structures. He was awarded with 4 awards Designer of the Year in 2008 and 2009 in National Competition "Building of the Year" held under the patronage of Ministry of Regional Development and Public Works (MRDPW).
In his educational and pedagogical activities Professor J. Milev held lectures, exercises and course design in Bulgarian and English language in a wide range of disciplines such as:
Reinforced Concrete, Reinforced Concrete Structures - general course, Special Reinforced Concrete Structures, Design of structures for earthquake resistance, Seismic study of tall buildings with reinforced concrete structure, Seismic assessment and restoration of existing buildings. He has composed two study programs and two in co-authorship. Prof. J. Milev was tutor of one PhD student who successfully defended doctoral dissertation. He was a member of State graduate examination committees. He is author or co-author of 7 manuals and 54 scientific papers presented in the competitions for associate professor and professor. Two of

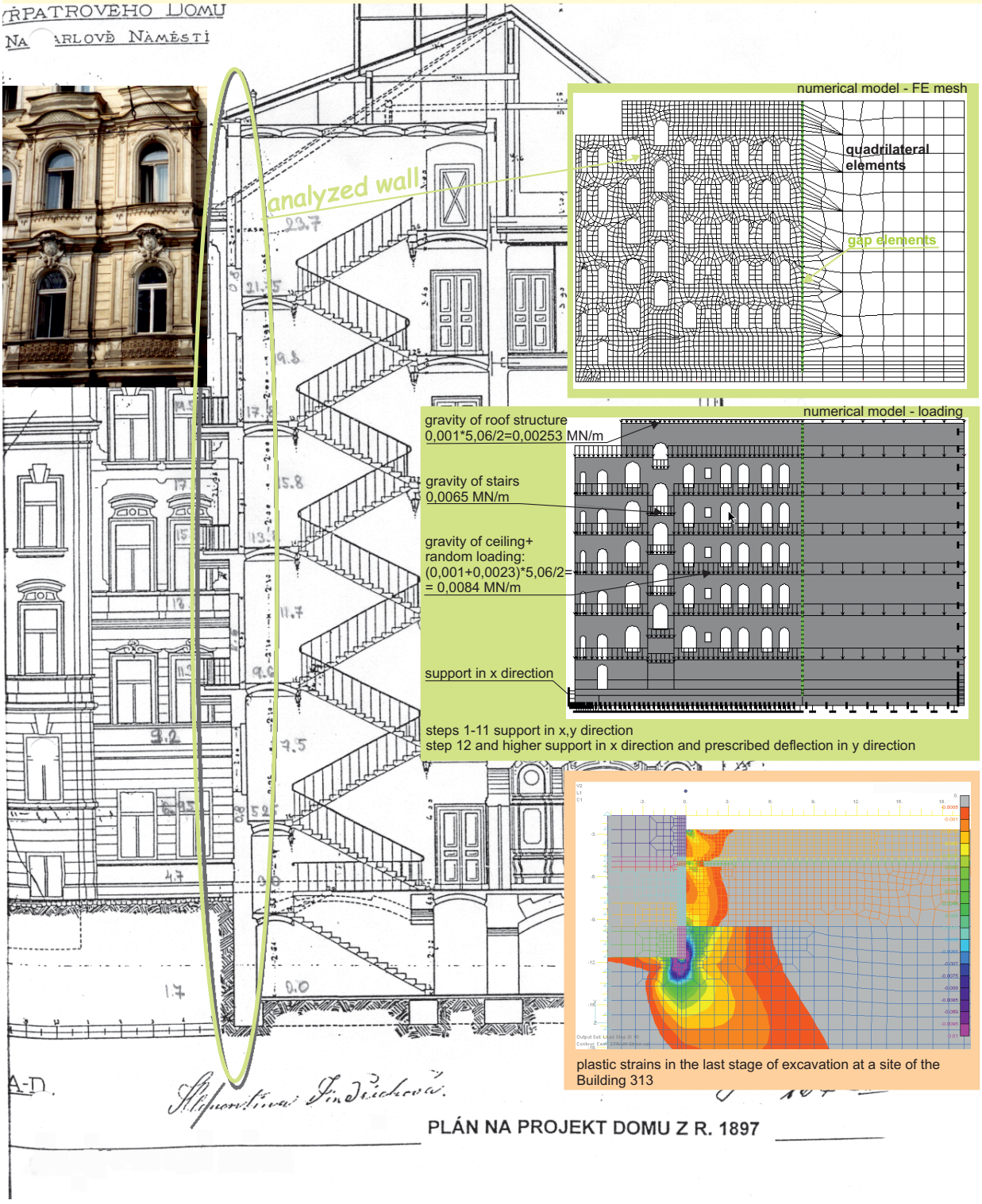
the manuals are the first manuals for the design of reinforced concrete multi-storey buildings according EC2 and EC8, which are extremely useful in the application of Eurocodes for students and designers not only in Bulgaria but also in the Balkan region.
Prof. Dr. Eng. Yordan Milev was acquainted in detail with the methods of modeling of different structural objects as well as with the capabilities of powerful computer programs for mechanical and mathematical modeling in structural design: SAP 2000, ETABS, Tower, Drain 2D and 3D, Abacus, CANY etc., which helped him both in design practice and in student education. He also had a very good knowledge of computer-aided design using the AutoCAD software.
In his science and science - applied activity, he wrote a monograph titled: Eurocode 8, Seismic Design of Reinforced Concrete Structures - Part One - Practical Guide, CEID, Sofia, 2012. The monograph is extremely useful to the engineering college for assimilation and the implementation of the EC8, assimilation and applying the experience of leading countries in seismic design and construction. The monograph is a complete concise, concentrated exposure of all the main principles, rules, limit states, structural features and normative requirements for the investigation and the provision of reinforced concrete structures for seismic actions. Basically, the rules of capacity design, the hierarchies of the load-bearing capacity of the elements and the construction features, the mechanism of energy dissipation are considered. New points in the use of materials - concrete and steel - related to restrained concrete, deformation limits, ductility, bond, over strength bearing capacity and other concepts and terms are explained. The modeling of structures for seismic impacts, methods of analysis, application of modern software products are considered. The method of horizontal forces, the spectral method with modal analysis applications and the rules for combining the response are clarified. The author offers a better reasoned classification of constructions compared to the EC8 when creating the computational model for determining the design response spectrum and the behavior factor. He suggests the stiffness of the cracked elements to be adopted with differentiated response factors, according to the Japanese standards, rather than the EC8. His conclusion is that according EC8 are obtained large lengths of hidden columns compared to Japanese and US building codes, leading to non-economic solutions. In Bulgaria there are no normative documents for seismic assessment and reconstruction of existing buildings. The author proposes a methodology for adapting the three-stage Japanese assessment procedure for the Bulgarian conditions, explaining all the elements of the procedure and their possible values. For illustrative purpose, the condition of a school building in Sofia is assessed. Emphasis is placed on the urgent need for the establishment of rules for seismic evaluation of existing buildings.
Prof. Y. Milev develops active design activity. More than 50 industrial, residential, administrative, office buildings and facilities have been constructed by his projects. A number of innovations have been implemented. He was awarded with 4 awards Designer of the Year in 2008 and 2009 in National Competition "Building of the Year" held under the patronage of Ministry of Regional Development and Public Works (MRDPW). He has carried out research and has developed and realized projects for strengthening of more than 70 buildings. He has applied modern methods, materials and technologies with externally bonded FRP and embedded steel connections.
In the eyes of his teachers, as a student and a graduate, Prof. Milev will remain forever excellent during the course of study, developed and defended his diploma thesis with excellent mark. In the minds of his colleagues, he will remain forever with the perception of a very modest and hard-working man, talented researcher, erudite lecturer, outstanding designer of modern structures. He has conducted numerous specializations at world-renowned universities and research institutes, with great ambition and desire to share and deliver the lessons learned to specialists and the engineering college in Bulgaria and abroad, especially our neighbors from Serbia and Macedonia.

Prof.Miles is a co-author of several edited manuals:
1) Methodological instructions for the course design for the subject "Reinforced Concrete" for the specialty "Industrial and Civil Engineering", HIAC, Sofia, 1992, 1993, 1994 - third revised edition; 2) Guidance on Reinforced Concrete Structures, Part II: Prefabricated one-storey industrial buildings, HIAC, Sofia, 1994; 3) Methodological instructions for course design of "Reinforced Concrete Structures" for the specialty "Industrial and Civil Engineering", Calculation of the transverse frames of one-floor industrial buildings, HIAC, Sofia, 1993; 4) Methodological guidelines for the course design of "Special Reinforced Concrete Structures" for the specialty "Industrial and Civil Engineering" (part-time study), Calculation of the spatial nodes of combined rotary shells from asymmetric loading, HIAC, Sofia, 1991; 5) Methodological instructions for the course design of "Special Reinforced Concrete Structures" for the specialty "Industrial and Civil Engineering", HIAC, Sofia, 1990; 6) EUROCODES: Guidance for the design of Reinforced Concrete Structures, part one - Multi-storey office building, Chamber of Engineers in Investment Design, Sofia, 2012; 7) Methodological guidelines for course design for the specialty "Construction of buildings and facilities" - Prefabricated Reinforced Concrete single storey warehouse building, according to Eurocode requirements, Sofia, 2012.

Drafted on behalf of your colleagues and friends:
Doncho Partov and Ivan Ivanchev

ATENA

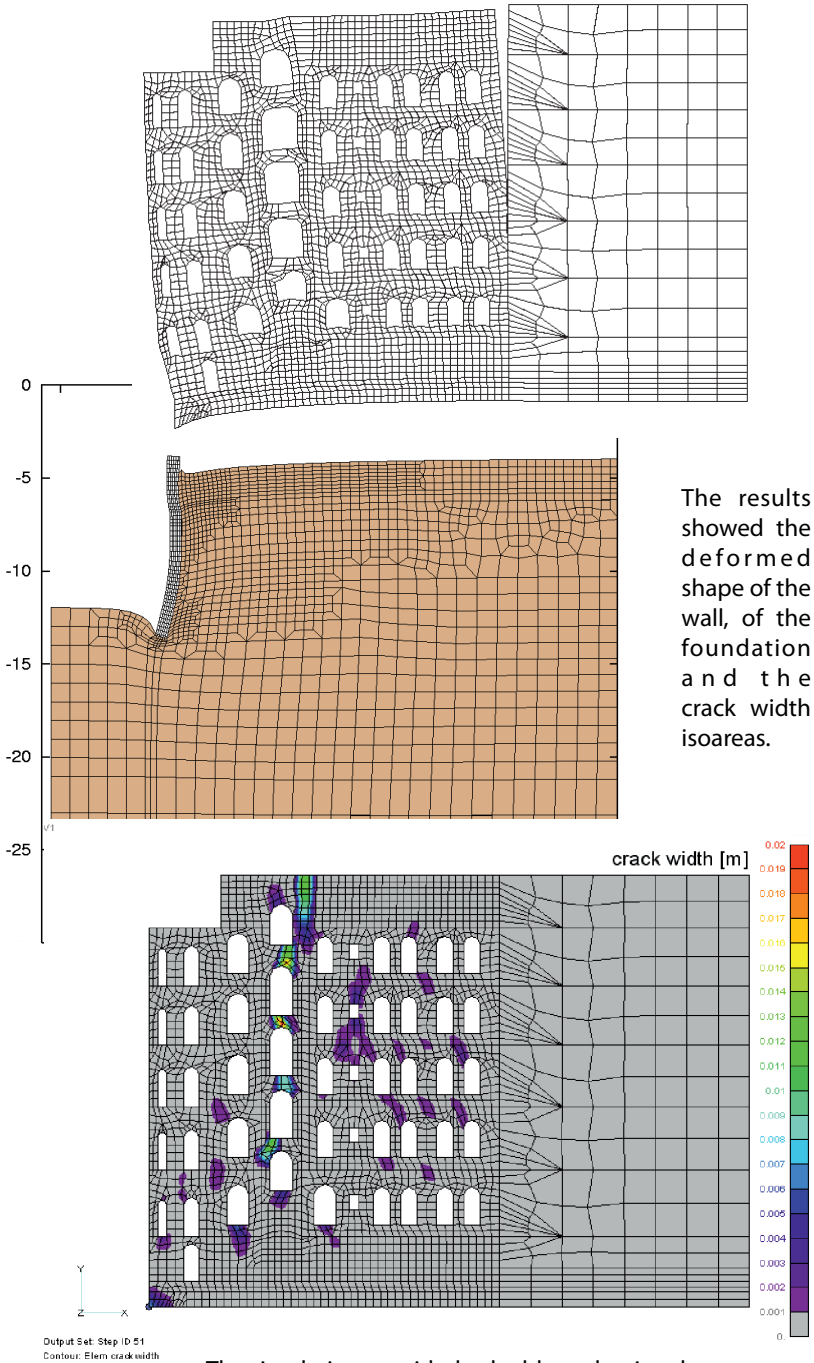
Effect of excavations on cracking of old historical buildings



Building process of new structures that occurs in historically sensitive urban areas can endanger existing valuable buildings. It is an interest of all parties involved in the design process to investigate the effect of construction phases on local buildings.



The front masonry wall of two houses were analysed for the effect of foundation settlement. Due to the extend of the structure, the finite element model was quite coarse, but still managed to realistically capture all openings, contact interface between houses, and foundation layer. The material properties of the masonry were taken from local non-destructive investigation. The masonry was considered an isotropic material with very low fracture parameters. The foundation settlement was calculated in a separate geotechnical analysis and was applied to the bottom boundary of the model step by step. Due to dead and live load, the cracks propagated in the wall as a result of the settlement.



The results showed the deformed shape of the wall, of the foundation and the crack width isoareas.

The simulation provided valuable and rational data for assessment of potential damage.

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ultimate load, pre- and postpeak response, deformations, crack propagation, crack width

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