

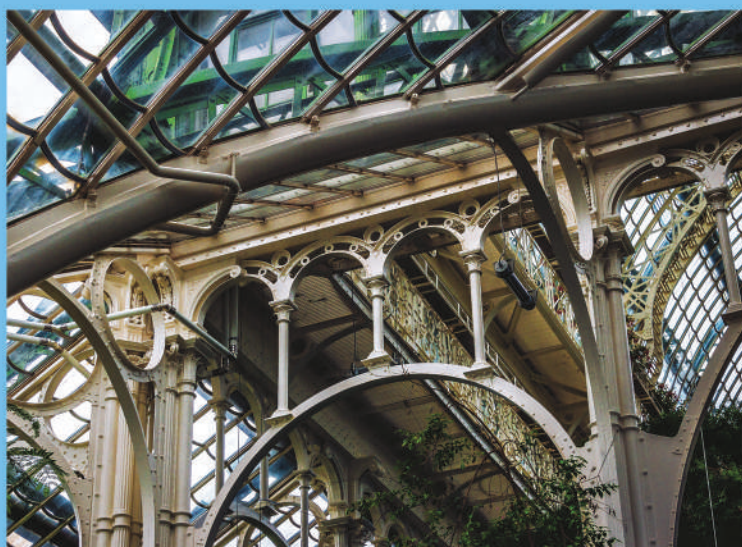


**ДОКЛАДИ  
МЕЖДУНАРОДНА НАУЧНА  
КОНФЕРЕНЦИЯ  
ПО ОПАЗВАНЕ  
НА  
НЕДВИЖИМОТО КУЛТУРНО  
НАСЛЕДСТВО  
БАНИ' 2020**

**21-23 ноември 2020**

**PROCEEDINGS  
INTERNATIONAL SCIENTIFIC  
CONFERENCE  
PRESERVATION  
OF CULTURAL HERITAGE  
BASA' 2020**

**21-23 november 2020**



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

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Scientific Institute for Architecture, Construction and Urban Planning



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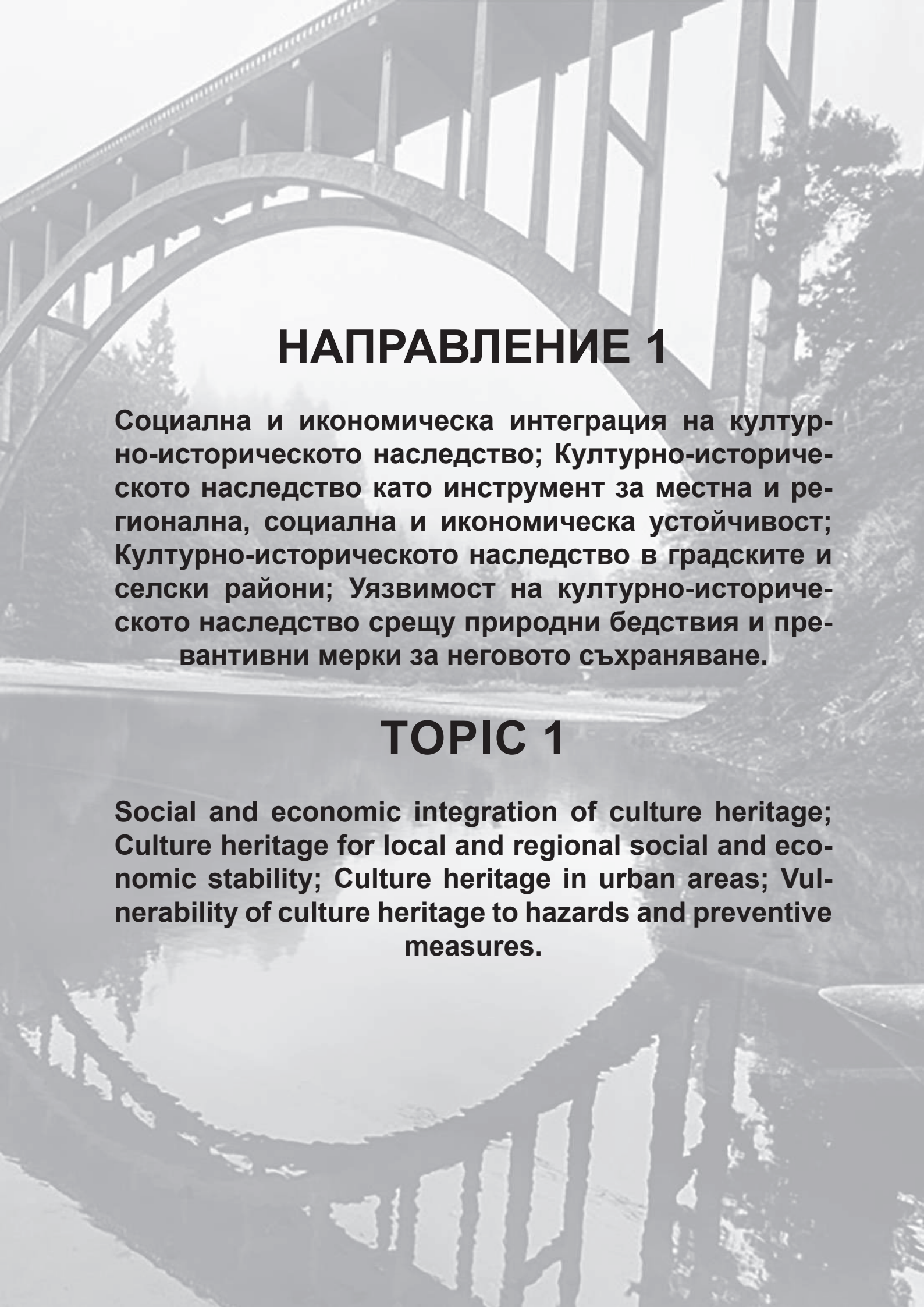
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## **НАПРАВЛЕНИЕ 1**

**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.**



## MANAGEMENT OF AMBIENT PARAMETERS AT THE MEDIANA SITE IN ORDER TO PRESENT IN SITU MOSAICS

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**Abstract:** *This complex was on the list of protected cultural assets of great importance in 1981. In the 1930s the first masonry building was erected to protect the mosaics in the central part of the residential complex. Overall protective structure was erected in 2014 to cover the entire building complex. The protective membrane structure construction is intended to create appropriate ambient conditions in terms of temperature and relative humidity, so that the presentation of in situ mosaics is possible throughout the year. However, due to the poor thermal insulation characteristics of the textile membrane selected as the material for covering the site, the energy efficiency of this facility is extremely low, so controlling the ambient parameters is very difficult.*

*Since the preservation of cultural heritage is a priority, and the mosaics on Mediana are unique in the Balkans, immediately after the construction of the protective structure, it was decided that they remain covered with geotextiles and sand, so that the new ambient conditions would not jeopardize the state of preservation. This paper will discuss what needs to be done and changed on the protective structure in order to create ambient conditions for the presentation of sites and mosaics throughout the year, in order to use the full potential of the protective structure, and realize the original plan.*

**Key words:** *Ambient parameters management model, Protective structures, Mediana mosaics*

### 1. Introduction

The site of Mediana is located on the outskirts of Nis, the former Roman city of Naissus, from which the residence was three Roman miles away. Mediana was built during the time of the Roman emperor Constantine the Great as his summer residence, which was later used by his sons, Constancius and Constase. The period in which Mediana was actively used as a summer residence, but also as a place to live, as shown by numerous auxiliary buildings discovered in the immediate vicinity of the residence is from 306 AD until 361. Naissus and Mediana were destroyed during the Hun invasions of the Balkans peninsula between 441 and 443, although the town and summer residence were later repopulated on a much smaller scale, and archaeological research has established the

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existence of three construction phases on Mediana, which confirms the continuous use of the residence (Jeremić 2005).

As an imperial summer house, Mediana was a complex of luxurious and spacious buildings that cover an area of about 40 hectares, and so far archaeological research has uncovered about twenty buildings, and some of the discovered buildings are a villa with a peristyle that occupies the central part of the complex, spa, wine cellar that is, a warehouse, two early Christian basilicas, a villa with an octagonal tower and large halls. In addition to these buildings, civilian housing facilities were discovered on the site, as well as military barracks, a water tower and a rustic villa not far from the entire complex [<https://narodnimuzejnis.rs/medijana/nggallery/page/5>]. Since this paper will write about buildings with mosaics, ie the newly climatic conditions created after the erection of the protective structure, we will focus on buildings with floor mosaics, of which there are seven, but only the villa with a peristyle is covered (Fig. 1).

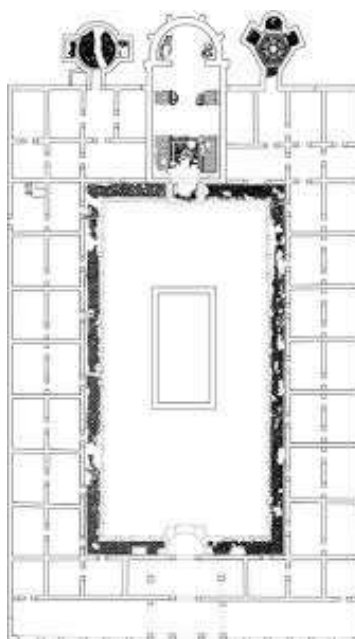


**Fig. 1** Villa with a peristyle above which a protective structure has been erected

The protective membrane construction was partially erected on the site in 2014 (Fig. 2), and completed by about 85% in 2019 (Figure 3). This is not the first protective building erected on the site, namely in 1936 a masonry building of the Mediana Museum was constructed above the triclinium with the intention of protecting the mosaic found in that building. The museum building was incorporated into a new project of a protective structure that was erected above the entire villa with a peristyle, with the intention that the mosaics discovered in the villa could be revealed and shown to the public at least most of the year. However, soon after the protective structure was erected in 2014, it became apparent that the emerging climatic conditions were creating more problems than before, so the mosaics were still covered with geotextiles and sand to prevent their further devastation.

This paper will present the problems and possibilities of their solution so that the mosaics, that are unique in this part of Europe, can be presented throughout the year, but in addition will talk about what needs to be done and how to manage the new conditions at the site to further devastation of cultural heritage would not occur.





**Fig. 2** The first phase of construction of the protective structure (2015)



**Fig. 3** The second construction phase of the protective structure (2019)

## **2. Site management with mosaics**

Mediana is a site where seven different buildings with mosaics have been discovered so far, the largest of which is a villa with a peristyle, which was the central building of the summer complex of Constantine the Great. Each room in the villa had a mosaic with different motifs and different workshop influences from all over the kingdom (Fig. 4). For this reason, the mosaics on the Mediana are unique throughout the Balkans and it is extremely important that they are treated properly, in order to prevent their further devastation. The condition of the mosaics from the moment they were discovered is influenced by atmospheric precipitation, the amount of sunshine, humidity and soil moisture, but also environmental influences, as well as human and animal influences.

There are many ways to present the mosaic in situ to the public and they mainly depend on the climatic area, and one of the ways is to raise the protective structure above the site. However, for the last 10 years, experts around the world have been advocating only temporary protection measures, ie reversible ones. This means that the erection of a concrete structure, in the case of a Median wooden structure with concrete structure plinths, is not recommended for many reasons. Some of them are listed above in the text, and some are very simple and come down to the necessary long-term multidisciplinary research, which costs a lot, before a permanent protective structure is erected above the site. Research conducted by the Getty Institute, the English Heritage and the Israel Antiquities Authority, which investigated the consequences of permanent protective structures erected above mosaic sites in Israel, concluded that more than 30% of mosaics were permanently and irreversibly damaged by new microclimatic conditions (Alef, Neguer, 2011)

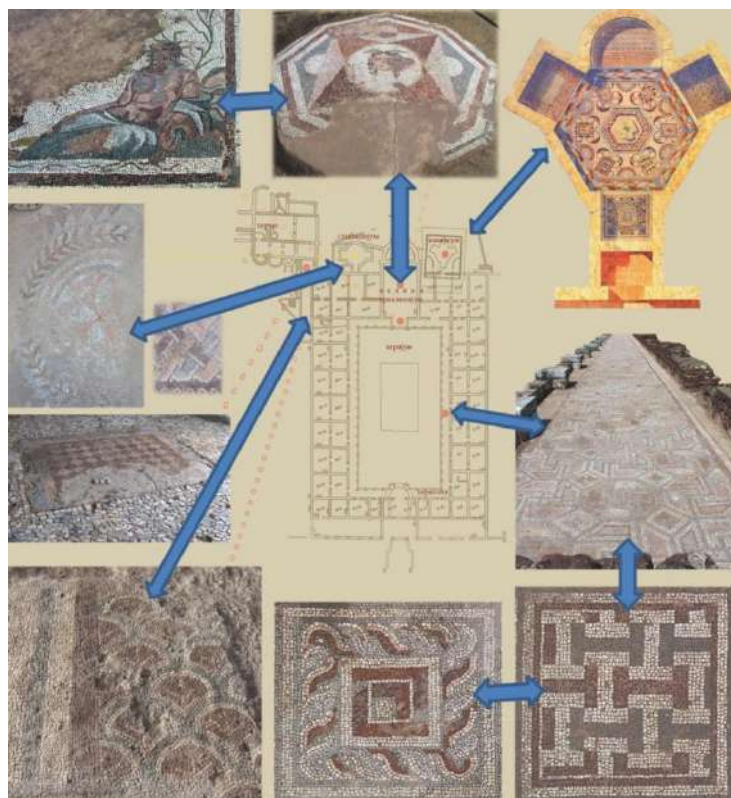


**Fig. 4** Mosaics in a villa with a peristyle above which a protective structure has been erected

In the process of deciding whether it is necessary to raise some kind of protection when it comes to mosaics, it is necessary to make a risk assessment of mosaics in relation to the climate in which they are located, or whether the purpose of the protective structure is to protect mosaics only from precipitation and sun radiation, or they should still be protected from freezing and / or salinity in coastal areas. In relation to what the primary goal is, it is decided whether it is necessary to erect a temporary, semi-closed or completely closed structure, as well as what kind of conservation is needed. In Mediana, this type of pre-research was not done before the erection of the protective structure, ie the primary goal of the protective structure is the presentation of the site, and not its protection.

Pre podizanja zaštitne konstrukcije na lokalitetu sa mozaicima potrebno je uraditi procenu rizika za mozaike, odnosno koji faktori najviše utiču na devastiranje lokaliteta. U procenu rizika spadaju tri glavna faktora, i to su uticaji životne sredine (atmosferske padavine, vlažnost vazduha i zemljišta, vetar, smrzavanje tla), biološki i ljudski uticaji (neprimereno rukovanje lokalitetom, vandalizam, invazivne biljne i životinjske vrste) kao i geomorfološki faktori (zemljotresi, poplave, klizišta). Ovakva vrsta procene rizika poželjna je da se radi i nakon podizanja zaštitne konstrukcije, kako bi se uočilo da li su se neki faktori promenili ili su se neki novi uticaji pojavili nakon podizanja zaštite (Pesaresi, Stewart, 2013). Takodje, pre izgradnje zaštitne konstrukcije mora se uzeti u obzir šta želi da se istakne kod mozaika koji se prezentuju, pa se tako uzima u obzir da li je potrebno istaći vrednost arhitekture i celog konteksta lokaliteta, jer je u tom slučaju bolje lokalitet ostaviti ne pokriven, ili je cilj istaći umetničke osobine ili detalje na mozaiku, jer je u tom slučaju bolja solucija pokrivanje lokaliteta iz razloga lakšeg kontrolisanja osvetljenja, prašine i kao i mikroklimatske uslove pod kojima se mozaici nalaze (Alef, Neguer, 2011).

Before erecting a protective structure on a site with mosaics, it is necessary to make a risk assessment for mosaics, ie which factors have the greatest impact on the devastation of



the site. The risk assessment includes three main factors, namely environmental influences (atmospheric precipitation, air and soil humidity, wind, soil freezing), biological and human impacts (inappropriate site management, vandalism, invasive plant and animal species) as well as geomorphological factors (earthquakes, floods, landslides). This type of risk assessment is desirable to be done before and after the erection of the protective structure, in order to observe whether some factors have changed or some new influences have appeared after the erection of shelter (Pesaresi, Stewart, 2013). Also, before building a protective structure, it must be taken into account what it wants to emphasize mosaics that are presented, so it is taken into account whether it is necessary to emphasize the value of architecture and the whole context of the site, because in that case it is better to leave the site uncovered or the goal is to emphasize the artistic features or details on the mosaic, because in that case a better solution is to cover the site for easier control of lighting, dust and microclimatic conditions under which the mosaics are located (Alef, Neguer, 2011).

All three influencing factors are present in Serbia, changing climate with higher precipitation in some periods of the year, while in the second period the problem of soil freezing and cracking of mosaics, vandalism and inappropriate handling of the site are not uncommon. Floods were common in Mediana until the construction of the protective rampart in the 60s of the last century. Given the great cultural significance of the mosaics on Mediana as well as the different workshop influences visible in the styles of the mosaics themselves, covering the site with a protective structure is a good choice. However, when all this is taken into account the construction of the protective structure had to be approached more seriously.

### **3. Protective structure at the Mediana / Serbia site and site impact assessment**

The construction of protective structures over sites and in situ findings such as mosaics is a complex and multidisciplinary project, which requires a lot of research and monitoring of site as well as the environment for the sake of quality and successful coverage of site. In addition, it is necessary that the protective structures perform the function for which they are intended, and that is primarily the protection of the site from external influences, including meteorological phenomena, and then comes the function of presentation. This is where the biggest problem arises when it comes to all protective structures in Serbia, especially at the Mediana site, given that during the construction of the protective structure, the presentation of the site and in situ findings was placed in the first place, followed by protection. Whether this protective structure performs the job for which it was planned, we can see from the fact that the site has been closed to the public for 5 years, considering that the newly created microclimatic parameters after the construction of the shelter structure endangered and worsened the condition of the site.

Assessing the condition of a site protected by a protective structure is a process that requires a comprehensive approach. In addition to archaeologists, who must be involved in all processes when it comes to the archaeological site, there are also architects, civil engineers, conservators and engineers of other professions in order to cover all the functions of the protective structure and site (Fig. 5). However, this principle is not practiced in Serbia, given that site assessments on a small scale were done only by the Central Institute for Conservation (CIK), and assessments of the effectiveness of the protective structure by the Faculty of Civil Engineering and Architecture Nis (GAF). With such a small volume of research, only problems that occur at the sites immediately after the erection of protective structures could be ascertained, and at Mediana, problems occurred immediately after the partially erected structure (Kostic, et.al.2018)

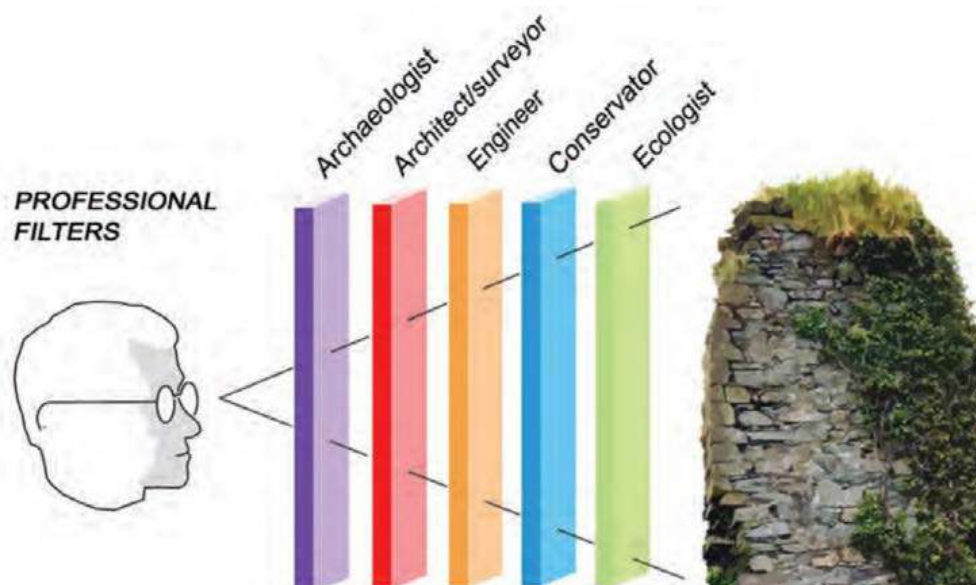


**Fig. 5** Differences in research by different experts and an example of why it is important that they all collaborate on the same project

When things are set up in this way, the question arises as to how and what needs to be done in order for the protective structure to perform the function for which it is intended, but also how to improve and stop further devastation of the site. The protective structure on Mediana is made of laminated semicircular wooden arches over which a PVC membrane is placed which does not descend to the ground level, but a space of about 1 m is left due to the air flow. Given that the site is located in a zone of variable climate, and that differences in temperature and humidity change periodically during the day / year, the biggest problem that arose after placing the protective structure above the villa with peristyle, is the condensation of water vapor on the membrane and metal parts of the structure that later merge to the lower parts, under which there are mosaics (Gligorijević, et.al. 2017). Condensation in the form of continuous droplets falling in the same places constantly during the day / year in the same places leads to accelerated devastation first of the surface layers of the mosaic, and then of the fundus itself, which eventually leads to irreversible damage to the cultural property. Apart from condensation, there are other problems that have changed the microclimatic conditions, such as different wind currents that do not dry the condensation, but create water accumulations, which lead to the development of microorganisms and mold and fungi, which also accelerate the devastation process. In addition, the accumulation of snow on the membrane and the deposits of water that pass through the raised part of the protective structure create additional problems when it comes to site protection and presentation.

Site condition assessment is a repetitive process, and there are several types of site protection assessments, from year to year, at 5 and 10 years depending on the assessment being done and the results they want to achieve. Estimates are performed by archaeologists, conservators and curators, while the condition of the protective structure is assessed by civil / architectural engineers. The assessment is made on the basis of several parameters that are monitored and compared with previous results and these two assessments are linked so that they do not work without each other. The sites are divided into several groups depending on the structure and type of site, and Mediana belongs to the sites with mosaics, so the assessment of the condition should be more rigorous than in other sites, that there are more problems that can affect the devastation of mosaics.

the mosaic fund, if possible, check the condition of the surrounding walls, check the conservation, which is almost always mandatory when are the mosaics in question. It is desirable to monitor the microclimatic conditions throughout the year using devices or probes that are placed on the site in several locations, and which measure air temperature and velocity, humidity, precipitation, and soil moisture. Such monitoring and monitoring of parameters provides insight into the general state of climatic conditions at the microlocation of the site, and it is possible to determine more closely if the new conditions have changed after the installation of the protective structure and whether they do not worsen the condition of the site. Also, such monitoring of the condition of the site can provide the necessary information for further response if it is noticed that the new microclimatic conditions accelerate the devastation of the site, or if it is necessary to react urgently in the form of additional conservation measures to prevent deterioration.

The assessment of the protective structure should be performed at the same time as the assessment of the site, at least once a year, especially when it comes to the site where the mosaics were discovered for the above reasons. The construction assessment contains a visual and effective assessment, ie it is checked whether the construction membrane is not damaged, whether the drainage systems function as intended and whether there is a drainage system at all, whether the structure did not give way and whether the construction joints did not give way. Considering the climatic conditions in which Mediana is located, it is necessary to assess the condition of the protective structure and locality at least once a year, considering the amount of precipitation and humidity, joints on structures corrode very easily if metal couplings are used on the wooden structure, because depending on the type of wood and the construction itself, it is possible that after a certain time, if the wood is poorly processed, it starts to rot, and then the protective structure is not safe for use, let alone for site presentation in situ mosaic.

According to the conditions set in this way for the assessment of the condition of the site and the condition of the protective structure, preliminary conclusions can be reached whether it is necessary to change or repair some parts of the structure to make it more energy efficient, and to perform the function of protection and presentation site. According to a study conducted in 2017 by the Faculty of Civil Engineering and Architecture in Nis, which dealt with the assessment of the condition of the protective structure at the Mediana site, but also with the assessment of the condition of the site, it was concluded that the energy efficiency of the protective structure at the site is extremely low. During the study they did, an evaluation system was used where the Site Health Index can be calculated from the collected data, based on when the management system of the protective structure is further developed, ie further measures can be taken in order to protect the site. According to this study, the Archaeological Site Health Index at the Mediana site is by far the largest of all sites covered by protective structures in the whole of Serbia, and amounts to 157 (Gligorijević, et.al.2017). The results were obtained by applying the model of Markov chains, ie the method by which it is possible to determine the possible condition of the protective structure over the years, so the management of the site can be adjusted to such findings, as well as preventively secure and preserve the mosaics and walls of the villa to avoid sudden devastation.

#### **4. Management of ambient parameters in order to present site and in situ mosaics**

With the help of studies that have been done, although individually and on a small scale, it is possible to conclude that the events since the construction of the protective structure above the Mediana site have not yielded satisfactory results, even in the presentation of the primary goal of the structure, nore to site and mosaics. This is not only

the case with the Mediana site, but we encounter this problem all over the world, because there is no communication between experts from different domains, as in most cases there is no cooperation and complex research before the protective structure is erected over the site they try to correct mistakes only after they are noticed (Alef, Neguer, 2011). This means that the accelerated devastation of the site has already occurred and that research into the impact of the protective structure is only trying to mitigate and slow down the already accelerated deterioration of the site. For this reason, studies are needed on how to use the potential of the already built protective structure that endangers the site with the newly created microclimatic conditions, and turn it in favor of protection and presentation of the site with modern technologies and remediation, to create an approximately ideal ambient environment within the shelter. .

In order to manage the ambient parameters at the site, more detailed research was needed, including results that now have possible structural changes and emergency remediation for known problems. First of all, it is necessary to change the protective structures to manage the ambient parameters, and to reduce the costs of maintaining the structures and sites drastically. It is extremely important that it also implements monitoring of ambient parameters and that the data is monitored in real time so that it can be temporarily determined if necessary (Fig. 6). In the current situation, the costs for conservation are too high, considering that it is necessary to renew and repair mosaics much more often than was the case before. Precisely for that reason, the mosaics on Mediana have been covered with geotextiles and sand for 5 years.

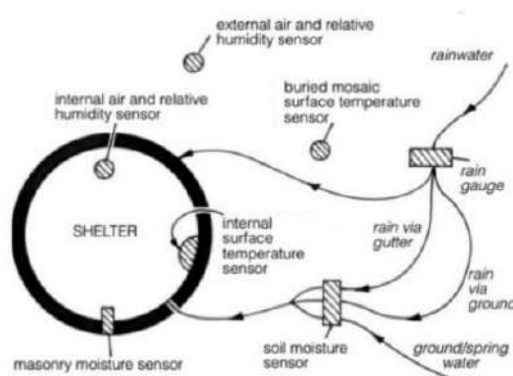


Fig.6 Scheme of monitoring ambient parameters

Ideal conditions for the presentation and protection of mosaics probably do not exist, however, approximately ideal conditions can be created for the purposes of presentation, using the technologies we have today and the data we have after research. Roman mosaics are made to be used on a daily basis, so the materials used for their surface making are quite resistant and solid, and they are mostly stone and glass paste, however, the biggest problem with the mosaics we have on Mediana is the fundus below the surface layer. The fundus starts to crack during changing weather conditions, due to the expansion and contraction of the material, and after that it starts to sink, and finally the surface layer of the mosaic collapses. To prevent this, the conditions under which the mosaics are located at the Mediana site should not vary drastically depending on the time of year. This means preventing drastic temperature variation by periodically opening and closing part of the protective membrane depending on the season, as well as preventing soil freezing, which is the factor that most endangers mosaics. In order to prevent rain and snow deposits under the part of the protective structure that is open, as well as the flow of rainwater through the membrane, that part should be closed, since water collects and falls directly on the parts with mosaics. In addition, it is necessary to solve the problem of excessive condensation of

water vapor on the membrane and metal parts, which further increase the humidity of the air and soil at the site. This problem could be solved by opening additional ventilation spaces at the top of the protective structure, in order to allow air flow, which would dry the membrane and wooden parts of the structure, in order to create ambient conditions without excessive humidity. In addition, it is possible to remodel the protective structure in order to add another layer of membrane to reduce the level of condensation to a minimum or not at all, and such a change in the structure would have an insulating property and temperature variations decreased (Kostic, 2020)

In addition to these main problems that create the greatest difficulties during the maintenance of the site and the protective structure, there is also a non-existent drainage system that makes things even more difficult. The drainage system is very difficult to carry out at the archeological site without additional permanent destruction of the cultural layer, but it is very necessary when it comes to this type of protective structure. All the water that flows through the membrane, as already mentioned in the text, flows directly to the parts with mosaics, which is a huge problem when the time of year comes when temperatures are below 0°C and when the soil starts to freeze. For this reason, it is extremely important to make the sides with the possibility of their opening / closing, as well as opening the lantern on the highest part of the shelter to enable the necessary transverse air circulation using appropriate thermo-technical measuring probes and circulation control automation. Also, the urban project needs to design a drainage system that does not endanger the site. The protective structure in its current state requires additional investments and correction of the project solution (Kostic et.al, 2019). After the alterations and the application of the control of the ambient conditions, relatively acceptable ambient conditions for the presentation of the site would be acquired, and the investments would pay off many times over.

## **5. Conclusion**

After the facts presented in this way, it is obvious that a lot of research and investment is needed for the mosaics on Mediana to be shown to the public throughout the year, but this is not impossible. Redesigning the protective structure can seem like a big project, however, soon after the changes, the condition of the site and mosaics would be improved, investment costs around the current maintenance of the structure would be reduced and conditions for further archaeological research would be created. Creating approximately ideal conditions for the presentation of mosaics is possible by adding an additional layer of membrane, opening additional ventilation spaces, introducing a drainage system as well as a central system for monitoring ambient parameters, so that it can react in a timely manner.

## **Acknowledgment**

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## THE ROLE OF MULTIETHNIC AND MULTIRELIGIOUS CULTURAL HERITAGE FOR THE SOCIAL STABILITY OF THE VOJVODINA REGION

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**Abstract:** *Because of cultural and religious diversity the regional care for heritage in Vojvodina have to introduce new heritage categories which have emerged in recent years that are strongly linked with intangible values: cultural itineraries, symbolic sites, cultural landscapes, and places of remembrances. By means of some concrete examples it could be examine to what extent it is possible for various intangible references to be taken into consideration or how more account might be taken of them through the regional planning: spiritual, political, or social references; elements relating to the origin of human or to migration movements: artistic aspects of communal places; or natural sacred sites. All of the above is observed in the function of achieving the necessary social stability that is in multinational and multi-religious environments such as Vojvodina Region.*

**Key words:** *Vojvodina Region, National and religious minorities, cultural heritage, evaluation, social stability*

### 1. Introduction

Already in the second half of the 20<sup>th</sup> century, the architectural heritage was viewed not only as a cultural and historical value of a region, but also as a significant factor of social stability. This is especially true for areas with a large number of national minorities, such as the Autonomous Province of Vojvodina within the Republic of Serbia. Due to its position, this region is considered within Serbia, but also within Europe. When it comes to architectural heritage, a special attention is paid to strengthening cross-border ties between municipalities and cities within European projects as well as connecting with other regions, especially Central Europe. Of all the interregional projects and strategic cooperations, the most important is probably the transnational project around Corridor VII, defined along the course of the Danube River.

A key role in the development balance of this region is played by dynamic and strong urban centers, in functional connection with their environment. The bearers of that integrated and sustainable social, economic and ecological development are the macro-energy centre Novi Sad, then Subotica, Sombor, Sremska Mitrovica, Kikinda Zrenjanin, Pancevo and Vrsac. All these urban centres are at the same time cities with significant

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historical cores and valuable historical buildings. These cities developed and strengthened in the past because they were created by the logical choice of a good position in the environment, on the axes of the Danube and Tisza rivers or along natural communications observed as early as during the Great Migration through the Pannonian Plain. Therefore, for the strategy of spatial development of Vojvodina, it is important to determine and evaluate those historical characteristic corridors, areas and points of regional identity, through which this region is recognized, affirmed and identified as a factor of social stability at the regional, national and international level. Parts and entities of cultural and natural heritage listed according to international and national standards (UNESCO, Ramsar Convention, etc.), as well as the heritage that is in the process of prior protection are of special importance due to the potential of their integral use within cultural landscapes. Extremely valuable combinations of the baroque-classicist town of Sremski Karlovci at the foot of Fruška Gora, a cultural and natural landscape with over 16 monasteries, vineyards nurtured since Roman times and other attractive features,; the ancient city of Sirmium (within the modern city of Sremska Mitrovica) with the Special nature reserve Zasavica (with a unique species of *aldrovanda vesiculosa*, the carnivorous waterwheel plant, which grows in only 50 places in the world); then the castles of Banat and Bačka in park environments, industrial heritage such as gates on rivers and canals, some of which were designed in the Eiffel Bureau are standing out in particular. All these characteristics of the mentioned spatial, cultural and historical units, belts and reference points are of great importance for the social development and its stability in Vojvodina. Strengthening the role and significant soft factors for overall social stability refers to the natural and cultural heritage, as well as landscapes. At the same time, these factors that Vojvodina has in abundance represent the identity of this region and a comparative advantage, as well as a long-term economic benefit, as one of the most important resources for spatial development and social stability, (Fig.1.) [12][8].

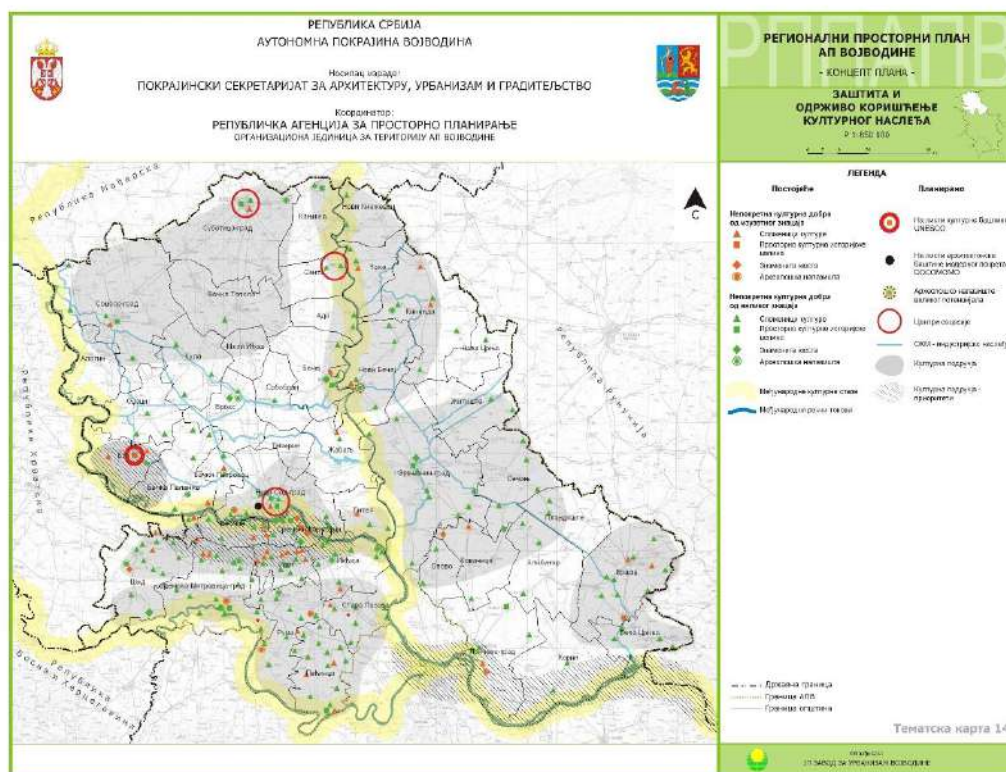


**Fig. 1.** Regional Spatial Plan of the Autonomous Province of Vojvodina until 2020 - sustainable use of cultural heritage (after [12])

After the general definition of the significance of the extensive cultural and natural heritage, one of the most important reasons why the historical background of the spatial development of Vojvodina is necessary for the realization of the social stability of this region should be emphasized. In Vojvodina, in addition to Serbs, today there are over 25 national or ethnic communities, which together make up 35% of the population of Vojvodina. Based on this data, it can be concluded that the past of this region was very turbulent, and that the cultural heritage preserved to this day is the result of diverse, heterogeneous, layered influences of the inhabitants, some of whom settled permanently in the region and some only temporarily, but sufficiently to leave traces of their material culture [9].

## 2. Cultural influences in Vojvodina through history

The archaeological heritage of prehistoric cultures, as well as the early historical heritage of the Illyrians, Thracians and Celts, will not be considered here. The Roman heritage is undoubtedly very important. One of today's cities literally covers one of the four late Roman capitals, Sirmium, so the presentation of Roman remains is a significant potential for tourism development. Six Roman emperors were born in and around the city. Slavs, including Serbs, began to settle during the sixth and seventh centuries, and in the ninth century the region was ruled by Bulgarian dukes. At the end of the twelfth and the beginning of the thirteenth century, a part of Vojvodina was included in the Kingdom of Srem by King Dragutin Nemanjić. From the middle of the fifteenth century on the territory of Vojvodina, Serbian despots ruled as vassals of Hungarian kings. The Turkish conquest during the sixteenth century extended across the Danube all the way to Buda in Hungary and left considerable traces of oriental urban culture (Fig. 2) [12].



**Fig. 2.** Hamam – remains of the Turkish bath in Bac (after [15] [16])

The people of Western Europe breathed a sigh of relief at the end of the 17<sup>th</sup> century, after 1683, when the Turks were defeated at Vienna, but in the Pannonian Plain two great powers, Austria and Turkey, continued to wage wars with changing war fortunes. Although from 1718 the Turks no longer controlled even of the remnants of their conquest north of the Sava and the Danube rivers, the border between the two empires continued to be fragile and difficult to establish. As a rule, war times alter the structure of settlements in the area exposed to destruction. The eighteenth century is certainly the most important one for the creation of urban structures in the region, those that today form the historical core of most cities in Vojvodina. It is difficult to enumerate, analyze and explain all the strange forms in which the oriental and baroque models of the city intertwined. It can only be stated, that for the region of Vojvodina, this is up to date the best testament of the strength



and vitality of built settlements which persevered in various spatial forms and survived different architectural concepts.

The eighteenth century is certainly the most important for the creation of urban structures in the region, those that today form the historical core of most cities in Vojvodina. It is difficult to enumerate, analyze and explain all the strange forms in which the oriental and baroque models of the city intertwined. It can only be stated that to have been maintained in various spatial forms and to have survived different architectural concepts is the best evidence of the strength and vitality of the built settlements in the region of Vojvodina to date. The 18<sup>th</sup> century was a time of intensive settling as colonizers from different parts of the then powerful Austrian Empire under Maria Theresa and her successors. This settling process was accompanied by the implementation of different cultural patterns in the architectural typology. The Danube, a mighty river that flows through Germany, Austria, Slovakia, Hungary, Croatia, Serbia, Romania, Bulgaria, served not only as a waterway, but also as a link between the people who lived around it. It is complicated, therefore, to conjure up the significance of the Danube throughout history, since it was alternately a political and military obstacle or salvation for thousands of unfortunate people who, sailing on ships and rafts, sought space for settlement and a better life. The most delicate point of study is to determine the scope of cultural ties, intertwining, influences, the struggle against the imposition of cultural influences. The propagation of influences, which are the backbone of the culture created in the area along the Danube, is so wide that sometimes it has to be accepted with disbelief. In that context of turbulent history, the built heritage of various cultures today represents the main support helping to preserve the identity of numerous nations and minorities and the social stability that enables them to coexist today [4].

All present-day habits and traditions, from housing, music and cuisine to institutions, are based on the superstructure of the cultures of the Danube countries, as interpreted and used by numerous ethnic groups inhabiting the Danube countries and religions that exist in those countries. That is why "today" the Danube region is the end product of an unbroken chain of various "yesterdays", which are firmly integrated into everyday life and more accessible as a way of life to people than any other.

From today's perspective, it is difficult to imagine the drastic changes that occurred during the 18<sup>th</sup> century. Turkish urban planning of oriental type replaced by baroque, central plan architecture with *selamluk* and *harem* replaced by an enfilade of chambers with decorated galleries, *zurle* replaced by spinet, Suzi-celebi replaced by Bucholz's, Grimmelshausens.

This is just a brief view of the time when cultural influences in the Danube region still came equally from the south, and not only from the west and north. When this is observed in a broader time frame, then it can be seen that the influence from the south is slowly fading and that today the culture of the Danube is fashioned after the models of the west, north and east, and that the connection with the south is traditionally considered as retrograde.

Spirituality, however, has never been uniform in the Danube region. Even when it came to large, unitary states whose territories far exceeded the wider borders of the Danube region, religions were numerous and accepted with more or less tolerance. We are inclined to consider the religious problems that existed in the area of the Danube region in our part to be extremely difficult and still insufficiently studied.

The seriousness of the problem certainly lies in the fact that there have always been two or more opposing religions. Occasional gathering of all the peoples of the Danube region around a single task, and then parting ways and renewed mutual attrition is a characteristic of this region. The level of civilization nowadays does not permit such



drastic changes. The same can be said about ethnic displacements and vice versa about living together in peace and harmony. There are no rules and there will be none [6].

Spiritual independence, however, is something completely different and there are differences in this area that ought to be nurtured. Even if only Slavic peoples were considered, it could be noticed that due to various circumstances, they have different spiritual preferences. There are similarities in languages, habits, anthropological characteristics and a number of other elements, but in some cases spiritual differences separate them to the extent there is hatred between them. It is, however, a feature that makes the whole Danube region so different from other parts of Europe. There are similar problems that occasionally arise, but religious affiliations have had a continuous development, while here this topic, especially in the period of socialism, was pushed aside, marginalized or suppressed, only to be revived in a specific way today. Throughout history, it has been shown that cultural influences could flow independently of religious ones, so this form of connection along the Danube should be adhered to in the future as well [3].

### **3. The problem of evaluation of the cultural heritage of minorities and the importance for social stability in the region**

Based on this historical overview of the pervading and partial merging of numerous cultures in the area of Vojvodina, it is clear that today there is an extensive and diverse architectural heritage, more or less preserved.

Regardless of the technical protection related to the problems of material presentation, based on the presented dynamic fluctuation and settlement of different ethnic groups, there arises a much more subtle problem of how to achieve social stability in Vojvodina in the process of evaluation of such a diverse heritage. Each of the groups, religious or ethnic, has the right to realize its identity by preserving buildings and historical entities that have a certain value for them. But society as a whole should establish criteria on the basis of which it will evaluate cultural heritage according to other characteristics that are internationally accepted as important, sometimes crucial. The selection of these characteristics was achieved through careful and long-term selection, by gradually including new properties and expanding the list to include precisely those characteristics that are in the intangible domain. This means that it includes everything that has been preserved over time and that has a certain value for people, which can be different. A value for which there is a public interest, and which is observed in a building, group of buildings or settlements, and which could be destroyed for various reasons, is clearly defined. This value determines what separates a building, group of buildings or settlements, according to most people, from other entities of one area, time or spiritual flows. An individual cultural property may have one or more superimposed characteristics, and then their sum increases the total value of the property (scientific-documentary, historical, artistic or artistic-artisanal, cultural, ethnographic, architectural, archaeological, structural and building-structural, urban, rural, ambient, value of the original, value of the unique specimens or collections, value of antiquity, *intangible spiritual value*) [14].

Each of the ethnic or religious groups inhabiting Vojvodina has, over time, built their places of worship, residential and public architecture, formed settlements according to their living habits. Each of these groups, settling in the area of Vojvodina, brought cultural patterns from the area from where it was displaced. Time ran its course, the groups mixed more or less and, in the process, created a mixture of structural and architectural forms [11].

What, then, is the problem when it comes to evaluating the architectural heritage of Vojvodina? Here the spiritual characteristics of cultural properties come to the fore.

Religious and ethnic groups came there, at different levels of cultural development, possessing more or less finances, more or less building talent, with longer or shorter presence in the area, with more or less repulsive attitudes towards the influences of building traditions that coexist and towards coexistence with others with whom they share the area of Vojvodina in general. How this was resolved in the past is known from historical documents and material remains. These testimonies indicate that in the past, disagreements among the settled groups were resolved by destroying religious buildings, burning houses, and even insulting conversions. In the church of the Franciscan monastery in Bač, the *mihrab* and the *kurna*, remnants from the time when the Catholic church was turned into a mosque, testify to such a transformation (Fig. 3).



**Fig.3.** Turkish mihrab and kurna – Church of the Franciscan monastery in Bač (after [17] [18])

Yet, for the present-day role of the architectural heritage in the implementation of social stabilization, examples from the recent past are more interesting. During the reign of the Austrian Empress Maria Theresa, a significant number of Hungarians, Germans, Slovaks, Dutch, and a few French people settled there. The area of Vojvodina was colonized in a systematic manner inhabited, and due to this extensive colonization, Vojvodina became one of the most ethnically mixed regions in Europe. The German population, mostly from Bavaria, Saxony and Swabia, built houses after the model of those they left behind. In addition to the characteristic floor plan of the so-called "long" houses, known in architectural language as the "Swabian" house, they designed facades with a finish known as the "Nuremberg gable". The privileged position they enjoyed enabled their faster economic prosperity, and this resulted in a higher level of housing culture (Fig. 4).



**Fig. 4.** . Old houses in Sremska Kamenica with so-called "Nuremberg gable"

After World War, followed a new great colonization of Vojvodina by the population from the poorer regions of the newly created Socialist Yugoslavia. The German population was expelled, the population of the lower housing culture moved into the "Swabian" houses and it resulted in the systematic ruining and destruction. Today, these houses are part of Vojvodina's rich and layered cultural heritage, but there is still some prejudice in a part of the public concerning the former owners, so that architectural heritage must be legally protected so that it is not completely devastated. This form of protection is twofold, the valuable material heritage of Vojvodina is protected, and at the same time social stability is maintained, i.e., the young generations are shown the need to respect different cultures [9].

Another problem that arises in multinational and diverse religious groups is the objective assessment during the evaluation, especially of sacral buildings. It is in human nature to strive for a certain form of equality that is not only reflected in legal and social equality, regardless of the number of members of the group and regardless of the objective value of the architectural heritage it inherits. This is, in fact, the segment that burdens the evaluation of cultural properties the most, because the share of spiritual value needs to be weighed and balanced and connected to other material characteristics. This is undoubtedly a very difficult task for conservators in regions with many ethnic and religious groups such as Vojvodina. However, this process cannot be spontaneous, but certain rules should be established by which conservators are guided when recognizing and determining the value of those buildings and units that will be declared a cultural property. The starting point is the internationally accepted principle that the peculiarity of architectural heritage lies in the fact that its value does not refer only to the physical structure, but that its value lies in what it means to people. Thus, the key to understanding the architectural heritage of people is their simultaneous assessment of the past, present and projections of values in the future. On that basis were listed the cultural goods in Vojvodina. Long-term field, and archival research of material remains and careful determination of the identity of the groups to which that heritage belongs, produced proposals for their protection and they were finally legally declared.

But, the next level of protection, declaring a cultural property to be of great importance, and then, possibly, of exceptional importance, is a real challenge for every conservator. This is where the skills of noticing all available characteristics of a cultural property and a comparative analysis of values with the properties of other ethnic or religious groups come to the fore. For example, almost all religious groups have the same or very similar architectural models and have been influenced by European dominant style trends. The Orthodox Church received privileges in the Austro-Hungarian monarchy, but under certain conditions. The shape of the church building was to be adapted to European architectural styles. As a result, the medieval tradition of Serbian church architecture formed under the strong influence of Byzantine architecture was abandoned. Thus, in settlements with a mixed population, churches of the Catholic and Orthodox faiths were built with a very similar outer shell and decoration, but with a different internal spatial arrangement, depending on the type of worship. Not all communities, however, were of the same numbers or equally wealthy. The two largest communities, Orthodox and Catholic believers, were able to invest more in building their places of worship than others. They also stand out in terms of number in the list of cultural properties of exceptional and great importance (Fig. 5).





**Fig.5.** The Orthodox Cathedral of Saint Nikolaj and the Roman Catholic Church of the Holy Trinity stand close together in the center of Sremski Karlovci (after [19])

Yet, some smaller and closed groups were very powerful to build works of exceptional creativity and beauty, so they were rightly classified as cultural properties of exceptional values, such as the synagogue. The condition of that building in Subotica after the Second World War was getting worse, but, despite the highest status of that cultural property, the state did not react energetically and invested very little in the reconstruction. This synagogue was on the list of the 100 most endangered cultural monuments in the world (World Heritage Watch). Only after more than four decades of fundraising and joint effort of the city of Subotica, the province of Vojvodina and the Serbian government was the renovation of the building's exterior financed, while the interior restoration was financed by the Hungarian government. The pooling of funds for financing the renovation of certain cultural monuments of the state administrations of the country in which the cultural monuments are located and the countries of origin of ethnic or religious groups is thus becoming an increasingly common way to preserve them, thus achieving the social stability in Vojvodina (Fig. 6) [7].

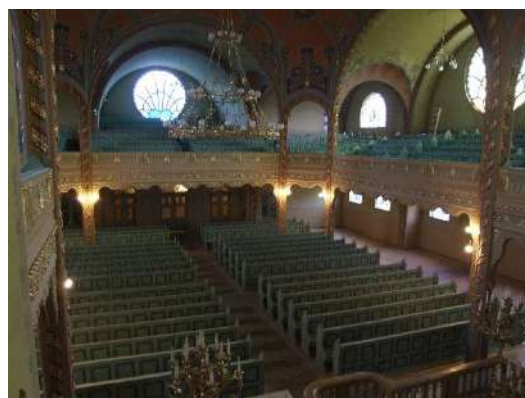


**Fig. 6.** Synagogue in Subotica (after [20])

The latest example, finished in October of 2020, is the handover of the renovated birth house of Croatian Ban Josif Jelačić in the suburbs of the Petrovaradin Fortress, across from Novi Sad, to the Croatian National Council. This happened after seventy-five years of

unsuccessful claims of the Croatian minority in Vojvodina, because the house was inhabited by several families, who demanded large compensation for displacement, even though they lived in ruins. At the opening, the President of Serbia said that it was "the most concrete way" to solve the problems of the Croatian community and that it sent a clear message that members of the Croatian nation are the full-fledged citizens of Serbia. This very elegant baroque building dating back 1745 thus became an active participant in the social stabilization of the region (Fig. 7.).

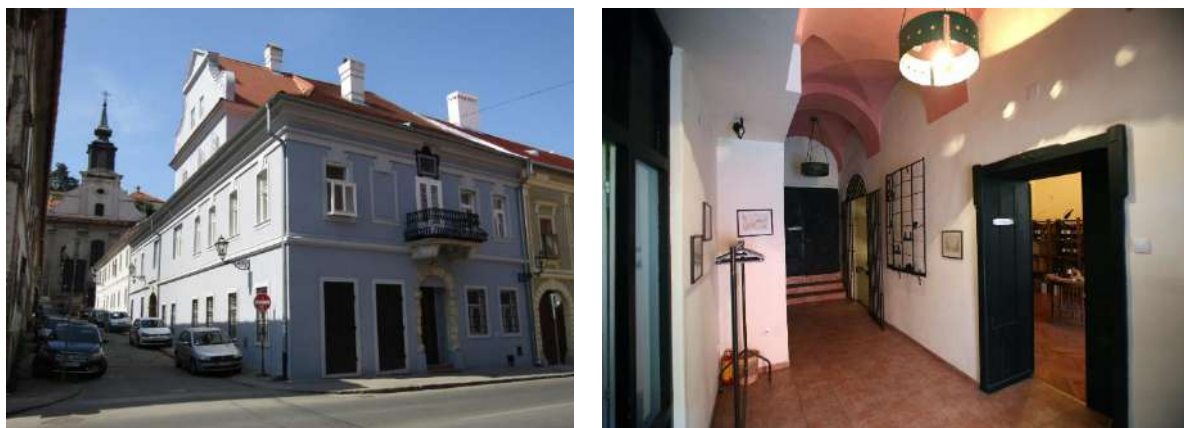


**Fig. 7.** Birth house of Croatian Ban Josif Jelačić in Petrovaradin (after [21])

The largest and most active ethnic minority in Vojvodina today is Hungarian. Historically, Vojvodina belonged to the Austro-Hungarian monarchy until the end of World War I, so the number of cultural assets of both folk and civic architecture is very large. As a rule, after the loss of some territory, the inhabitants of the country that lost it move out and leave their homes, and the winners begin to use them. The attitude towards something that belonged to someone else is often careless, so a large number of buildings erected by the Hungarians in the area of Vojvodina was exposed to decay.

The neglect was particularly pronounced during the period of socialism. In the early 1990s, there was a change in treatment, signing of international agreements began, and in the 21<sup>st</sup> century, cross-border projects became a regular way of cooperation between conservators. The Hungarian state invests in the restoration of many cultural monuments known to have been designed by Hungarian authors or public and religious buildings erected during the time when Vojvodina belonged to the Austrian, and afterwards to the Austro-Hungarian monarchy. The renovation of these buildings undoubtedly has the effect of social stability strengthening (Fig. 8) [7].



**Fig. 8.** National rural heritage of Hungarians in central Banat (after [22])



In addition to the Hungarian and the already mentioned Croatian minority, the settled Slovaks, Ruthenians and Romanians also have a more extensive cultural heritage. The value of this heritage is primarily in the specific residential architecture that is preserved in the places where they are the dominant inhabitants. In the first half of the 18<sup>th</sup> century, Slovaks migrated to the western part of Vojvodina, so Bački Petrovac became the largest Slovak settlement in Vojvodina during the 18<sup>th</sup> and 19<sup>th</sup> centuries. Afterwards, the inhabitants moved to other parts, and during World War II, many returned to what was then Czechoslovakia.

Slovak residential architecture is especially characterized by vivid colours on the street fronts of buildings. What dominates is the intense blue colour of the details on the white walls, the details being all renewed monthly in some villages, while once a year, they are renewed obligatorily. There are also opposite examples, the entire facade is painted azure blue, and the details are highlighted in white (Fig. 9.)



**Fig. 9.** | Slovak houses in Bački Petrovac and in vicinity (after [23])

The Slovak community used to be compact and isolated in some settlements, but that is no longer the case today. However, in many places they still nurture their very richly developed cultural life. In the town of rural tradition, Kovačica, near Belgrade, there is a tradition of naive painting. A special curiosity is that women are more numerous and more famous painters. Their paintings have become the global heritage and they are printed on UNICEF New Year greeting cards and can be found in the most famous museums in the world. The interest of tourists for this village, visits to the families of women painters and the Gallery of Naive Art is great and significantly contributes to the economy of Vojvodina (Fig. 10).



**Fig. 10.** Kovačica naïve painters – E. Cizikova, Interior of Slovak house, 1997; Z.Chalupova, The Lovers, 1995, indicating the use of intensive blue colour in architecture

Although the urban influence had an effect on the recent architecture of the place, naive painters have left us a lasting testimony to the Slovak rural tradition of building and their way of life. In addition to Slovaks, there are residents of other religions in Kovačica, so evangelical, Catholic and Orthodox churches were built at a short distance from one another. In other settlements with a predominantly Slovak population, emigration to Slovakia is becoming more and more widespread, so the architectural heritage is increasingly left to decay and transformation of its original appearance and function. Emigration, however, is not caused by the social marginalization of the minority, but by the unemployment that affects other groups of people, too.

Ruski Krstur is the oldest Ruthenian settlement, created in the middle of the 18<sup>th</sup> century, under the administration of the Austrian monarchy which governed the settling. The old homeland of the Ruthenians is in Carpathian Russia, i.e. part of Ukraine, on the border with Poland, Slovakia, Hungary and Romania. Ruthenians are of the Greek Catholic faith, their oldest cathedral being the Cathedral of St. Nicholas from the 18<sup>th</sup> century in Ruski Krstur. It is also the only Greek Catholic parish in Serbia, where the holidays are observed according to the Gregorian calendar (Fig. 11).



**Fig. 11.** Greek Catholic Cathedral of Saint Nicholas in Ruski Krstur (after [24])

Since they are few in number, they suffered many influences from other inhabitants of Vojvodina. Nevertheless, they are managing to keep the cultural identity, although in the architectural sense it is indistinctive. The president of the Association of Local Communities Ruski Krstur has expressed this very accurately, by stating that "Every custom has its own symbolism and its own holiday, in its own way it has its own meaning, I don't know what I could point out, but I know there are differences regarding other nations, and that is what we are succeeding in maintaining." Ruthenians as a minority, although socially well-received and many of whom are engaged in public administration, tend to emigrate and there is fewer and fewer of them. Young generations, due to unemployment and dissatisfaction with life in small towns, are emigrating in large numbers to Canada (Fig.12).





**Fig. 12.** The oldest Ruthenian houses in Ruski Krstur (after [2])

The Romanians stand out among the minority communities that have achieved a good integration into the social community of Vojvodina, i.e., Serbia. Although they are present in all major places in Vojvodina, they are still grouped in large numbers in the South Banat district, where even today there are settlements with predominantly Romanian inhabitants. The town of Uzdin, whose origin is related to the formation of the Military Border, which defended Banat along the Danube, from Titel to Orshava and Karansebes, preventing an invasion of the Turks stands out for its importance. Over time, the Romanian community has preserved its spirituality, both in an institutionalized environment and in a family circle. In Uzdin, the church, school, cultural centre and various associations are completely Romanian. The Romanian Orthodox Church of St. George in Uzdin, which was built by Romanian immigrants in 1801 represents the greatest architectural value. This temple is one of the largest and most luxurious places of worship in Vojvodina, with an iconostasis in the Biedermeier style and paintings by the famous Biedermeier Serbian painter Konstantin Danil. The church is classified as a cultural monument of exceptional importance, which represents the highest level of value within the state of Serbia.



**Fig. 12.** Romanian Orthodox Church of St. George in Uzdin (after [25])

#### 4. Conclusion remarks

One of the basic features of Vojvodina is multiculturalism, ie a high degree of coexistence of different ethnic communities in the same area. In addition to the majority nation, there are numerous national minorities distinctly differentiated in terms of demographic development, socio-economic, historical and cultural-civilizational characteristics, national emancipation and political organization.

Social stability is ensured through collective acts within religious and social activities. Overall social stability depends on how an individual will fit into the norms and values of the society in which he lives. The greater the number of integrated individuals, the better its societal balance and more harmonious its nature, which prevents its instability and general dissatisfaction [5].

The spatial distribution of minorities is characterized by spatial dispersiveness or ethnic compactness, i.e., the pronounced process of ethnic homogenization conditioned by numerous factors. Good interethnic relations, respect for the rights of minorities, integration and loyalty of minorities to the country in which they live, are necessary for a balanced demographic, stable social, economic and political development of Vojvodina

In a society with a large number of heterogeneous social and religious minorities, such as Vojvodina, the role and importance of cultural heritage are extremely important for its stability. In some cases, such as the birth house of Croatian Ban Josip Jelačić, the dissatisfaction of the Croatian national community with its treatment in Vojvodina to date has been greatly alleviated by the renovation of the house. Architectural heritage, i.e. building styles, spatial organization due to the specificity of the functions that once existed, authors who were hired from their home countries are nowadays important for the sense of identity of minorities. At the same time, the fact that minorities accepted and repeated the building styles of other social communities, that they hired famous artists from different backgrounds unites the social and religious minorities, with more or less success, into a single, stable social community of Vojvodina.

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## THE 7 STEEL HALLS OF THE BRUSSELS' THURN AND TAXIS FORMER FREIGHT STATION

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**Abstract:** *The freight railway station of Thurn and Taxis in Brussels was built from 1901 to 1907 during the period of large activity in railway and fluvial transport. Its style is a perfect example of the Brussels' art nouveau, a mix of the organic approach inspired by natural elements and the geometric viewpoint. Many of the original decorations of the steel halls façades have been removed, since deterioration due to lack of maintenance made them rather dangerous to the public. Since 1994 the station is no longer in use by railways and an impressive number of ideas has been proposed to find a new use of the station buildings. Finally, this year the premises have been transformed to a combination of housing, offices and a park. The structures themselves show some particularities as an eccentricity of the vertical loads on the columns and the uncertainty of the cohesion of the 7 roof trusses.*

**Key words:** *Historic railway station, art nouveau in industrial buildings, three-hinged arched frames, load eccentricity of columns.*

### 1. Introduction – historic locations

The former freight station of Thurn and Taxis to the North of Brussels is named after the famous German nobles with the same name, who resided during the 16<sup>th</sup> and 17<sup>th</sup> century in Brussels. Already at the end of the 13<sup>th</sup> century they started a regular mail service, first for the Venetian Republic and later on for the emperors and other princes and gradually for more organizations. In 1490 the emperor Maximilian granted them the exclusive rights for mail services. The family became extremely wealthy and acquired a lot of land around Brussels. The ground parcel on which the freight station was built, was purchased from this family. However the parcel was on swamp soil.

From 1835 railway lines and services grew steadily on the European continent [1]. Simultaneously, the maritime canal near to the location, experienced a large increase of shipping traffic. In 1895 the company 'SA du canal et du port de Bruxelles' was created to take over the obligations of the city of Brussels and to realize the necessary installations for the transshipment of goods and the construction of quays. The need to connect shipping with rail transport became obvious and the city was looking for a suitable terrain to create a new freight station and found the premises of Thurn and Taxis.

The design of this station was ordered to the head of the buildings division of Belgian State Railways, Frédéric Bruneel [2], who associated immediately with Jules Zone, chief engineer of the SA du canal. The architects Constant Bosmans and Henri

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Vandeveld were asked to contribute, although the collaboration of the latter seems uncertain. Work began in 1900 and it took till 1907 to finish the entire project of 40000 m<sup>2</sup> of steel and glass halls. Today, people passing by identify the former freight station to the 'entrepôt' building, which is closest to the main road along the quay. However, this building was merely a warehouse for goods, the main activity taking place in the station located in the perpendicular road of the Rue Picard. The latter was designed for rational operation, the railways being the main way of distribution of freight. The plan view of the whole site is seen in Fig. 1. Clearly, the 7 halls are the largest structure within this facility.



**Fig. 1.** View from above of former Thurn and Taxis freight station

## **2. The freight station**

The total surface of the former Thurn and Taxis station is 300 000 m<sup>2</sup> and included 26 km of railway tracks, some of this also running along the port quays. The cargo was unloaded from moored ships and loaded onto the train wagons to be further distributed in the station. Further transport was either by rail or by road. Loading of train cars was possible from 12 tracks inside the cargo hall, 26 tracks in the warehouse and 42 outside any building [3]. About 1500 people worked 24 hours a day in this station during the period of maximum exploitation. Every day 26 trains were entering the station and 700 train cars were unloaded. Especially parcels and goods packed in boxes or on pallets were traded. Southern fruit was received and distributed from October to January. There were also several gantry cranes and weighbridges in the station.

As in the late 20<sup>th</sup> century transport by road became more competitive and flexible due to the large system of roads, the station declined. In addition customs were no longer required and Thurn and Taxis gradually lost its importance. Moreover competition in the parcel market became very tough and rail transport was no longer able to keep this part. The station was finally closed in 1994 and was sometimes used for events. Finally it was sold and the city of Brussels had to decide on a future use [4].

However, the main buildings and especially the brickwork 'entrepot' and the steel halls where trains were loaded and filled again, became a protected monument in 1987. Hence no demolition plans were acceptable. Various proposals were introduced to reuse the buildings. From 1993 till 2001 the grounds were used for a music city. The competition with other event halls in Brussels was too harsh and the activity was abandoned. In addition ICOMOS protested against this activity. Another idea was to build a permanent



museum for transport and railways, or a city of fine arts and communication. Alternatives as a science and knowledge centre or a museum for immigration were proposed. Most proposals consisted of a mixed furnishing as offices, housing and a park. The latter idea persisted, although it took till 2016 before a final destination was found as housing, including some 45000 m<sup>2</sup> of living space and 3000 m<sup>2</sup> of inside gardens [5]. The houses are mainly of wood and energy neutral. The open space outside has become a local park.

The connection to the railway network required passing below the 'Boulevard du Jubilé' and Bruneel designed a remarkable steel bridge in the typical Brussels' version of 'art nouveau'. This bridge is still in use and can be seen in its original state in Fig. 2 left and present state Fig. 2 right.



**Fig. 2.** Boulevard du Jubilé bridge (left : original, right today)

The Brussels' version of 'art nouveau' (Jugendstil, Wiener Sezession) is a compromise of the organic approach as promoted by Victor Horta, and inspired by nature [6] and the more geometric viewpoint as promoted by the Austrian architect Joseph Hoffmann [7]. Thus, the whiplash and spaghetti shape not always appears and especially the asymmetrical character of decorations may be missing. From 1893 till 1914 around 500 houses have been built in Brussels, in this particular style.

### 3. The station building

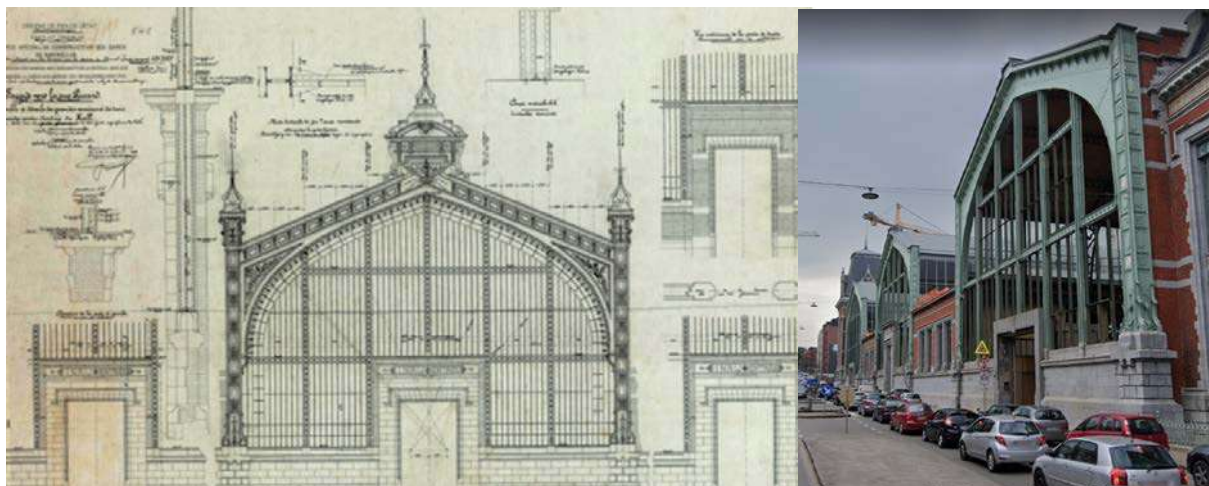


**Fig. 3.** Bruneel's drawing of façade (left) present state (right)

The building and in particular the 7 halls were richly decorated. Bruneel's original drawing can be seen in Fig. 3 (left), the present status being shown to the right.



Obviously, many of the original decorations have been removed, as the central and column pinnacle, the decorated pediments, the winged wheel and the worked infill of the plated part of the triangular nodes. The columns of the main halls and those of the two small central spaces rest on a high hard stone plinth. Those of the first halls are metal columns with lighter decoration and also have a stone base. Of the central entrance gates with stone frame, which are set in large glass sections, only the central space is original. It consists of double wooden wings with handles, impost window with iron rod distribution on a wooden main frame, profiled frame under reverse archivolt, and a main frame on which the inscription "Entrance" appears on a panel. The writer must confess that he was forced to ask the removal of the pinnacles back in 1979, since parts of these elements caused accidents as they fell down on the street. During this, the original drawings by Bruneel, made with white ink on blue paper, had to be consulted often. Unfortunately these drawings of far better quality and detailing than Fig. 4 (left) seem to have disappeared.

#### 4. The steel structure of the halls

The large halls Nos. 2, 4 and 6 consist of 22 parallel frames and have a span of 26 m and maximum height of 23 m. Each frame is completely riveted and has a N-truss shape as shown in Fig. 4.



**Fig. 4.** Inside view of large hall frames (© GOB-DML)

As shown in this figure, the vertical, as well as the sloping members of these frames are ultra-light and are supported by heavy columns. The frames are connected by purlin profiles, whereas the column tops are linked by more complex N-truss girders of variable depth. In the end nodes, the frames are decorated by curved strips as the art nouveau style requires. In addition, the connection of the light vertical post to the heavy column also is decorated by a curly detail of Fig. 5.

The frames have a central hinge. The connection of the vertical posts to the columns seems rather fixed. However the lower hinges are located at the base of the heavy columns. This can be seen in the detail of Fig. 6. Hence, the frames are 3-hinge structures. In view of the rather small radius of the cylindrical hinge, the free angular rotation may very well be limited, since the compression stress must be very high. Accordingly, friction builds up easily, thus reducing the rotational capacity.



**Fig. 5.** Curved node at connection of column to frame (© GOB-DML)



**Fig. 6.** Hinge at the base of heavy columns (© GOB-DML)

However, these columns show a rather weird composition. This is found from Fig. 7 and the redrawn cross-section. The location of the upper corbels does not correspond to the centreline of the columns. Instead an additional angle profile allows connection of the upper part of the frame.

As a consequence the axial force shows an eccentricity of 0.19 m. This would easily increase the column stress by around 15 MPa. Probably this increase has no particular consequence and the eccentricity was acceptable at the time of the design. However it is an unusual detail and disturbs the styled ornament at the column base as can be seen in Fig. 7 (left).

The roof structure of the smaller halls is simpler and consists of a continuous N-truss, without any hinge and rigidly connected to either the heavy columns or the brickwork walls as can be seen in Fig. 8. The span of these trusses equals 16 m and the depth is higher in the central part than at the ends, since the central hinge is missing. However, as the truss is connected to the larger frames, an interaction of forces occurs. In particular the in-plane horizontal displacements, caused by wind on the smaller trusses, will be transferred to the large frames and introduce higher member forces, particularly in in frame nodes

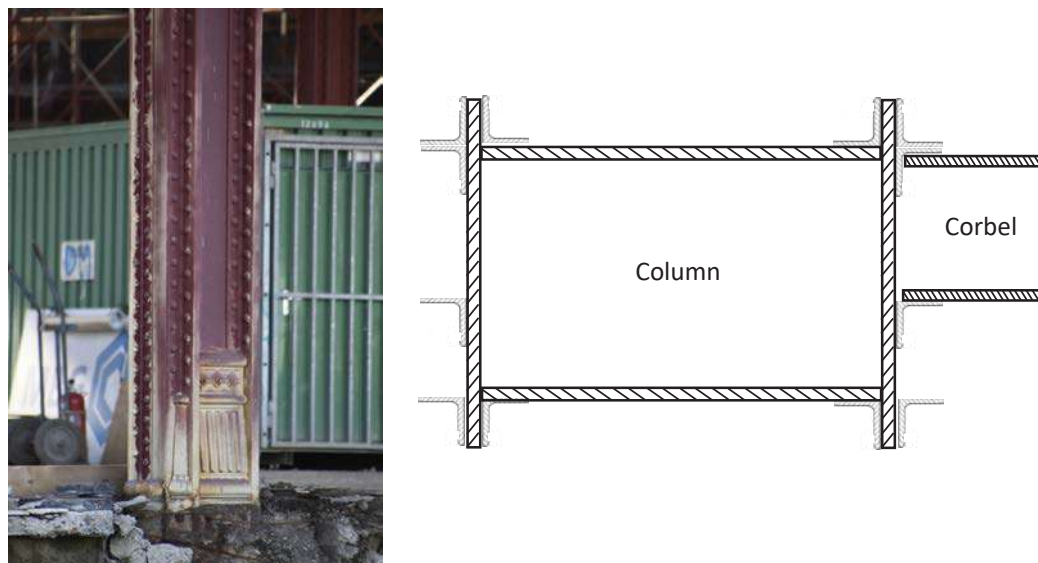


**Fig. 7.**Column base (left © GOB-DML) and cross-section

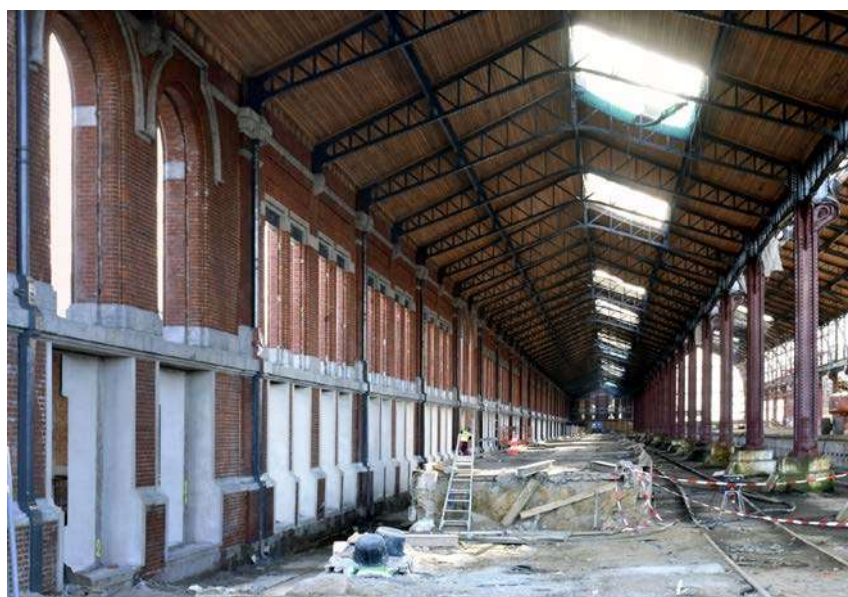


**Fig. 8.**Smaller hall 7 (© GOB-DML)

This may well suggest that the 7 hall roof structures were designed as independent trusses and not as a whole and interacting system, in opposition to for instance the Ghent Floralia hall [8]. This idea of separated trusses certainly does comply with the concept of 7 independent structures, having a total width of 142 m. The global outside view of the former freight station, immediately after building of Fig. 9, certainly suggests the structurally minor importance of the smaller halls.

### **Acknowledgement**

Gratitude goes to the Regional Public Service of Brussels, on whose website <http://www.irismonument.be/> are pictures, which can be used, and are of better quality than one would be able to produce oneself. The quality certainly is much better than of the photographs the writer took more than 40 years ago.



### Conclusion

The former freight station of Thurn and Taxis in Brussels consists of various buildings and open spaces. From the structural point of view, the larger halls are covered by steel riveted 3-hinged arched frames, connected to heavy columns, hinged at the base. The effectiveness of these hinges may be questioned, since the diameter is limited to 130 mm and the lubrication is not assured. In addition, the upper part of the large frames shows an eccentricity of 22 % with respect to the column axis. This introduces additional bending in the columns, probably without further consequence.

The 7 halls of the former freight station have a long history for finding a new use, which has resulted today in a combination of energy-neutral housing and offices, the outside premises having become a local park.



**Fig. 9.**Outside view immediately after its construction (1910)

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## ABOUT THE OLD RIVETED BRIDGE IN BULGARIA IN THE LIGHT OF THE METAL HERITAGE BRIDGES IN THE WORLD

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**Abstract:** *In this paper the authors describe the construction of some riveted metal bridges, in Bulgaria built in the end of XIX and in the beginning of XX century. The importance of this bridges for our engineering society is viewed in the context of world historic-cultural heritage in the field of old riveted metal bridges.*

**Key words:** riveted metal bridges, context of world historic-cultural heritage

### 1. Introduction

Wooden was probably the first material used by humans used to construct a bridges. Although metal bridges have existed for centuries, the real Iron Age began with the Industrial Revolution. In 1779 Abraham Darby III built the first iron bridge spanning the Severn, but the material was not strong enough.



Fig.1.1 The Iron Bridge in Coalbrookdale



Fig. 2.1.1 Victoria Bridge, Montreal

### 2. Development of metal bridges in the world

#### 2.1. Bridge Engineering in Canada

##### \* Canada's First IRON Bridges

One of the most notable bridge from the mid-1800s is Montreal's Victoria Bridge (see: **Fig. 2.1.1**)[1.Chapter 1,Joost Meyboom]. The Victoria Bridge was the first crossing of the St. Laurence River and was completed in 1859. It was constructed to provide a fixed railway link between the Island of Montreal and Montreal's South Shore. When it was opened it was the longest

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bridge in the world, with a length of 3 kilometres. The original superstructure of the Victoria Bridge was a closed steel tube, similar to that used for the Britannia Bridge. This tube was replaced in the 1890s with open trusses to relieve the noise and smoke created in the tube by locomotives.

### 3. Introduction to Bridges in USA

The evolution of bridges in the United States is probably not much different from anywhere else in the world[1.Chapter2, M.M. Lwin]. Civilizations have borrowed their bridging ideas from each other for centuries. The earliest bridges in America took the form of logs placed over an obstacle. In roads through swampy or low-lying areas, logs were placed side-by-side, perpendicular to the direction of travel. This type of construction, known as corduroy construction, consumed a substantial amount of timber and resulted in an uneven roadway surface.

#### 3.1. Historical Development

Beginning in the early 1800s, truss bridges were introduced as a means capable of spanning longer distances. Squire Whipple and Herman Haupt, two American railroad bridge engineers, are credited with being the first to calculate methods for determining stresses in truss members and were thereby able to determine their appropriate sizes. The knowledge to engineer trusses made their construction popular. The concepts of rational principles are equally applicable to both timber and metal trusses. Many other engineers quickly embraced the concepts and patented various truss diagonal configurations for their own use. Many of their names are familiar today: Pratt, Parker, Howe, Burr, Fink, and Warren.

##### \* Steel Arch Bridges

Steel truss bridges retained their popularity, but the need for longer spans necessitated the use of continuous span trusses. Continuous span truss bridges required a shallower truss depth than their simple span equivalents, and, therefore, were more economical. The Huey P. Long Bridge just west of New Orleans, Louisiana was opened in 1935. Fig. 3.1 illustrates the proposed structure after completion of widening of this bridge, which is currently underway.



Fig. 3.1 Proposed Huey P. Long Bridge



Fig. 3.2 Blennerhassett Island Bridge

##### \*\* Steel Arch Bridges-Blennerhassett Island Bridge

The Blennerhassett Island Bridge (Fig. 3.2), which spans the Ohio River between West Virginia and Ohio, was identified as the critical remaining “missing link” of the final segment of Appalachian Highway Corridor D. The structure includes an (267.6 m), tied arch main span with a rise of (53.5 m) and approach spans that consist of variably spaced steel plate girders with spans up to (122.2 m) in length. To minimize the size and weight of the approach span superstructure, the design utilizes hybrid girders and high-strength steel. The bridge’s tied arch ranks as the longest networked tied arch structure in the United States and is among the longest in the world.

### 4. Introduction to Bridges in Argentina

#### 4.1. Argentina iron Bridges

##### \*Truss Bridges -Dulce River Bridge

The first studies for the construction of this railroad and roadway bridge began in 1880, but it was not until 1924 that works began[1.Chapter3, T.A. del Carril]. It was considered an important and very costly construction work for the country during that period. The structure was designed for dual purposes: railroad and roadway traffic. The bridge (Fig. 4.1) is 840 m (12 x 70 m) total length. Steel trusses (6400 tons) were built by were built by Gutehoffnuugshüte (Ruhr, Germany). In October 1927, it was opened to the public. Each 70 m span has two truss beams built 11m apart. The original design comprised 6.7m width for the roadway and 3 m width for the tracks and

sidewalks. In 1950 a bicycle lane of 1.5m was added. *There are two curious legends about the construction of this bridge.* One is that all the steel needed for the bridge was given by Germany to Argentina free of charge as a compensation for the sinking of three Argentinean cargo vessels, Toro, Monte Protegido, and Oriana, by German submarines in the Atlantic Ocean during World War I in 1917. The other myth tells us that there is a solid gold rivet placed somewhere among the thousands of the structure. Yet nowadays, one can seldom see people searching for said rivet.



**Fig.4.1** Dulce River Bridge between the cities of La Banda and Santiago del Estero, built in 1927.

### **5. Introduction to Bridges in Brazil**

Brazil is known to have some of the most notable structural engineers in the world, designing memorable bridges despite the frequent difficulties encountered[**1.Chapter 4, A. C. de Vasconcelos, G. L. Marchesini, J. Timerman**]. Examples include the set of railway bridges designed for the railway Mayrink–Santos in 1940s; the first prestressed bridge in South America (Galeao Bridge in the harbor of Rio de Janeiro), built in 1949; and President Costa e Silva Bridge (also called Rio–Niterói Bridge), the longest span in the world with a continuous straight beam.

#### **5.1 Historical Development**

##### **\*President Costa e Silva Bridge**

The President Costa e Silva Bridge, (**Fig. 5.1**), was inaugurated on March 4, 1974 and is a national engineering landmark because of its size, signature design, and the creativity of the processes for executing the work. It is the seventh longest steel girder bridge in the world, but experts insist that it is still the biggest, in terms of spatial volume, because of its gigantic pillars and the caissons sunk into rock, over 60 m below the water level. Above all, it highlights the setting of Guanabara Bay.



**Fig. 5.1** Side view of the Rio-Niterói Bridge.

### **6. Introduction to Bridges in Bosnia and Herzegovina**

Bosnia and Herzegovina is a country in southeast Europe, on the Balkan Peninsula [**1.Chapter5, B. Koboevic, B. Karalic-Hromic, D. Zenunovic**]. Bordered by Croatia to the north, west, and south, Serbia to the east, and Montenegro to the southeast, Bosnia and Herzegovina is almost landlocked, except for 26 km of Adriatic Sea coastline, centered on the town of Neum. The country is mostly mountainous, encompassing the central Dinaric Alps. The development of building bridges in Bosnia and Herzegovina began with the arrival of the Turks.



### 6.1. Historical Development

Today there are about 3000 bridges in Bosnia and Herzegovina, of which 1000 are on the federal roads and the rest are regional and local roads. The bridges are of different ages, shapes, and structural systems, mostly built of concrete, stone, and steel. About 70% of the bridges were built in the period from 1955 to 1985, and 90% of bridges were Constructed mainly of reinforced concrete and prestressed reinforced concrete. Other bridges are mostly composite steel–concrete. Thoses bridges destroyed during the war from 1991 to 1995 have been mostly reconstructed.

#### \*Road Bridge over the Sava River in Bosanska Gradiška

It was built in late 1939 or the beginning of 1940. The main bridge structure over the Sava River is continuous steel truss with a constant height of 6.5m, with horizontal upper and lower chords, diagonals and bracing against the wind (**Fig. 6.1a**). It was completely destroyed in World War II. The new bridge was built in 1956 with a total length of 283.85m (9.8m + 93.795 m + 72.9m + 72.9 + 9.8 m) (**Fig. 6.1b, 6.1c**). The steel truss of the first span, about 44m in length, was destroyed in the 1992 war. Reconstruction of destroyed parts of the bridge was done in the eight phases, with one temporary pier. Reconstruction started on May 1, 1999 and finished on May 4, 2000.

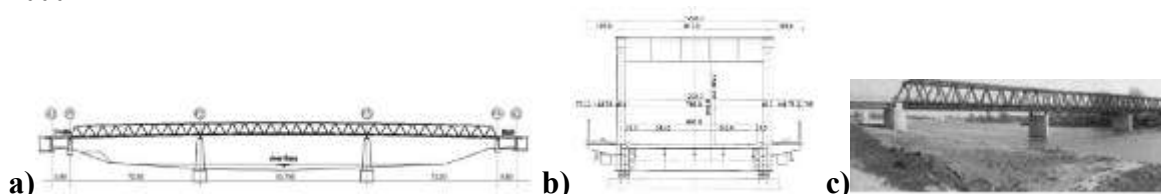


**Fig. 6.1** Bridge over river Sava in Bosanska Gradiška. (a) Elevation, (b) typical section, (c) overview

#### \*\*Gazelle Bridge over the Neretva River in Jablanica

The railway Sarajevo-Ploce was opened to traffic in 1965. The total track length is 195 km, with 108 tunnels, 64 concrete and reinforced concrete bridges, and 10 steel bridges. Among these bridges, the Gazelle Bridge over Neretva River in Jablanica in particular stands out (**Fig. 6.2a**), with a total length of 300m consisting of six structural units (**Fig. 6.2b**). The approach structure in the direction of Sarajevo has four simply supported steel composite box girder spans of 36m. The steel girders' height is 3.32m and the reinforced concrete deck width is 5m (**Fig. 6.2c**). The main span of the bridge is 121m and crosses the river Neretva, and it is supported with angled piers with spans of 100m, which are jointly supported on a special foundation. The superstructure of the main span is a steel box with a trapezoidal cross section with a height of 3.6m and width of 2m to 3.57m (**Fig. 6.2c**). The angled piers are also steel box sections, which are linearly tapered down, while the top has a circular haunch. The height of the piers is 40m, and the height of the structures above the water of the Neretva River is about 55 m. In this part of the bridge there is no road bed, and the track was laid directly on the sleepers.

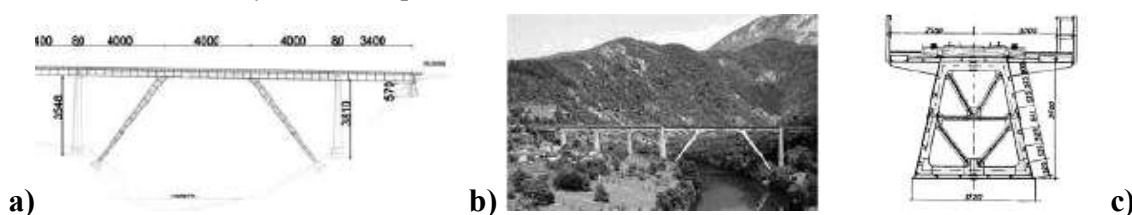


**Fig. 6.2** Gazelle Bridge over river Neretva. (a) Elevation, (b) typical section, (c) overview

## 7. Introduction to Bridges in Bulgaria

The oldest stone bridges in Bulgaria date from the age of the Ottoman Empire (1386–1876). The modern development of the road and railroad infrastructure in Bulgaria began in 1866 with the official opening of the 223 km long Rouse-Varna railway[1.Chapter 6, D. Partov, D. Dinev]. During railway construction in Bulgaria after 1866, bridges were mostly steel trusses with riveted joints and spans between 20 m and 50 m.

### 7.1. Historical Development

The bridge structures were imported from Belgium, Germany, France, Italia, Austria and Hungary. According to the Road Infrastructure State Agency the total number of the road bridges in Bulgaria

is approximately 5000. The Railroad Infrastructure State Company maintains about 623 concrete bridges and 327 steel bridges. Three of them will describe in the next text of the article.

#### **\*Lom Bridge**

The Lom Bridge is a steel trough-truss structure (**Fig. 7.1**) over the Lom River near the Danube river side town of Lom. The bridge was designed and built in Italy and was put into operation in town Lom, as a road bridge. The bridge is opened to traffic in 1890. Today the bridge is still in use with a limited load carrying capacity of 200 kN vehicles. The bridge is a five-span structure with a total length of 117.5m. Each of the spans consists of two parallel Pratt-type trusses with a distance between them of 5.7m. The trusses have 10 panels with a total length of 23.5m and height of 2.2m. The bridge deck is 8.7m wide. All fabricated cross sections are fastened by a riveting. The floor beams are placed at the lower joints of the trusses. The bridge deck has five stringers, placed at a distance of 1.14m. The concrete deck slab is 120 mm thick. The bridge superstructure has a lower lateral bracing made of hot-rolled angle sections.

#### **\*\*Stambolov Bridge**

The Stambolov Bridge is a steel deck-truss arch structure over the Yantra River at Veliko Tarnovo (**Fig. 7.2**). The bridge is named after the eminent Bulgarian prime minister Stefan Stambolov, who gave the idea to build a new modern steel bridge in the city. The bridge was designed and built by Waagner- Biro. Agency and it was proclaimed a monument of culture. The bridge opened to traffic in 1892. The structure consists of two parallel Pratt-type truss arches. The truss arches are 8.6m high at the abutments and 1.2m at the middle. They have 26 panels with a total length of 81.6m. The bridge is supported on the abutments by steel hinged bearings. The abutment's foundations are bedded on solid rocks. The bridge structure survived one of strongest earthquakes in Bulgaria in 1913, with a magnitude of 7.0 on the Richter scale.



Fig. 7.1 Lom Bridge.



Fig. 7.2 Stambolov Bridge.

#### **\*\*\*Lovech Bridge**

The iron bridge over the Osam River in the town of Lovech was built in Belgium, at the beginning of the XX century, and was put into operation in 1907, as a road bridge. They consist of three separate double steel truss girders of the simple beam statically system. The main girders of the 20,00+24,00+20,00m., span is with a parallel upper and lower chord. Each of them perform an bi-construction of two main steel trusses, connected between each other at the nodes of the bottom chords with cross horizontal situated web beams and horizontal X-type bracing, situated under the horizontal beams. The appearance of the bridge is shown in **Fig. 7.3a,b, c**. A lower wind bracing connected the two lower chords of the main girders. The height of the every truss girder is 3220mm. The distance between verticals for the middle truss, is equal: 3035mm and for the another two bridges parts in the left and right equal: 2480 mm. The distance of the main girder axes is 4800mm. All elements of the trusses are made of angles and flats connected by rivets.



Fig.7.3a,b,c View of a road pedestrian steel bridge on Osam stream

## 8. Introduction to Bridges in Croatia

The Republic of Croatia is a medium-sized Central European and Mediterranean, that is, Pannonian– Adriatic, country. It has a surface area of 56,500 km<sup>2</sup>, with a population of 4.7 million. Considering the large diversity of obstacles, Croatia has developed a school of bridge design and construction whose accomplishments have worldwide significance.

### 8.1. Historical Development

The steel arch bridge over the Maslenica Strait (**Fig.8.1**) is a part of the state road connecting the Croatian north and south [1.Chapter 7, J. Radić, G. Puž]. The original bridge from 1955 was demolished in 1991 during the war, only to be reconstructed in 2002 in a shape very similar to the original but much stronger. The Maslenica Strait was crossed by a 155m parabolic double-hinged steel arch span, made of two box-type structures connected with a bracing. The total bridge length was 315m, with a height of grade line 55m above the mean sea surface. The carriageway width on the bridge is 7.1m, with walkways on both sides 1.5m wide each, making it 1m wider than the original bridge. The superstructure consists of continuous steel–concrete composite girders over 17 spans  $17.5\text{m}+3\times 19.7\text{m}+7\times 17.5\text{m}+5\times 19.7+17.5\text{m}$ . It is a grillage structure consisting of steel longitudinal and transverse girders composite with the carriageway deck made of reinforced concrete. The longitudinal supports of the steel superstructure are 1.2m high welded sheet metal supports, rigidly connected with crossbeams. The piers are steel, quadratic-like cross sections. Portal piers are connected by a beam at the top with a lattice rhomb filling, whose shape corresponds to the fill connecting the arches. The main load-carrying structure consists of two hinged arched girders, span 155m, arch rise 41.5m. The arch axis is shaped as a second-degree parabola running through three determined points. Support hinges are hidden, so visually it seems to be a fixed arch. The arches are interconnected by a rhomboid filling of counterbalances without crossbeams. In 1955, when the original bridge was constructed, it was an example of the trends in bridge design at the time.



**Fig. 8.1** Reconstruction of the Maslenica Bridge for the state road



**Fig. 9.1** Stadlec Bridge across the Luznice River

## 9. Introduction to Bridges in Czech Republic

The Czech Republic's central European landscape is dominated by the Bohemian Massif, which rises to heights of 900 m above sea level. The principal rivers are the Elbe, the Vltava, the Morava, and the Odra. The Czech Republic is a small country that occupies 78,865 km<sup>2</sup> with 10.5 million inhabitants. The Czech Republic's central position within Europe has always been a natural crossroads for continental trade routes, which later became tracks, roads, and railways, and in recent decades have advanced to freeways and international railway corridors.

### 9.1 Historical Development

The roots of Czech contemporary engineering and bridge building date back to the beginning of the nineteenth century, when the Czech Corporate Polytechnic Institute was established in Prague [1.Chapter 8, J. Straský]. At this time the historical lands of the Czech Republic, consisting of Bohemia, Moravia, and Silesia, began to become similar to the prominent countries of Europe in regard to industry, technology, and education, which was clearly manifested in the advancement in structures and technologies of transportation, including bridge structures.



### \* Chain Bridges

The first suspended chain road bridge in continental Europe was erected in 1824 in the city of Straznice across one of the branches of the Morava River. Other chain bridge constructions soon followed—near Zatec, in Podebrady, Locket, Strakonice, and across the Vltava River in Prague. The only remaining bridge structure of this type in the Czech Republic still used as a road bridge is the bridge across the Luznice River in Stadlec, which bridged the Vltava River in Podolsko since 1848. In 1960, the bridge was dismantled and transferred to a new location as a technical monument of our bridge engineering, and has been registered as a national cultural heritage landmark since 1974 (Fig. 9.1).

### \*\*Steel Bridges

Steel bridge constructions in the Czech regions at the end of the XIX and beginning of the XX centuries was linked mainly to the construction of new railroads and roads in the Austria–Hungarian Empire, many of which are used to this day. Welding technology brought a breakthrough in steel bridge construction, and was used in CZ country— among the first in the world—in 1930 during the construction of a truss bridge in the Škoda Works in Plzen, and a year later on an arch bridge across Radbuza in Doudleby. The road bridge across the Radbus River in Pilsen, built in 1932, **was the world's first welded arched bridge**. The individual mounting parts of the bridge were welded in the Škoda Bridge Factory, where prof. Fr.Faltus was a Head of "Welding Structures". The main solid arches are welded beams of cross-section "I" with a span of 40.6 m and a high of 10.4 m, mounted on supports from the cast joint. The arched beams, with welded legs at a distance of 4.35 m, carry a bridge deck, formed by welded crossbars and longitudinal beams of cross-section I and reinforced concrete slab. Walkways were conveyed directly to the bridge cross members. The total width of the bridge was 9m, of which 6m was a roadway (Fig.9.2).

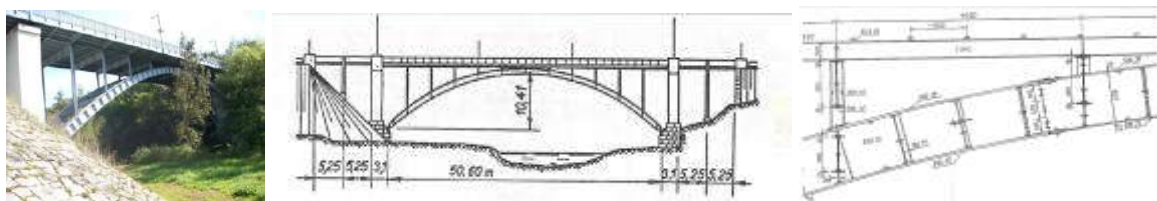


Fig. 9.2 Arched bridge over the river Radbuza (Tyršůvmost)

## 10. Metal Bridges in Denmark

Denmark consists of the peninsula Jutland, extending north from Germany, and 78 inhabited islands (plus more than 200 small uninhabited islands). The capital of the country, Copenhagen, is situated on the largest island-Zealand. The Danish Straits, Lillebaelt (Little Belt), Storebaelt (Great Belt), and Øresund, form the only natural water ways between the Baltic Sea and the North Sea. In area Denmark is the smallest of the Nordic countries (Denmark, Finland, Iceland, Norway, and Sweden), but it has the second-largest population, although it is only 5.5 million.

### 10.1. Early Bridges in Denmark

In prehistoric times minor bridges **were probably built of wood**, but they all disappeared long ago and nothing is left to indicate the location and form of these bridges[1.Chapter 9, N. Jørgen Gimsing]. In the Middle Ages wooden bridges were built in large numbers across brooks of moderate widths, but none of these bridges have survived to our time. **Up until around 1770 the majority of road bridges were still built of wood**, but after that time natural stone was used to build more reliable and durable bridges, forming a part of a national highway network.

### 10.2. Start of Major Bridge Construction

More permanent bridges on fixed piers and with steel superstructures (Fig. 10.1) appeared sporadically in the second half of the nineteenth century as part of the new railway network, but it was not until the 1930s that a real effort was made to substitute the many ferry routes by bridges. From 1930 to 1940, 11 bridges were built across straits with widths from a few hundred meters to



more than 3 km. Five of the bridges were for both road and rail traffic, five for road traffic only, and one for railway only. The most notable bridges were the high-level bridges: the Lillebelt Bridge from 1935 (**Fig. 10.2**) and the Storstrøm Bridge from 1937 (**Fig. 10.3**). With its total length of 3.2 km, the Storstrøm Bridge was at completion the longest bridge in Europe, and it remained the longest road and rail bridge for a period of 60 years until it was surpassed by the West Bridge of the Storebaelt Link in Denmark. The ambitious bridge construction program of the 1930s was abruptly interrupted at the outbreak of World War II in 1939. However, two more bridges that were already under construction were opened to traffic in 1942 and 1943, respectively. The bridges from the 1930s to early 1940s were generally built with steel superstructures in the form of riveted plate girders, trusses, or arches. In a few bridges the approach spans were made as concrete arches, for example in the Queen Alexandrine Bridge, shown in **Fig. 10.4**. After World War II, due to lack of resources during the post war period, the bridge building program was not revived until 1952, when the small Munkholm Bridge (**Fig. 10.5**) was opened to traffic..



**Fig. 10.1** Railway bridge from 1862 across the Gudena



**Fig. 10.2** The Lillebelt Bridge from 1935.



**Fig. 10.3** The Storstrøm Bridge from 1937



**Fig. 10.4** The Queen Alexandrine Bridge.



**Fig. 10.5** The Munkholm Bridge from 1952.

## 11. Introduction to Bridges in Finland

Finland began to build roads during the late Middle Ages[1. **Chapter 10, Esko Järvenpää**]. The first important bridges were built over the rivers and straits of Finnish lakes when the main road from the Russian border to west cost was built. The road was called the King's Road, **and wood was the main material** for the bridges because of its availability and low costs. There were also skilled carpenters just for bridge construction. The problem with wooden bridges was the short life span of the materials and work amount needed often for repair of the damages. One of the most interesting wooden bridges was built in Oulu in **1867 (Fig.11.1)**. The spans of the bridge were 38,10m + 45,70m + 38,10m. The bridges served until the year 1950.

### 11.1. Old Steel Bridges



**Fig.11.1** Historic wooden bridge of the Oulu River



**Fig. 11.2** Tornio River Bridge, built in 1939.

The first railway in Finland was built from Helsinki to Hämeenlinna and opened for traffic in the year 1862. The first iron bridge was built at that time. The knowledge of how to build steel bridges came from England. The first major steel bridge was built over the river Kymi in 1870. The bridge exists today as a museum bridge. Road bridges were also built using steel. Truss bridges have been a solution for railway bridges and bigger road bridges over the stronger rivers in the country. One example is the Tornio River Bridge (**Fig. 11.2**), which was completed in 1939. The bridge was the biggest road bridge at the time. The main span has a length of 108 m. The bridge has been renovated. The concrete deck has been changed to an orthotropic deck and the pedestrian lanes have been widened outside the truss girders.

## 12. Introduction to Bridges in France

Stones, wood, and ropes were probably the first elements used to build bridges. In historical times, the most exceptional bridges were built by Romans. The Pont du Gard in southern France stands as a lasting example of their engineering talent. This majestic three-tiered aqueduct, built in 18 BC, is approximately 49 m tall by 273 m in length. The world's great cities—Paris, London, and Florence, among others—naturally flourished along rivers that were essential to the transport of merchandise. Waterways brought prosperity, while bridges ensured a sense of community. Of the 38 bridges spanning the Seine River in Paris, the Pont Neuf, despite its name, is not only one of the oldest bridges in the city—it is as much an emblem of the French capital as the Eiffel Tower or Notre-Dame Cathedral. Connecting the right and left banks of the city with its historic center, the “Ile de la Cite,” the masonry structure is formed by two successive bridges, one with five arches, the other seven.

### 12.1. Historical Development

The design and construction of metal bridges was developed quickly with the use of steel of increasing quality. Gustave Eiffel's projects are striking examples of the innovative use of steel (**Fig.12.1**)[1.Chapter 11, Jean-Armand Calgaro]. Before the advent of, first cast iron(the iron Bridge near Coalbrookdale on the river Severn opened in 1781), then steel (one of the first major steel bridge, the King Albert Bridge over the Elba, was opened in 1893), and finally reinforced concrete, stone and timber were the only available materials for bridge building. While we have many examples of very old stone bridges, this is for obvious reasons not the case for steel bridges.



**Fig. 12.1** The Garabit viaduct, designed by G.Eiffel

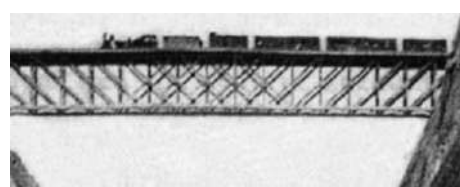


**Fig. 13.1** Bridge of Corinth Canal(1893)

## 13. Introduction to Bridges in Greece Historical Development

### 13.1. Modern Greek Period (1828 A.D.–2010 A.D.)

Following the progressive liberation of Greece and its independence from the Ottoman Empire (1821–1920) an increase occurred in the construction of steel railway bridges, which were exceptionally advanced for their period. Some of them exist today and are in operation despite being more than 100 years old and the relevant fatigue of the material as well as increases in the weight and speed of the trains[1.Chapter 12,Stamatios Stathopoulos]. Seventy-five bridges with spans longer than 15 m exist now in the entire Greek state. The majority of these bridges are truss bridges and a smaller portion are beam bridges; on the whole, the bridges were riveted. Their abutments were stone, while their piers were stone or steel. Those bridges are usually simply supported with spans up to 80m or continuous with spans of 50–60m. Sporadically, double overhanging beams with spans up to 120m, three-hinged arches with spans up to 80m, and tied arches were constructed. Certain bridges had a total length over 300m, spans up to 120m, and piers of heights up to 60m. Included among the largest bridges are the bridges of Corinth Canal (**Fig. 13.1**), span 81.4m; the Achladocampas Bridge, total length of 253m (36.2m + 54.2m + 72.2 m+ 54.2m + 36.2m), continuous, with pier height over 40m (**Fig. 13.2**); the railway bridge on the

Alpheios River, total length 6×51.6m (**Fig. 13.3**); the Gorgopotamos Bridge (**Fig. 13.4**), total length 7×29.12m; the Asopos Bridge (**Fig. 13.5**) total length 180 m (5×25.3m + 1×80m-(three-hinged arch)); and the Bralos Bridge (**Fig. 13.6 and 13.7**) total length ~325 m (16m + 42m + 52.5m + 60m + 120m + 33m). After World War II and its severe destructions, the construction of modern bridges began in Greece. Reinforced and prestressed concrete was widely used, while steel, due to its cost and lack of domestic industry, passed unnoticed, and appeared again after 2000 and the incorporation of the country into the European Union.



**Fig. 13.2** Achladocampos e **Fig. 13.3** Alpheios Bridge, Bridge

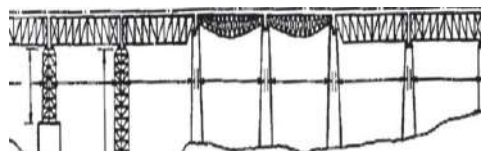


**Fig. 13.4** Gorgopotamos Bridge,



**Fig. 13.5** Asopos Bridge, total length 180, **Fig. 13.6** Railway Bralos Bridge (1905), max. 120m,

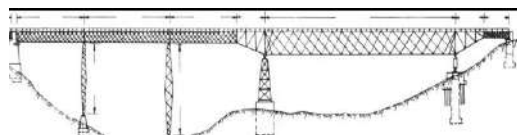


**Fig. 13.7** Steel pier of height 50m

#### 14. Bridge Engineering in Nord Macedonia

The Republic of Nord Macedonia is a small country in the central part of the Balkan Peninsula. To the north, south, east, and west, it borders on Serbia and Kosovo, Greece, Bulgaria, and Albania, respectively. Its territory covers an area of about 25,713 km<sup>2</sup> and it has about 2 million inhabitants. The biggest river is Vardar, with a total length of 420 km, which rises in West Macedonia, near Gostivar, passes the border with Greece in the vicinity of Gevgelija, and empties into the Aegean Sea near Thessaloniki.

##### 14.1. Introduction

Most bridges in this country are reinforced concrete structures with a plate or beam structural system [1. Chapter 13, T. Nikolovski, D. Ivanov]. The percentage of steel and stone bridges is less than 4%. Since Macedonia is a small country and does not have large rivers, deep and wide valleys, or many natural obstacles, the average length of bridges is only about 36 m, while that of bridges on the national roads is around 47 m. Bridges with a length of over 200 m and spans larger than 50 m are scarce. All the bridges of the current network of national and regional roads were built after 1950 (98%), which reflects the conditions prevailing until and immediately after the Second World War. The most intensive period of construction was between 1960 and 1980, when over 62% of the bridges on national roads and 75% of the bridges on regional roads were built. They are now about 40 years old. At first, bridges were constructed of wood. Then, steel bridges started to be built, ordered from Dortmund, Westphalia. Along the Thessaloniki–Skopje railway line, over 50 bridges were constructed with an opening larger than 10 m, of which 6 were bridges over Vardar River with a total length exceeding 1000 m. The completed bridges, with a span of up to 10 m, were put in place as delivered.

##### \*Steel Girder Bridges Melnicki Bridge Near Debar

The Melnicki Bridge over the Spilje artificial lake is located at the exit of the wonderful Radika River canyon in Western Macedonia, on the regional road Mavrovo–Debar (**Fig. 14.1**). The bridge is a composite structure with steel main girders and a prestressed reinforced concrete deck. The superstructure of the bridge consists of two separate parts: in a straight alignment, with spans of 68 m + 2 × 83 m + 68 m, and in a curvature with a radius of 70 m, with spans of 2 × 35 m. The curved part of the superstructure is a closed steel box cross section with a cast-in-place concrete deck. The height of the steel girders is 2.5 m and the total height of the composite cross section is 2.8 m. The width of the carriageway is 6 m, while the total width of the bridge is 8 m. The bridge was built in 1970 according to the design of academician Nikola Hajdin from Belgrade.





Fig. 14.1 Melnicki Bridge near Debar.

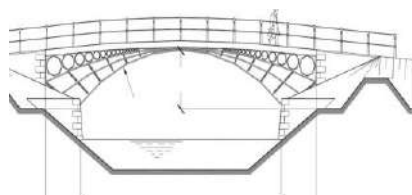


Fig. 15.1 The bridge over the Strzegomka River in Łažany .

### 15. Introduction to metal Bridges in Poland

Poland as a state was established in 966 AD [1. Chapter 14, J. Biliszcuk, J. Bien, W. Barcik, P. Hawryszkow, M. Hildebrand]. Its territory and frontiers have changed throughout 1000 years of history. Within its contemporary boundaries Poland boasts not only Polish accomplishments, but also historical structures of great value to bridge engineering erected by German engineers. The total area of Poland is 312,679 km<sup>2</sup>), and it has a population of over 38 million people. Rapid development of bridge engineering in Poland began in the mid-nineteenth century with the introduction of railway transport and modernization of the road network.

#### 15.1. Historical Iron Bridges

In Poland, iron was introduced into bridge engineering already in the 18th century. The first iron bridge in the continental Europe was built in the Silesia region. In the late 1700s, two steelworks functioned in Silesia, which significantly contributed to the development of iron bridges in this part of Europe. These steelworks produced over a dozen of the first iron bridges located in Silesia and in Germany. The structure (Fig. 15.1) was a one span bridge. The span consisted of five iron arches, each 16.85 m long and 2.885 m high, with spacing appropriate transverse elements. The 46 ton structure was cast in 1794–1795. In the fall of 1795 it was transported to its destination. The footbridge in Opatówek was built in 1824 in Józef Zajączek's palace and park complex. The structure (Fig. 15.2) is a one-span arch structure consisting of four main girders cast in iron. Each of the girders is made up of three segments joined by bolts. Originally, the deck was built up of timber planks. The span between the supports extends to 10.3 m and the total length of the superstructure is 13.8 m. The total width of the deck is 3.5 m. The main girders are adorned with ornaments cast as wholes. In 1827, a suspension bridge was built over the Mała Panew River (Rabiega and Biliszcuk 1996). The designer and builder of the structure was Karl Schottelius. The main span between pylons is 31.4 m. The bridge is 6.28 m wide. The open-work pylons form an ornate portal for the gate to the bridge (Fig. 15.3). The road surface, built up of three layers of timber planks, was placed on crossbeams formed of rolled I-beams. The era of iron bridges lasted from 1779 to the late 1830s. Technological development in metallurgy led to massive low-cost production of steel, a material of high tensile strength and of high resistance to cracking, which limited the use of cast iron.

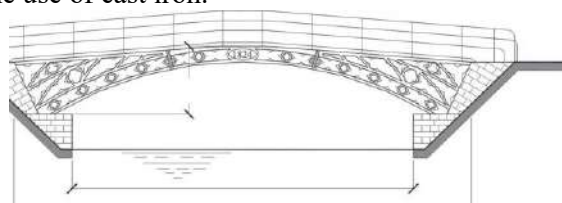


Fig. 15.2 Bridge over the park moat in Opatówek (1824), the oldest iron bridge in Poland still open for pedestrians.

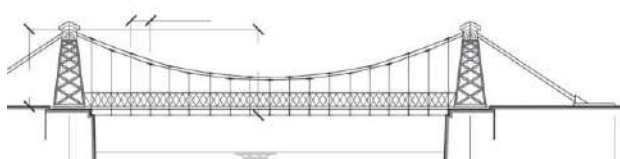


Fig. 15.3 Bridge over the Mała Panew River in Ozimek (1827).

#### \* Steel Bridges

The two oldest steel bridges are located over the Vistula River in Tczew and the Nogat River in Malbork. They were designed as tubular (box) structures modelled on the Britannia Tubular



Bridge (1846–1850) constructed by George Stephenson. As opposed to Stephenson, Lentze used a lighter truss construction, which better resists wind forces (**Fig.15.4**). The superstructure was formed of three two-span space trusses with parallel chords and a dense system of diagonal rods. The distances between the axes of the piers are  $6 \times 130.88$  m. The total length of the bridge is 785.28 m. The girders are 11.8 m high and the clearance between the truss webs of the box is 6.5 m. The construction of the girders was formed of a dense riveted truss of flat bars. The bridge over the Nogat River in Malbork had identical construction and consisted of one two span segment. The structures in Tczew and Malbork were designed for rail and road traffic. Road traffic was withheld when trains crossed the bridge. Both structures were severely damaged during World War II. In Tczew, only three out of six spans have survived (**Fig. 15.4**). The first durable bridge structure over the Vistula River in Warsaw was constructed by famous Polish engineer Stanisław Kierbedź in 1859–1864 (Jankowski 1973). It was a steel truss bridge with a total length of 475 m. The spans were 79 m long. The structure was similar to that of the bridge in Tczew. The superstructure was divided into three segments, each in the form of a continuous two-span beam. The clearance between the 9.1 m high girders was 10.5 m. The bridge was a steel riveted structure. A tram track with groove rails was laid down on the bridge. The sidewalks were placed outside the box girder. **Fig. 15.5** shows a general view of the structure.



**Fig. 15.4** Carl Lentze's bridge over the Vistula River in Tczew **Fig. 15.5** Kierbedź's bridge over the Vistula River in Warsaw .

Construction of the first welded steel road bridge in the world (**Fig. 15.6a,b,c**) was a significant achievement of Polish bridge engineering. The test results together with the engineering intuition of Bryła were the basis on which the road bridge over the Słudwia River near Łowicz, was fabricated by means of arc welding in 1928. The welded steel bridge was completed in December 1928 and the bridge on the Słudwia River was opened to traffic in August 1929. It is worth mentioning that it was the first welded steel bridge in Europe and the first road bridge of this type in the world. The bridge consists of two main truss beams with a straight bottom chord and parabolic top chord, as presented in **Fig. 15.6a,b,c**. The main truss beams are connected with crossbeams and horizontal braces located at the bottom chord, without braces at the top chord. In 1977 the bridge was closed and is today exhibited as a technical monument.

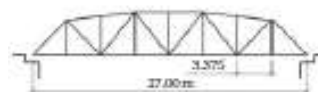
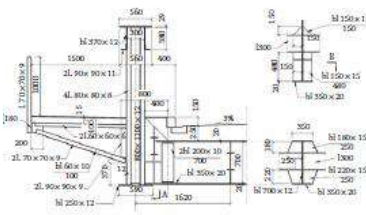


**Fig. 15.6a,b,c** The first welded steel road bridge in the world erected over the Słudwia River near Łowicz (1929).

## 16. Introduction to Bridges in Russia

Extensive progress in bridge construction in the beginning of the nineteenth century was influenced by overall industry development [1.Chapter 15, S. A. Blank, V. A. Seliverstov]. A number of cast iron arch bridges for roadway and railway traffic were built. At about the same time construction of steel suspension bridges was started in St. Petersburg. In 1824 the Panteleimonovsky Bridge over the Fontanka River, with a span of about 40 m, was built. In 1825 the pedestrian Potchtamsky Bridge over the Moika Bankovsky was constructed, and the Lion's and Egyptian Bridges over the Fontanka River, with spans of 38.4 m, were constructed in 1827. In 1851–1853 two suspension bridges were constructed over the Velikaya River in Ostrov with spans of 93.2 m. Development in suspension bridge systems was based on the invention of wire cables. In Russia, one of the first suspension bridges using wire cables was built in 1836 near the Brest–

Litovsk fortress. This bridge crossed the West Bug River, with a span of about 89 m. Bridge design and construction practice in former USSR, especially Russia, is not much known by foreign engineers. Many advanced structural theories and construction practices have been established. In view of the global economy, the opportunities to apply such advanced theories to practice became available with the collapse of the iron curtain. In 1931.

### 16.1. Historical Iron and Steel Bridges

The most widespread types of bridges in past times were timber and masonry bridges. Because there were plenty of natural wood resources in ancient Russia, timber bridges were solely built up to the end of the fifteenth century. Ivan Kulibin, Russian mechanics engineer, (1735–1818) designed a timber arch bridge over the Neva River with a span of about 300 m, illustrating one attempt in the search for efficient structural form. The first I-beam bridges in Russia were the Semenovskiy Bridge over the Fontanka River in St. Petersburg, constructed in 1857, and the bridge over the Neman River in Kovno on the St. Petersburg–Warsaw railway, constructed in 1861. However, I-beams for large spans proved to be very heavy and this led to a more wide application of truss systems. The construction of the railway line in 1847–1851 between St. Petersburg and Moscow required a large number of bridges. Zhuravskiy modified the structural system of timber truss implemented by Howe in 1840 to include continuous systems. These bridges over the Volga, Volhov, and some other rivers had relatively large spans; for example, the bridge over the Msta River had a 61.2 m span. This structural system (known in Russia as the Howe–Zhuravskiy system) was widely used for bridge construction up to the mid-twentieth century. Steel truss bridges at first were structurally similar to types of timber bridges, such as plank trusses or lattice trusses. A distinguished double track railway bridge with a deck truss system, designed by Kerbedz, was constructed in 1857 over the Luga River on the St. Petersburg–Warsaw railway line (**Fig. 16.1**).

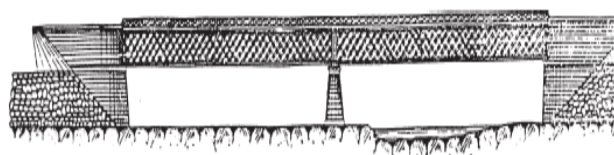


**Fig. 16.1** Bridge over the Luga River (1857)

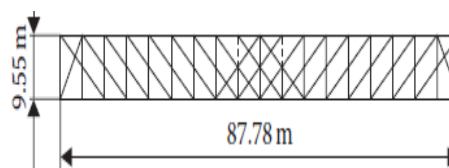


**Fig. 16.2** Standard truss superstructure.

The bridge consists of two continuous spans (each 55.3 m). P-shaped cross sections were used for chord members, and angles for diagonals. Each track was carried by a separate superstructure that includes two planes of trusses spaced at 2.25 m. The other remarkable truss bridge built in 1861 was the Borodinsky Bridge over the Moskva River in Moscow, with a span length of 42.7 m. Until the 1880s, steel superstructures were fabricated of wrought steel. Cast steel bridge superstructures appeared in Russia in 1883. In 1884, Belebubsky established the first standard designs for steel superstructures, covering a span range of 54.87–109.25 m. For spans exceeding 87.78 m, polygonal trusses were designed. A typical superstructure with a 87.78 m span is shown in **Fig. 16.2**. The first riveted triangular truss bridge in Russia was constructed in 1887 on the Romny–Krmenchug railway line. Many beam bridges built in the mid-nineteenth century were the continuous span type. In order to transfer a continuous system to a statically determined cantilever system, hinges were arranged within the spans. This led to a new direction in bridge construction.

## 17. Introduction to Bridges in Serbia

The Republic of Serbia, has a population of about 7.3 million. [1. Chapter 16, R. Folic]. There are 2638 bridges on the roadways of the Republic of Serbia. with a total surface area of 800,000 m<sup>2</sup>, of which 215 bridges are on highways, 996 bridges are on first-order state-owned roadways, and 1427 bridges are on second-order state-owned roadways.

### 17.1. Historical Iron Bridges

The first two iron bridges in Serbia were constructed in 1883 over the Danube in Novi Sad (supported by six concrete columns) and 1884 over the Sava in Belgrade, respectively. Since the Franz Joseph Bridge in Novi Sad was a railway bridge with an additional pedestrian footway, pontoons and ferries were used for roadway traffic. The roadway–railway bridge near Bogojevu over the Danube was constructed in 1911. The railway bridge Bogojevu–Erdut (**Fig. 17.1**) was

built of seven single-span trusses, with a total length of 620.7 m. The first bridge was built in 1911, containing six single-span trusses on two abutments and five piers. It was destroyed in 1941. The second structure, which is still in operation, is a combined highway and railway bridge, built on the original piers and abutments. One side span was divided into two, thus, a new pier was built there. Now the bridge carries only a railway line, as a new highway bridge was built nearby.

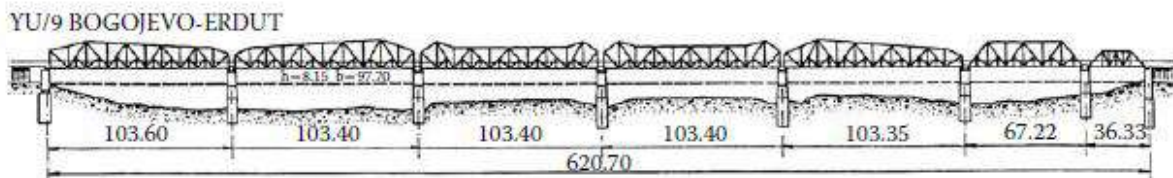


Fig. 17.1 Longitudinal section of Bogojevo-Erdut railway bridge. (Ivanyi and Stipanac 1998.)

During the war in 1915 a roadway bridge was constructed over the Danube near Novi Sad. *It was destroyed by floating ice in February 1924 as a consequence of inadequate ice blasting.* It was in service until 1922. Also in 1915, a roadway bridge was constructed over the river Tisa near Titel for war purposes, which was in service until 1927. Modern bridge construction over the Danube began in 1928, with the construction of a roadway bridge—one of the most beautiful bridges in Europe at the time, in Novi Sad. Named the Bridge of Prince Tomislav, it was mounted without supporting yokes in the river bed. Unfortunately, the bridge was demolished in 1941, and the new, Tito Bridge was erected on its pillars (later named Varadin Bridge) for only 160 days; it was the first steel bridge constructed in Europe after World War II (Fig. 17.2).

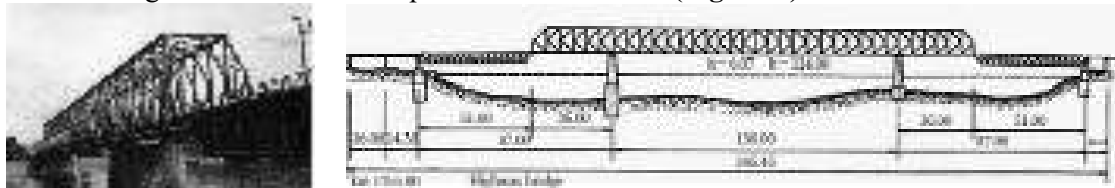


Fig. 17.2 Varadin Bridge on the Danube in Novi Sad (destroyed in 1999). (Ivanyi and Stipanac 1998.)

The bridge was opened to traffic in January 1946 and was destroyed during the NATO campaign on April 1, 1999 (Folić, Radonjanin, and Malesev 2000). There are 37 steel truss bridges over the Danube. The Varadin Bridge, connecting the central part of Novi Sad with Petrovaradin, was erected on the well-preserved piers of the Prince Tomislav Bridge, from May 1945 to January 1946. It was mainly built of the remains of the old railway bridge. Until 1962 it was used both for road and railway traffic, and afterwards only for roadway and pedestrian traffic. The truss was a gerber beam girder system (with a span of 87m + 130m + 87m). (Fig. 17.6).

## 18. Introduction to Bridges in Slovakia

Slovakia became an independent country on January 1, 1993, after peaceful separating of Czechoslovakia into Czech Republic and the Slovak Republic. It is located in Central Europe, bordering Hungary, Poland, Czech Republic, Ukraine, and Austria. The total area of Slovakia is 49,035 km<sup>2</sup>. The population is about 5.43 million people. [1. Chapter 17, I. Baláž]

### 18.1. Historical Cast Iron Bridges

The first cast iron bridge (Fig. 18.1 and 18.2) in Middle Europe was built in 1810 in Hronec (Rohniz, Kiszgaram) near Podbrezová during the time of the Hungarian Kingdom (Nesvadba and Plevák 1982). There were seven blast furnaces on Hronec in 1740. The rolling mill of the plate originated in Hronec in 1814. The bridge had an arch span of 3.01m and consisted of 36 components, including 5 arch ribs. The bridge had a length of 4.87m, width of 4.845m, and height of 1.31m. The bridge served 152 years until 1962. Only part of the bridge has survived, consisting of three arch ribs. It is exposed in the front of the local foundry. Bridge details are shown in Fig. 18.2.



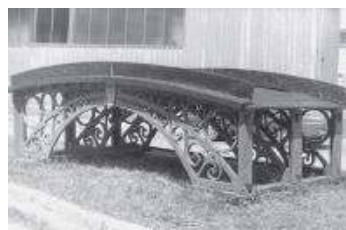
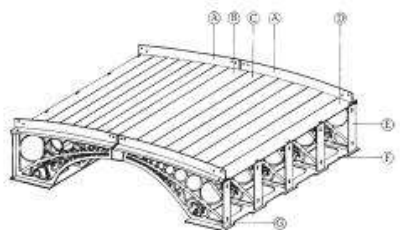


**Fig. 18.1** The first cast iron bridge in Hronec (1810–1962)



**Fig. 18.2** Details of the first cast iron bridge in Hronec (1810–1962) **Fig. 18.3** The third cast iron bridge in Hronec over Čierny Hron River (1815–1945).



The second cast iron bridge in Hronec was built probably at the same time as the first one. The second bridge, with four arch ribs, was covered by stone. It was 5m long and 3.5m wide. After repeated repairs it was removed in 1942. Its remaining parts disappeared. The third cast iron bridge (**Fig. 18.3**) in Hronec over Čierny Hron River was built in 1815. The bridge, with five arch ribs, was 10.2 m long and 4.9 m wide. A rib consisted of two half-arches connected in the middle of the span. It was destroyed by German soldiers in 1945.

### 18.2. Rail Network and Railway Bridges

The current railway network of the Slovak Republic is with a total length of 6876 km. As of 2011 there were 2282 railway bridges with a total length of 50.3 km, including 456 steel bridges and 1826 concrete bridges. About 25% of bridges were built before 1933 and 95% of bridges are in good or sufficient condition. It should be noted that 798 bridges, 72% of the total 1111 bridges, were destroyed during World War II, from 1944 to 1945.

### 18.3 Longest Bridge Spans in Slovak Republic

The longest bridge spans in Slovakia with record span lengths in various types of static schemes are listed in next some photos (**Fig. 18.4–18.9**).

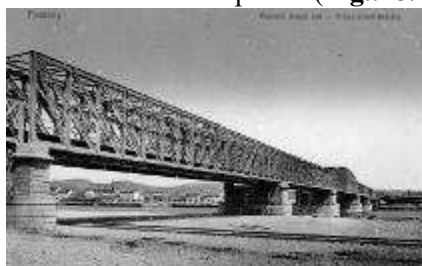


**Fig. 18.4** Emperor Franz Joseph Bridge over Danube River **Fig. 18.5** Steel lattice railway bridge in Marchegg over Morava River (1868–1945).



**Fig. 18.6** Steel lattice railway bridge over Váh River in Turany (1871) **Fig. 18.7** Viaduct Pod Dielom (under Mountain Diel)







**Fig. 18.8** Devils Viaduct over Devils Valley on the Brezno

**Fig. 18.9** Bridges over Danube River in Bratislava. Downstream from the left: Lafranconi Bridge (1991), New Bridge (1972), old road (1945), and railway (1950) bridges on the same piers, Apollo Bridge (2005), Port Bridge (1983, 1985) under an airplane, not seen. (Courtesy of M. Maťašík.)

## 19. Introduction to Bridges in Turkey

### 19.1. Historical Development

In 1923, at the time of the establishment of the Republic of Turkey, the total length of the roads was around 18,000 km with 94 bridges. In 1929, an early version of the Highway Department was established within the Ministry of Public Works. Not many bridges were constructed between 1923 and 1950. In the 1950s, bridge and road construction were accelerated and the total length of highways reached 62,000 km. The total number of bridges reached 5860 with a total length of 255 km. The dominant bridge type in Turkey is the simply supported reinforced concrete bridge, with about 2800 bridges and a total length of 82 km. Steel-type bridges in highways are quite rare. On the other hand, the steel-type bridge is the most common one in railways[1.Chapter 18, Cetin Yilmaz, Alp Caner, Ahmet Turer]

#### \* Steel bridge in Mekece

**Steel girder bridges are not common in Turkey.** Steel girders are typically used to span more than 40 m of clear span length. The construction of a composite steel girder bridge in Mekece is shown in **Fig. 19.1**. The composite effect between the steel girder and the concrete deck is achieved by the I-profile shear studs, as shown in **Fig. 19.2**. Each 58.5 m long span has five 3 m deep girders connected by cross braces at a certain spacing. The allowable tension stress is equal to 140 MPa. The 25 cm thick reinforced concrete deck has a 28-day concrete compressive strength of 30 MPa.



**Fig 19.1** Construction of a steel girder bridge **Fig. 19.2** I-profile shear studs over the top steel flange.

#### \*\*Karakaya Bridge

The Karakaya Bridge, 2030 m in length, was built over a dam reservoir close to the city of Malatya in the late 1980s. The main construction phases were foundation works, pier construction, and steel truss erection. The 64 m long steel truss superstructure was supported with hollow concrete bridge piers spaced about 70 m from each other, as shown in **Fig.19.3**. Each steel span was weighed about 366 tons. The yield strength used in the design of the steel elements was 235 MPa. The steel trusses were built on the ground and lifted to their positions, as shown in **Fig.19.3**. The lift speed was 6 m/h. Each pier was constructed using a traveling form. The longest pier was about 58 m long. The foundation was formed on piles.



Fig. 19.3 Karakaya Bridge around the mid-1980s

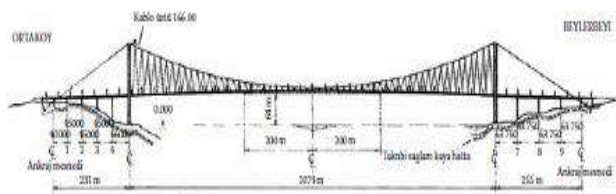


Fig. 19.4 Bosphorus Bridge elevation drawing.

### \*\*\*Bosphorus Bridge

The Bosphorus Bridge is located on the major Istanbul belt highway, close to the Marmara Sea at the shortest distance on the Bosphorus (Fig. 19.4). The bridge has a total length of 1560 m and a main span of 1074 m. The bridge has extraordinary shaped cables that are inverse “V” shaped and located at 59 positions along the length of the main span (Fig. 19.4). The Bosphorus Bridge has a width of 25 m which carries total of six lanes of traffic, three lanes in each direction (Fig. 19.4). The bridge opened to traffic in October 30, 1973, for the fiftieth anniversary of the Turkish Republic.

## 20. Introduction to Bridges in Ukraine

Ukraine, the largest European country, has a population of almost 47 million people. The country is bisected in the north–south direction by the Dniro River. Bridges crossing the Dniro play a vital role in the Ukraine transportation system. The Ukrainian bridge construction industry, much affected by economic crisis in the early 1990s, is currently in the process of revival. At the present, three major bridges over the Dniro River are being constructed, two in the city of Kyiv and one in Zaporizhia City.

### 20.1. Historical Iron Bridges

The first iron bridge over the rivers was built in railway construction in the second half of the nineteenth century. For long-span railway bridges across the Dniro, iron closed-mesh lattice trusses were used. Two of the structures of this type, in Kyiv (1870) and in Dnipropetrovsk.; 1884), are shown in Figures 20.1 and 20.2, respectively[1. Chapter 19, Mykhailo Korniev].



Fig. 20.1 Bridge in Kyiv across the Dniro, 1870.



Fig. 20.2 Bridge in Dnipropetrovsk across the Dniro, 1884.

The bridge in Kyiv had 14 simple spans of 76 m with a total length of 1068 m. It was at that time the longest bridge in Europe. Individual truss spans were assembled on the bank and floated to their locations on the piers. The total weight of iron for this structure was about 4000 tons. Every day seven trains crossed the bridge at a speed of 30 km/h (Fig. 20.1). The two-level railway bridge in Dnipropetrovsk (then Katerynoslav; 1884), with 15 spans at 83 m and a total length of 1250 m, was even longer (Fig. 20.2). Its total weight was 9525 tons. The bridge was destroyed twice, in 1918 and 1941, respectively and was rebuilt twice in 1920 and 1943, and then in 1955 it was completely rebuilt. The first major bridge for vehicular and pedestrian use was built in 1853: the chain suspension Mykolaivskyj Bridge over the Dniro in Kyiv. This bridge is 7909 m long, with 68.58 m + 4x134.1m + 68.58m fixed spans and one swing span of 15.24m (Fig. 20.3). The width of the deck is 16m, with a vehicular roadway 10 m wide. The iron structure, weighing 1600 tons, was fabricated in Birmingham, England, delivered by steamships to Odesa, and carted by oxen carts to Kyiv. The bridge was designed for a live load of 520 kg/m<sup>2</sup>. This bridge was designed by the British engineer Charles Blacker Vignoles. A silver bridge model was exhibited in London in 1854.



**Fig. 20.3** Mykolaivskyj suspension bridge in Kyiv across the Dnipro, 1853.



**Fig. 20.4** Kichkas bridge across the Dnipro, 1902

The Kichkas Bridge over the Dnipro near Zaporizhia was built in 1902 (**Fig. 20.4**). This bridge was designed by famous bridge designer L. D. Proskuryakov (author of works in structural mechanics and teacher of E. O. Paton). The bridge had span of 190 m and arch height of 20 m, with a total length of 336 m. On the top level was two railway tracks (railway traffic on the bridge opened in 1908) and on the bottom level by two sides of the bridge were pedestrian sidewalks. In 1920 the central part of the span was exploded and was recovered in 1921. Due to construction of the Dnipro hydro electric power plant and its reservoir the Kichkas Bridge was disassembled in 1931.



**Fig. 20.5** Mykolaivskyj bridge failure



**20.6** The rebuilt Mykolaivskyj bridge on old piers



**Fig. 20.7** Rusanivskyj Strait bridge, 1906

*In 1920, during the civil war, one of the spans of the Mykolaivskyj Bridge in Kyiv was blown up and the link chain was broken, which caused its total failure (**Fig. 20.5**). In 1925, a new bridge was built utilizing the existing piers, designed by E. Paton (**Fig. 20.6**). Bridge construction activities were much intensified prior to World War I across all of Ukraine, which was then divided between the Russian empire and the Austro-Hungarian monarchy. Many steel bridges of all types were built during this period. The twin tied arch bridge over the Rusanivskyj Strait in Kyiv, constructed in 1906 and designed by M. Belelubskyj with spans of 100 m, is an example (**Fig. 20.7**). However, most of these bridges were destroyed during World War I from 1914 to 1918 and the subsequent civil war from 1918 to 1921.*

## 21. Introduction to Bridges in China

Even though China erected thousands of ancient bridges, very few of them survived until now. The oldest bridge in China, which survives today and is well-preserved, is a single-span segmental stone arch bridge known as the Anji Bridge or Zhaozhou Bridge. It is located in Zhao County, Hebei Province. It was completed in the first year of Daye's reign of the Sui Dynasty (605 AD). It is composed of 28 individual arch ribs 0.33 m thick boned transversely, 37.03 m in span, rising 7.23 m above the chord line, and averaging 9 m in width, narrower in the upper part and wider in the lower[1.Chapter 20, Quan Qin, Gang Mei, Gongyi Xu].

### 21.1. Historical Development of Steel Bridges

#### \*Qiantang River Bridge

A railway-highway bipurpose bridge (**Fig. 21.1**) with a total length of 1453 m, the longest span being 67 m, was constructed over the Qiantang River. It might be the first modern bridge designed by Chinese engineers and is a double-deck highway-railway bi purpose steel truss bridge. The 6.1 m wide upper deck supports two highway lanes and two 1.52 m wide sidewalks. The lower deck supports a single-track railway. It consists of 2×14.63 m spans of steel plate girders and 16×65.84 m spans of simply supported parallel chord rivet Warren trusses with a height of 10.7 m and center-to-center distance of 6.1 m, supported by reinforced concrete piers. It was designed by the Design Group of the Qiantang River Bridge, constructed by Corritt Co. (Denmark) and Dormanlong Co. (England), and completed in October 1937.

**\*\*Wuhan Yangtze River Bridge**

In 1957, the Wuhan Yangtze River Bridge (**Fig. 21.2**), the first bridge across the Yangtze River, was built. It is a highway– railway bipurpose double-deck steel truss bridge with nine main spans of 128 m and a total length of 1155 m. The 18 m wide upper deck supports six highway lanes and two 2.25 m wide sidewalks. The lower deck supports a double-track railway. Its major part consists of three 128 m span continuous steel trusses supported by reinforced concrete piers and tubular concrete piles. The trusses were erected by the cantilever method and pipe foundations were driven by vibrators for the first time. It was designed by the Major Bridge Reconnaissance and Design Institute, constructed by the China Railway Major Bridge Engineering Bureau, and completed in 1957.



**Fig.21.** Qiantang River Bridge



**Fig.21.2** Wuhan Yangtze River Bridge



**Fig. 21.3** Nanjing Yangtze River Bridge

**\*\*\*Nanjing Yangtze River Bridge**

The Nanjing Yangtze River Bridge (**Fig. 21.3**) is a highway–railway double-deck continuous steel truss bridge in Nanjing, Jiangsu Province across the Yangtze River. The upper deck provides four lanes of highway traffic, which are 15 m wide, plus two sidewalks each 2.25 m wide, and the lower deck carries two railway tracks. The main bridge is 1576 m long. Ten spans were arranged for the main bridge, including a 128 m simply supported steel truss side span and nine 160 m continuous steel truss spans, continued every three spans. The main truss is a parallel chord rhombic truss with a reinforcing bottom chord. It was erected by cantilever assembling. Considering the complex geologic conditions at the bridge site, different types of foundations were used, including heavy concrete caissons with a depth freeway of penetration reaching 54.87 m for areas with shallow water and deep coverings and a floating-type steel caisson combined with pipe column foundations, which was used for the first time at a deep water site. The bridge was designed and constructed by the China Railway Major Bridge Engineering Group Co., Ltd. It was completed in December 1968.

**22. Introduction to Bridges in Indonesia**

Indonesia is the world's largest archipelago country, covers around 17,000 islands. The accumulation of land and water is around 1.9 million km.sq. The population in Indonesia is 222 million. Over half the population lives on Java Island, which is only around 7% of the total area. In general, the population is concentrated on five major islands: Sumatra (474 km.sq.), Java (132 km.sq.), Kalimantan, the third world's largest island (540km.sq), Sulawesi (189 km.sq.), and Papua New Guinea (422 km.sq.). Most of these main islands, except Kalimantan, are located along earthquake rings.[**1. Chapter 21, W. Dewobroto, L. Hidayat, H. Vaza**]

**22.1. Historical Development**

The most popular bridge type in Indonesia is the steel bridge. From 1970 to 1990, steel bridges with a total length of about 237 km were imported from various sources, for instance, the United Kingdom, the Netherlands, Australia and so on. **Fig. 22.1** shows the Binamarga standard steel bridge. One of the main reasons for using a steel truss bridge is due to the high strength-to-weight ratio, so that the structure is relatively lighter, which results in smaller foundations. Another special feature is that steel truss elements are prefabricated in workshops and assembled on the project site by bolting small segments together. Standardized elements and products have made the steel truss bridge segments(spans of 40m to 60m and some up to 80m and 100m), easy to put together with high quality control and high precision, as shown in **Fig. 22.2**.



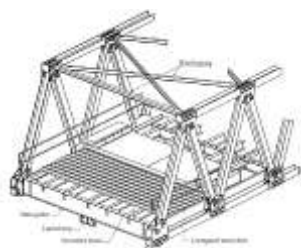


Fig. 22.1 A perspective of the Binamarga steel bridge

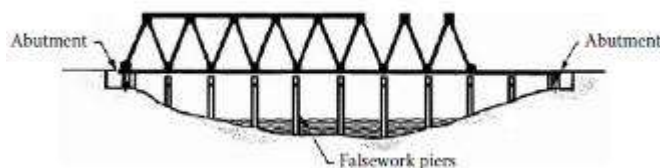


Fig. 22.2 Erection of the falsework.

### ***\*Bantar II Bridge***

The bridge total length is 226 m, composed of four spans ( $51.4\text{m} + 2 \times 61.6\text{m} + 51.4\text{m}$ ), with a width of 9m ( $1\text{m} + 7\text{m} + 1\text{m}$ ). The maximum span length is 61.6m. This bridge is the one that has taken over the role of the old Bantar Bridge, a suspension bridge inherited from the Dutch (before the independence of Indonesia). The image in Fig. 22.3 was taken from the old Bantar bridge. At the time the photo was taken, the Bantar III Bridge (prestressed concrete girder), located between the old Bantar Bridge and the Bantar II Bridge, was not built. Currently the old Bantar Bridge, Bantar II Bridge, and Bantar III Bridge are lined up in parallel in that location.



Fig. 22.3 Bantar II Bridge (226 m), Yogyakarta



Fig 22.4 Koto Panjang Bridge (295 m), Riau.

### ***\*\*Koto Panjang Bridge***

The Koto Panjang Bridge, in Kampar regency, Riau, Sumatra, was built over a lake that formed as a result of a barrier of the Kampar River for the construction of a dam for a hydropowered electricity source for Koto Panjang (Fig. 22.4). The bridge total length is 295 m, composed of five spans with a width of 6 m. The maximum span length is 60 m.

## **23. Introduction to Bridges in Iran**

Iran has 1,648,000 km<sup>2</sup> of land area and is home to over 70 million people. Its rugged mountainous Landscape has made road building very costly and difficult, requiring tunnels and bridges in most parts. There are more than 316,000 roadway and 21,000 railway bridges in the country. The modern era of bridge building in Iran started after World War I [1. Chapter 22, Sh. Maleki].

### **23.1. Historical Development**

By this time Europeans had gathered tremendous amount of experience in building all types of long-span concrete and steel bridges. Meanwhile, in Iran during this period, no advancement in construction materials or methods was noticeable. The Great Powers desire to expand and enter the global market of engineering was ever increasing after the recession following the war. **The newly found oil reserves in Iran made the country more desirable for such investments.** In the bridge engineering sector the leading role was almost exclusively reserved for a Danish engineering and construction company by the name of Kampsax, which was founded in 1917. Kampsax started its work in Iran in 1933 by winning the contract to build the first major railroad in the country, running from the Caspian Sea in the north to the Persian Gulf in the south. The length of this railroad is 1400 km and it passes through 230 tunnels and 4100 bridges. The railroad and its bridges are still in operation and many of its bridges have survived the moderate earthquakes that happen routinely in the country. All sorts of bridges were constructed for this project including steel, plain, and reinforced concrete, brick, and stone masonry. Following the success of Kampsax, other European countries like Sweden, Germany, England, and later the United States got involved in domestic Iranian projects and have contributed to building many long-span bridges, among which the Sefid and Ghotur bridges are noteworthy. The Sefid Bridge was built in 1936 by a

Swedish company over the Karun River in the city of Ahwaz in southwest of Iran. It has a through-arch main span made of steel with the deck hanging from suspension cables (**Fig. 23.1**).



**Fig. 23.1** Sefid Bridge in Ahwaz

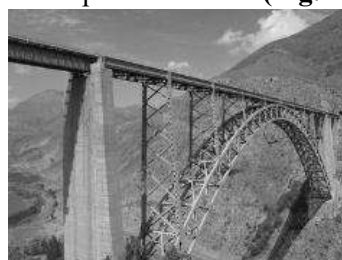


**Fig. 23.2** Ghotur Bridge near Khoy.

The spans consist of two steel arches of 130 m and 136 m length and three deck-type steel arches of 49 m span. Up until 1971 this was the longest span bridge in the country. In 1971, a joint venture of an American and Austrian company designed and built a 223 m trussed steel arch as part of the western Iranian railroad project that connects to the Turkish railroad. The bridge is called the Ghotur Bridge (**Fig. 23.2**) and has a total length of 448 m. It was the longest arch bridge until a decade ago and is the second-longest arch bridge in the country today. Iranian bridge consulting firms played a more active role in the 1970s. **Starting with short-span bridges, eventually over two decades they were able to master the art.**

#### \* Steel Girder Bridges

Most steel girder bridges in Iran are of the plate girder type. The local steel mills do not produce deep rolled sections for bridges. Therefore, designers use welded plate girders even for short spans of 12 m. Consequently, the fabrication cost and the susceptibility to corrosion has made steel bridges less popular for short spans. **Fig. 23.3** shows a typical composite plate girder bridge in the capital city of Tehran over the Chamran expressway. Elastomeric bearings are also the bearings of choice for most bridges in the country (**Fig. 23.4**).



**Fig. 23.3** Plate girder bridge



**Fig. 23.4** Elastomeric bearings



**Fig. 23.5** Sixth Bridge in Ahwaz.

The Sixth Bridge in Ahwaz (**Fig. 23.5**) was built in 2004 using haunched steel composite plate girders. The bridge spans 428 m over the Karun River and has a main span of 75 m. The bridge is 21 m wide and uses multicolumn concrete piers and solid wall abutments (HCE 2011). This is one of the longest plate girder bridges in the country.

#### \*\* Arch Bridges

The Karun-3 Bridge (**Fig. 23.6**) is the longest span arch bridge in Iran today. It was built in 2004 as part of the Karun-3 dam project. It is 12 m wide and carries two lanes of traffic. The two-hinged arch spans 264 m over the river at an elevation of 250 m. The arch is a trussed arch and supports a deck that has four steel girders (**Fig. 23.7**). A total of 2500 tons of steel was used in this project.



**Fig. 23.6** Karun-3 Bridge in Ahwaz.



**Fig. 23.7** Karun-3 steel arch.

## 24. Introduction to Bridges in Japan

The territory of Japan is about 378,000 km<sup>2</sup>. The population of Japan is about 127.43 million as of February 2010 (provisional estimates). **The total length of bridges on the roads is 9,502,572 m as of April 2008.** *The total number of bridges on the roads is 153,529; steel bridges, prestressed concrete bridges, and reinforced concrete bridges occupy: 38.3%, 41.3%, and 16.9%, respectively.* The oldest steel bridge in Japan is Kurogane Bridge in Nagasaki, Nagasaki prefecture, which was constructed in 1868 by a Japanese ship building company. Before World War II, only a few welded steel bridges were constructed. In 1939, specifications for welded steel bridges were issued, and were revised in 1957. Since then, welded steel bridges have been preferred and have become commonly used. ***Steel-concrete composite bridge construction started in 1951 in Japan.*** After issue of the guideline for composite bridge construction in 1960, construction of composite bridges started increasing. Since the late 1970s, in Japan seldom employ composite design. In the 1960s, the so-called modern steel plate girder bridge system was established in Japan and since then this system has been preferably employed. [1. Chapter 23, M. Nagai, Y. Okui, Y. Kawai, M. Yamamoto, K. Saito]

### 24.1. Historical Development

From 1980 to 2000, Japanese bridge engineers enjoyed long-span bridge construction, in particular long-span cable-supported bridge construction, in the Honshu-Shikoku islands connecting project, on the Tokyo and Osaka-Kobe bayshore route projects. In these big projects, the world's longest suspension bridge, Akashi Kaikyo Bridge, with a span of 1991 m, and the third-longest cable-stayed bridge, Tatara Bridge, with a span of 890 m, were constructed. Cost reduction for bridge construction became top priority in the early 1990s, when new Tokyo-Nagoya-Osaka Expressway construction began. Responding to this, the steel-concrete hybrid (composite and mixed) structure has been preferably selected from an economical viewpoint. In 20 years, around a half or more existing bridges will exceed 50 years of age. Repair, strengthening, and rebuilding of old bridges probably will be one of the future major works in Japan.

#### \*Steel Girder Bridges

The first iron plate girder bridge in Japan was Kurogane Bridge in Nagasaki, and is a riveted cast iron bridge completed in 1868. It was reconstructed three times, most recently completed in 1990 as a walkway bridge. Imajuku Bridge (bridge length 9.5 m; width 3.2 m) was completed in Kanagawa in 1931, and was the first welded steel plate girder bridge in Japan. It is the oldest welded highway steel bridge in Japan. *In 1952, Hon Kyu Bashi Bridge in Hyogo (bridge length 112.9m; maximum span length 24.6m; width 9m) was completed. It was the first all-welded highway steel plate girder bridge in Japan, and is a five-span cantilever-type plate girder bridge.* Steel girder bridges are the most popular bridge type for medium span lengths ranging from 30m to 200m, especially in urban highway viaducts. Since the mid-1990s, the concept for optimum design has widely changed. The minimum fabrication cost design, instead of the minimum steel weight design, became considered the optimum bridge design procedure, because of rising labour costs. One of the solutions to the minimum fabrication cost design is twin I-girder bridges. Twin I-girder bridges have been specially developed for the New Tomei Expressway project, headed by the Japan Highway Public Corporation (currently Nippon Expressway Company Limited). Since 1995, more than 800 bridges of this type have been constructed in Japan. **Fig. 24.1** shows **Warashinagawa Bridge** (total bridge length 1005 m; maximum span length 75m; width 18.01m).



**Fig. 24.1** Warashinagawa Bridge **Fig. 24.2** Tokyo Bay Aqua-Line Bridge **Fig. 24.3** Shin-Koiwa Overpass .

This bridge was completed in 2003 on New Tomei Expressway and the total steel weight is 6938 tons. The span length of the slabs is 11m. When Warashinagawa Bridge was constructed, rapid construction, cost savings, and structural durability were required. To satisfy these demands, various advanced construction technologies, such as a large traveling form and advanced field joint methods for deck slabs, were adopted. **Trans-Tokyo Bay Highway(Fig.24.2)** (TTB; total length 15.1km) was completed in 1997 and connects Kawasaki City in Kanagawa, on the west side of the



bay, to Kisarazu City in Chiba, on the east side of the bay. The bridges are continuous steel box girders whose span lengths decrease gradually from two 240m central spans to the approach spans. *In recent years, steel girder bridges with a particular purpose have been developed. To solve this problem, several "quick construction methods" for crossover bridges, in which steel plate girders or box girders are used for the main girders, have been developed.* **Shin-Koiwa Overpass(Fig.24.3)** (length of bridge 557 m; maximum span length 26 m; width of bridge 9 m; total steel weight 2780 tons), is one of the typical applications of "quick construction methods." The bridge was completed in 2007. It is a four-span continuous steel orthotropic box girder bridge. In this case, the flyover bridge, with a length of about 818 m, was completed in only 108 days.

#### **\*\*Steel Arch Bridges**

Steel arch bridges in Japan have been applied to medium- and long-span bridges ranging from 40 m to 300 m. The first steel highway arch bridge, Eitaibashi Bridge in Tokyo (bridge length 184.7 m; bridge width 22 m), was constructed in 1926 and it is the oldest existing steel tied arch bridge in Japan. This bridge was reconstructed from a steel truss bridge with wooden decks to the steel arch bridge as part of the Great Kanto Earthquake Reconstruction Project. **Ujina Bridge** (total bridge length 550m; main span length 270 m; bridge width 18.8m), as shown in **Fig. 24.4**, was completed in 1999 and the total steel weight is 8585 tons.



**Fig. 24.4** Ujina Bridge



**Fig. 24.5** Goshiki Zakura Big Bridge

This is a three-span continuous box girder bridge with steel deck slab reinforced with a monochord arch member, and it has the longest girder span length in Japan. **Goshiki Zakura Big Bridge** (total bridge length 142.2 m; span length 142.2 m; bridge width 17 m), as shown in **Fig. 24.5**, was completed in 2002 and the total steel weight is 4036 tons. This bridge was the first double-deck-type Nielsen arch expressway bridge in the world without upper lateral bracings. The New **Kitakyushu Airport** Access access bridge (total length 2100 m; main span length 210 m), as shown in **Fig. 24.6**, was completed in 2005, and has a unique configuration. This bridge connects the man-made island airport, New Kitakyushu Airport, and the Kyushu mainland. It is the first balanced type steel arch bridge with a monochord arch rib in Japan and a ten-span continuous steel box girder bridge. New **Saikai Bridge** (bridge length 620 m; main bridge 300 m + approach bridge 320 m; bridge width 14 m), as shown in **Fig. 24.7**, is a steel braced-rib half-through arch bridge with a 230 m arch span.



**Fig. 24.6** New Kitakyushu Airport access bridge



**Fig. 24.7** New Saikai Bridge

#### **\*\*\* Steel Truss Bridges**

Steel truss bridges in Japan have been applied to medium and long span lengths ranging 40m to 500m and started with imported bridges from Europe. The first truss bridge, Yoshida Bridge in Kanagawa (bridge length 24 m; bridge width 6 m), was completed in 1869. **It was designed by an English engineer and made of iron.** The bridge has been reconstructed five times. Shinsai-Bashi Bridge in Osaka, which was imported from Germany, was completed in 1873. It was originally made of wrought iron and was reconstructed as a stone arch bridge in 1900. The first steel truss highway bridge in Japan was Eitai-Bashi Bridge in Tokyo, completed in 1897. As mentioned, the



bridge collapsed in the Great Kanto earthquake and was rebuilt as a steel arch bridge in 1926. Third **Chikumagawa Bridge** on the Hokuriku Shinkansen line (bridge length 309 m; span length 103 m), as shown in **Fig. 24.8** was completed in 1996 and the total steel weight is 1885 tons. This bridge is a double truck railway bridge and one of the largest railway truss bridges in Japan. This bridge consists of weathering steel and its colour matches with the environment.



**Fig. 24.8** Third Chikumagawa Bridge (Nagano).



**Fig. 24.9** Sky Gate Bridge R (Osaka)

**Sky Gate Bridge R**, shown in **Fig.24.9**, is the access bridge to Kansai International Airport completed in 1992 and has a total length of 3750 m. This bridge is the longest truss bridge in the world (structural height from 13.7m to 17.5m; bridge width 30 m), and consists of a three-span continuous steel truss, a three-span continuous steel box girder, a two-span continuous steel box girder, and simply supported steel box girders. The bridge carries both roadway and railway access to the airport. The bridge is double-level type and has a six-lane highway on the upper level and a two-track railway on the lower level.

## 25. Introduction to Bridges in Chinese Taipei

Historically, Taiwan has been regarded as a remote island off the central mainland of China. As such, the development history of Taiwan is relatively short, that is, less than 200 years., compared with other parts of China that have a history of thousands of years. A brief retrospect of the construction history of bridges in Taiwan is helpful for us to comprehend the economic development curve of the residents in Taiwan since the last quarter of the nineteenth century. [1. **Chapter 24, Yeong-Bin Yang, Dyi-Wei Chang, Dzong-Chwang, Ping-Hsun Huang**].

### 25.1. Historical Development

The construction of railways in Taiwan started at the time when Liu Mingchuan was appointed as the governor in 1884. The railways from Taipei to Keelung, 28.6km long, were completed in 1887 and then extended to Hsinchu in 1893. Among the 106.7km of railways, there were more than 70 bridges, most of which were made of steel or wood. As an important link of the first railways built between Keelung and Hsinchu, the Taipei Bridge crossing the Danshui River was completed in 1889. The upper part of the bridge was made of wood and iron, and the lower part was made of wood only. Starting with their occupation of the Taiwan island in 1895, the Japanese government introduced technologies from the Occident to build highway and railway networks. Most of the bridges constructed during the Japanese-controlled period were reinforced concrete T-girders, arch bridges, steel plate girder and truss bridges, along with some suspension bridges.

### 25.2. Historical Development of Steel Bridges

After Taiwan was handed over to the Nationalist government in 1945, there was a general lack of steel for construction, as most steel was imported from foreign countries: such as in high mountains as suspension bridges or in railways as truss bridges. Among those few steel bridges that were built in this period, the most famous two are the Loufu Suspension Bridge, included as a part of the Northern Cross-Island Highway, and the Xiluo Bridge, with a total length of roughly 2 km. This bridge is a truss bridge with 31 spans, each with a length of roughly 60 m. This bridge has been an important scenic spot in central Taiwan for a long time, attracting visitors from many places. In 1977, the China Steel Corporation began its operations, particularly producing steel for structural use in Taiwan. However, not until 1983, after the completion of Guandu Bridge a steel arch bridge, was steel widely accepted in bridge construction. In the past two decades, due to rapid progress in analysis software and computing devices, enhanced by advances in construction technologies, the construction of steel bridges has entered a new era. Among these, the Gaoping

Creek cable-stayed bridge, with a span length of 330 m and completed in 1999 as part of the Second National Freeways project, deserves special mention.

**\*Truss Bridges- Daan Creek Bridge and Dajia Creek Bridge**

Completed in 1906, both the **Daan Creek Bridge** and **Dajia Creek Bridge** are steel truss bridges of the through type with a length of 62.4 m for each span. The Daan Bridge has a total length of 630 m divided into 10 spans (Fig. 25.1 and 25.2), and the **Dajia Creek Bridge** has a total length of 380 m divided into 6 spans (Fig. 24.3)( 1911). The super trussed girders of both bridges were replaced in the 1960s by the Taiwan Railway Bureau with the foundations kept intact. Both bridges were used until 1998, when the new mountain-side railways were completed. The Dajia Creek Bridge is now used as a bike road by the local residents.



Fig. 25.1 General layout of the Daan Creek Bridge (1906)



Fig. 25.2 Daan Creek Bridge



Fig. 25.3 Dajia Creek Bridge



Fig.25.4 Second-generation Taipei Bridge

**\*\*Second-Generation Taipei Bridge**

The Taipei Bridge of the first generation for highway use was built in 1901 by the Qing Dynasty by transferring the one for railway use. The second-generation Taipei Bridge was built in 1925 as a seven span through-type steel truss bridge. Each span has a length of 62.4 m. The design vehicle load was the standard 8 ton load based on the old Japanese specifications. This bridge was considered one of the eight scenic spots of Taipei for its poetic colourful view under the setting sun (Fig. 25.32). The bridge was rebuilt for the third time in 1969 due to serious corrosion on the steel truss and the lack of strength to sustain increasing traffic loads.

**26. Introduction to Bridges in Thailand**

Thailand is separated into six regions based on geographical characteristics. It consists of northern, southern, western, eastern, north eastern, and central parts. There are many small canals, river branches, and main rivers. Therefore, the need for short- and long-span bridges is relatively high. As a result, bridges in the central part, especially those crossing the Chao Phraya River, require high clearance and longer span lengths. In addition, ship impact analysis also needs to be taken into account in the bridge design. Thus, higher lateral stiffness and strength of the piers is needed to counteract the lateral forces from ship collision or floating logs along the river.

**26.1. Historical Development**

The development of bridges in Thailand started over 750 years ago with the Sukhothai Kingdom. Typical bridges are short spans made from timber or masonry[1.Chapter 25, E. Limsuwan, A. Pimanmas]. Bridges influenced by religion can be seen from eminent temples and royal places (for example: Naga Bridge) (Fig. 26.1). The major function of this bridge is to fly over the pool around the main temple while the minor function, based on religious belief, is to separate the human area and temple area (area of gods). **Therefore, Naga Bridge were created to represent the linkage between human earth and heaven.** These examples of bridge construction show the harmony between Thailand's lifestyle and civil engineering structures. However, the development of modern bridge engineering in Thailand has started solidly since the beginning of the Rattanakosin period.

**26.2. Rattanakosin Bridges**

When King Rama I established the Chakri Dynasty (1782–present), he also created the new capital city on the east side of the Chao Phraya River. For reasons of security and delay of enemy invasion, the old Rattanakosin area (currently Bangkok's downtown) was surrounded by many small canals. Therefore, it was sometimes known as the Rattanakosin islands. Bridges were built to

facilitate commerce and linkage between outbound communities and the Rattanakosin islands. Here, the technology of bridge engineering was progressing continuously. The main function of the bridge in the early Rattanakosin period was mainly to support the transportation of people and animals (i.e., horses and elephants). At this time, there were still no long-span bridges. **Bridges were typically constructed from thick timbers assembled to form a structure**, so that they could be demolished or removed in case of war. By this technique, rivers and canals became natural barriers for the protection of the main islands. However, in case of the Chao Phraya River, even though there was some demand for long-span bridges, it was considered not economical to construct long-span bridges at that time. Thus, the solution was to adopt cross-river boats instead of bridges.

#### \*Bridges before World War II

Before World War II (1939–1945), Thailand was ruled by an absolute monarchy. Civil infrastructure projects were decided under the judgment of the king. Many projects, such as railway, highway, electricity, and water supply, were authorized by the king (Rama V–Rama VII). The first long-span railway truss bridge across the Chao Phraya River was Rama VI Bridge (1927). The original bridge was destroyed in the war and the new bridge was rebuilt in the same place (1953). The new Rama VI Bridge consists of five spans and has a total length of 442.08m. The width of the superstructure is 10m(**Fig. 26.2a,b**).



Fig. 26.1 Naga Bridge



a)



b)

Fig. 26.2 Rama VI Bridge: (a) original Rama VI Bridge and (b) present Rama VI Bridge.

#### \*\*Steel Truss Bridges

Rama VI Bridge can be recorded as the first long-span steel truss bridge across Chao Phraya River. After this bridge was constructed, the other steel truss bridges were created continuously. However, one important steel truss bridge is Memorial Bridge. In order to memorialize the benevolence and kindness of the King Phra Buddha Yodfa, it was named **Memorial Bridge (see: Fig. 26.3a,b)**. The bridge was the first highway bridge across the Chao Phraya River in Bangkok. In the past, to accommodate ship transportation underneath, the middle span of bridge could be opened; however, it is now fully closed. Construction of the bridge began on December 3, 1929, and it opened officially on April 6, 1932. The bridge consists of three spans (through truss for the end spans and half through truss for the opening span). Both end spans are 75.25 m and the opening, Bascule-type span is 60 m. The total length is 229.76m. The width of the superstructure is 10 m. The piers were constructed from double-cell hollow piers and supported by caisson.



a)



b)

Fig. 26.3a,b Memorial Bridge.

## 27. Introduction to Bridges in Egypt

Egypt is located in the north-eastern corner of the African continent. Egypt is a square-shaped country, and has a north-to-south distance of 1,024 km and east-to-west distance of approximately 1,240 km. The Suez Canal divides the total land into two parts: the mainland and the Sinai Peninsula. [1.Chapter 26, Mourad M. Bakhoun].

### 27.1. Historical Development

It is quite strange that the ancient Egyptians with all their huge and advanced construction did not build bridges or have impressive ones. Boat construction in ancient Egypt is one of the most interesting fields when discussing bridges. *The reasons have a paradox nature*. On one hand the

advanced methods the ancient Egyptians used in building boats and the high level of technology they reached may be one of the strongest reasons why they did not build bridges. On the other hand one finds that the same advanced methods and techniques as those used in constructing some elements in their boats are the same those used now for building cable (hanging) bridges and cable structures. It is quite interesting to see how they reached this level with intuition and through trial and error, without the knowledge of the theories concerning stresses, forces, etc. With the increasing use of modern transportation techniques since the late 1800s and early 1900s, and the wish to connect the two sides of the Nile, the existence of bridges crossing the Nile became one of the priorities in governmental projects.

#### \*Movable Swing Bridge Over the Suez Canal at Ferdan

The fourth bridge at El-Ferdan was completed in 1963 (Fig. 27.1 and 27.2). It was a double-cantilever truss girder bridge with an electro-mechanical operated opening swing span of 167.5 m. The bridge had an overall length of 318 m and provided a clear width for shipping of 148 m. The bridge provides a single-track rail crossing of the Suez Canal with a 3-meter wide roadway on each side of the rail track, which is fixed on wooden sleepers supported directly by the steel deck. The cantilever trusses are 60 m deep at the pile caps and the main span is 340 m, which would be exceptional even for a static truss structure. Steel members were brought to the site in pieces by road from or from Germany through the ports of Alexandria and Port Said. Sufficiently well in advance of erection, these pieces were welded together to form the truss and deck members. Here, 1,200 tons of mechanical items are built into the 10,500 tons of the net steel superstructure. The overall time for bridge opening or closing is 30 minutes.

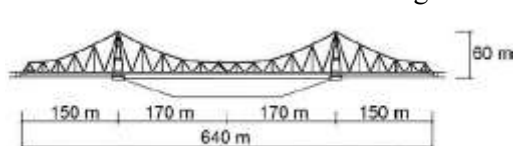


Fig. 27.1 Elevation view of the superstructure

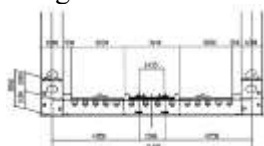


Fig. 27.2 Cross-section of the road deck



Fig. 27.3 Cross-section at pylon.

The bridge's net weight is approximately 1,150 tons of each superstructure, half of which is situated in the bridge rotation centre. The truss system net weight is 2,800 tons. Each of the 320m long road deck areas of the 150 m and 170 m long cantilever arms have a weight of 1,000 tons. The remaining 100 tons is needed for wind bracing and minor construction components. The pylon and most of the remaining truss walls consist of fully welded box sections. Only the short posts and the tension diagonals at the bridge ends are designed as open sections (Fig. 27.3). To simplify transportation, the road deck member is divided into six deck panels with dimensions of 3.50m × 18.00m for each lower chord. The outer two deck panels form the road deck, whereas the center panel carries the railway track. The longitudinal stiffening of the road decks is achieved by cold-formed trapezoidal stiffeners. The welded cross girders transmit the road and railway loads into the main truss system.

#### \* \*Bridge Movement

When the details of such a system were worked out, it became clear that, although the time taken to swing the bridge through 90° would be about 12 minutes, the total time for a closing operation would be about 55 minutes (Fig. 27.4a,b,c). Following discussions between reviewer and Contractor, the Contractor proposed an alternative system, which would take advantage of modern electronic systems and would reduce this time to about 30 minutes. This proposal was accepted and the design has proceeded on this basis.



Fig. 27.4 a,b,c: Bridge positions during swing to 90° (taken after 12 minutes) and closing (taken after 55 minutes).

## REFERENCES

[1] W.F.Chen, L.Duan., Handbook of International Bridge Engineering, CRC Press, 2014 by Taylor&Francis Group, LLC, pp. 1376.





**ARCHITECTURAL TYPOLOGY OF THE  
MULTI-FAMILY RESIDENTIAL BUILDINGS IN BULGARIA  
IN THE 1900- 2013 PERIOD  
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***Testing of renewable raw material Typha (bulrush) with reference to the production of building boards and their use in Bulgaria for thermal insulation and interior insulation of exterior walls of buildings, including training sessions***

**1.Housing construction from 1879 to 1919**

After the liberation of Bulgaria from the Ottoman rule (1878), alike to what was happening in other European countries the tendencies of construction with massive walls grew at the expense of the more popular, lightweight post-and-beam construction (Fachwerk), with fillings of packed clay, bricks, adobe and randomly applied stone inserts. Construction in the villages and towns continued to be mostly in the form of one-, or more often two-storey singlefamily detached homes. The only larger new buildings from the first decades of the 20<sup>th</sup> century until the 20's were schools, city halls, barracks, chambers and other public buildings of the kind. In the more developed cities, there rose hotels, mostly two- or three-storey, which were considered big for their time. Larger commercial buildings also sprung: they were designed as string or block structures of two-storey shops, where the upper level was usually used as office or warehouse space, or rented out as a lucrative site.<sup>1</sup> In that period built were also the first cathedral temples - massive in structure and dimensions, as well as large monolithic monastery wings.

During this first period until the beginning of the decade after the First World War, housing construction was dominated by the parlour-type functional design not only because it was characteristic of contemporary European practice but also due to the legacy of old Bulgarian construction traditions. The parlour-type type of residential buildings was the most representative one from the end of the 18th century until 1978 and remained popular in the decades to follow. In this house plan the parlour is a representative living space, but also a territory of functional connections with the other premises. Typically, kitchens, and in many cases the sanitary premises, laundry and some warehouse spaces, were build as external independent low structures linked with the main buildings.

In this period built were also not a few one-storey detached homes standing on high semi-basements where the auxiliary premises, rooms for the servants and the cellar were located. In some cases, for example in the city of Ruse, this set of auxiliary premises was located in a separate building in the yard without a direct functional connection with the house, but sometimes it also occupied part of the ground floor with a separate entrance.<sup>2</sup>

Towards the end of the 19th century, the vestibule planning system appeared as a variant of the parlour-type. Perhaps the reason for the introduction of the vestibule as an internal communication space usually linked with the parlour by a glazed portal was rooted in the multitude of small-size plots in the commercial downtown part. Often they were of great depth at a small width. The latter plots had formed because of the historically determined, continuous hereditary division of the original larger estates. Ultimately, given these narrow fronts, the buildings were constructed with lateral blank walls, great in depth and with only two fa9ades - toward the street and the yard. Thus the central spaces remained without daylight and were used for communication purposes. They were indirectly lighted only through glass portals linking them to the parlour. This

functional pattern was introduced in Bulgaria by both foreign practitioners in the country and Bulgarian architects who had graduated in Western European countries.

The architectural appearance of the period was characterized by the transition from the traditional old facade design developed in the principles of the Bulgarian Revival Period Baroque to the imported Western European styles. Initially, this was Neoclassicism, while later eclectic strands were introduced that were developed mainly on a Baroque basis.

The beginning of the second decade of the 20<sup>th</sup> century was marked by the appearance of Secession buildings which followed both the typical European patterns and a variant specific to Bulgarian architecture, based on borrowings from the representative medieval Balkan architecture and the Bulgarian picturesque architectural style in particular. Bulgarian architectural history refers to this style as National-romantic.

Despite the stylistic differences, urban architecture of this period was characterized by quite rich volumetric decorations, not only on the facades but also in the representative interior spaces. In the small towns and villages, where the houses were still constructed after projects by local masters, the old decoration ideas were still largely applied. Subsequently, however, houses with massive walls and stucco decorations appeared. They were built by these same local masters in the spirit of the new types they had adopted while working with architects and foreign masters. Until 1912, the best of these masters who made draft design plans, built churches and public buildings.<sup>3</sup> (PLATE I)

## 2. Multi-storey housing construction in the period between the two world wars

Shortly before and after the First World War, more than 100,000 refugees arrived in the Tsardom of Bulgaria, escaping from the brutal ethnic cleansing by the Greek, Serbian-Croatian and Romanian states in the Bulgarian lands which they had acquired pursuant to the Neuilly Treaty (1919). This was the second blow after the genocide of 1913 in the Turkish parts of Aegean Thrace, and the violence in the same year in Macedonia and the rest of the Aegean Sea by *antartiko* [Greek armed propaganda]. Or, altogether, since 1879, more than 235,000 refugees have settled in Bulgaria.

Human migration took place in the form of displacement within the country itself as well. Large territories were abandoned when the local Muslim population migrated to Turkey. They were then populated not only by refugees but also by families originating from overpopulated or poorly productive lands. By 1888, according to statistical data, 15.75 percent of the total population in the country lived outside their native places, and by 1900 this was already a fact for 17.49 percent. In this period birth rate was very high not only among the predominantly rural population (20.5 per 1000 inhabitants), but also, although lower, in the urban areas (9.4 per 1000 inhabitants). In 1887, the latter population represented 18.8 percent, and in 1910 - 19.1 percent of the total population. Steady growth of cities was reported. For example, in 1879, the sea port of Burgas had 2,950 inhabitants, while in 1900 they already numbered 11,738.

The capital of Sofia, the city reporting the fastest growth, sheltered about 40,000 refugees. The city expanded along the periphery with large refugee districts, while the old downtown was overbuilt with new structures. The process was regulated by the outdated provisions of the plan approved in 1880 by the City Council (developed on the basis of the preliminary urbanization plan from 1878 authored by architect Adolf Kolar), the first regulatory plan (authored by engineers L. Amadey, Mihad and Sheax, in effect until 1892 when it was updated by architect Nachev).

PLATE I

Even in the largest and most rapidly developing city of Bulgaria, the capital Sofia, until 1920, residential buildings over three floors are exceptions.



Sofia. „Alexander I“ Blvd, circa 1902



Sofia. „Knyaz Dondukov“ Blvd, 1912



Sofia. House „Gendovitch“, arch. Nikola Lazarov, 19



After that, the city's expansion was regulated by partial plans, building after 1897 on the first Settlement Development Act in the Principality of Bulgaria. Updates were made in 1903, 1907, 1910, 1912 and 1914. They were necessitated by the stormy expansion of Sofia. While in 1887 the city's territory was only 2.49 sq km, and 6.64 sq km in 1907, in 1916 - 1918 it spread on 8.5 sq km. **(PLATE II) (PLATE III) (PLATE IV)**

After the First World War, due to the demographic crisis resulting from the huge migration wave from the Bulgarian lands in Macedonia, Thrace, Dobrudzha and the Morava Valley, a new process of intensive urban planning was implemented throughout the country. Again in Sofia, it was most encompassing.

Having started in the period 1920 - 1924, the process produced a comprehensive plan in 1928, when many new residential quarters appeared. One year earlier, the city whose area was 28 sq km was inhabited by about 220,000 people. By 1938, the number of new quarters had grown to 53, and a little before 1934 the territory of the city encompassed already 42 sq km. As early as 1924, however, taking into account the stormy problem-driven development, a special Sofia Construction Zones Plan was adopted, and similar plans were developed for the other cities as the tendencies of growth, albeit not so expressive, were similar to those in the capital city.

Along with the serious refugee urbanization pressure, in the post-war years there was also a significant housing problem. It engaged mainly middle-class families who had insufficient means to build their own single-family detached homes. The process, even though it most evident in the capital city in particular, affected other Bulgarian settlements and especially the cities like Plovdiv, Varna and Ruse. So in parallel with the other post-war problems for Bulgaria, there rose the problem of housing provision of the refugee families, the internal migrants from villages and small towns, to Sofia and the other large settlements, and the families which lacked sufficient funds to build their own single-family detached homes.

Because of all this, the first post-war government of the Agrarian Union, headed by Alexandar Stamboliyski, was forced to actively address the housing problem. And it started developing program-type legal frameworks to solve it. The beginning was put in 1920 with the successive adoption by the National Assembly of two special acts, devoted partially or wholly to the co-operative, mainly multi-storey construction. The acts further elaborated the ideas, developed on the basis of the already existing Co-operative Association Act (1907, amended in 1911), which underlay the development of the co-operations, and referred to the **Act on Associations for the Construction of Residential Buildings (Article 2, Item 6)**. New ideas were governed by the Act on Meeting Housing Needs (1920) and the Economical House Act (1922), subsequently supplemented (for the last time in 1936).

Around 1919, in his desire to develop Sofia as a modern European capital providing sufficient housing space for its inhabitants, architect Lazar Parashkevanov (1890 - 1997), graduate of the Prague Polytechnic, proclaimed the thesis of the necessity of mass construction on the co-operative principle. Perhaps he had taken into account the possibilities of co-operation regulated by the law of 1907, although it contained only one sentence related to housing construction. He spoke on this issue at meetings in metropolitan quarters, highlighting the possibilities by pooling resources from the savings of affiliated individuals, to pool capital for the purchase of land and construction of a common home. The thesis was also defended by the BIAD (Bulgarian Engineering and Architectural Society, founded on February 2, 1893), as well as in a number of publications in the press.

In these conditions the first building cooperative tenements appeared. However, unlike what was happening in Western Europe, these were not lucrative multi-family low-cost residential buildings (such as the German *Zinskasernen*). In Bulgaria, parallel to the wide-scale single-family detached home construction all round the country, as from 1924 the system of **building co-operatives** started developing in the big cities, a unique practice in the world.<sup>4</sup>



PLATE II

Urban development plans of Sofia, illustrating a continuous expansion.



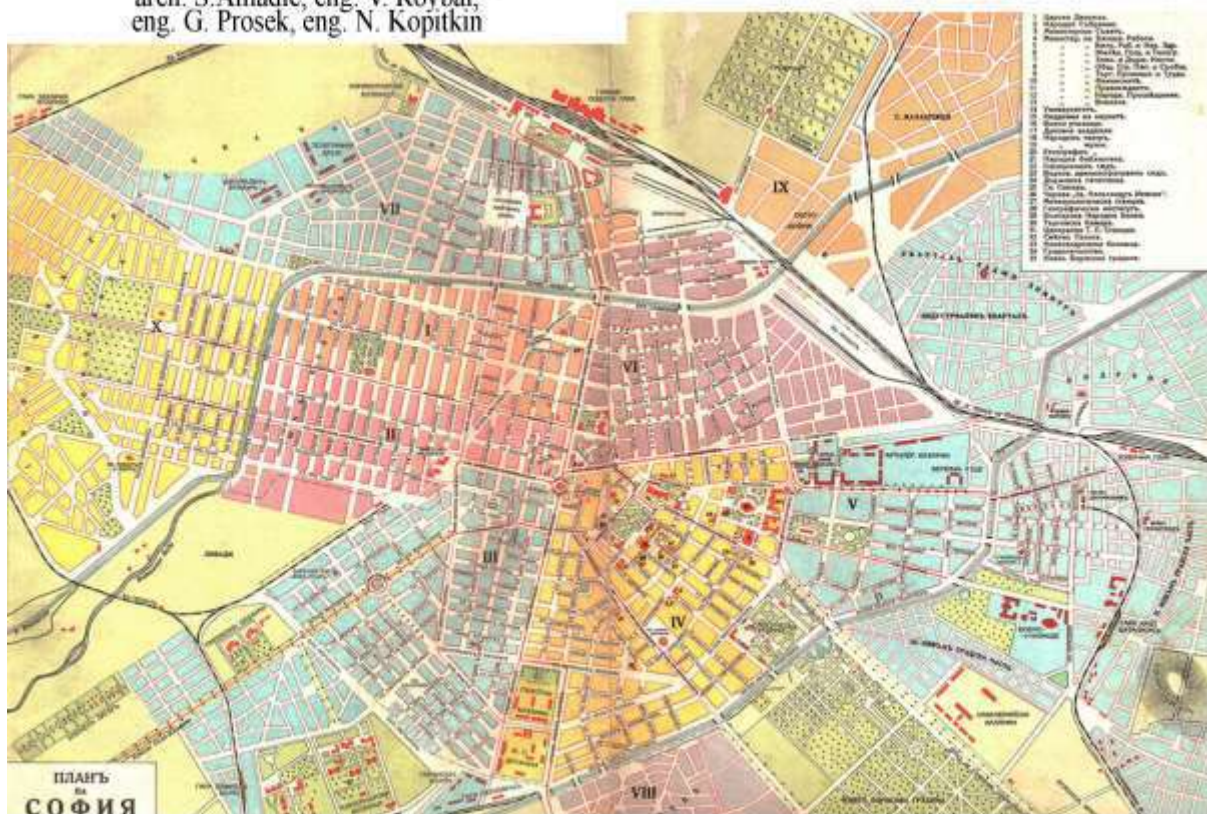
Sofia. 1879 Urban plan designed by  
arch. S. Amadie, eng. V. Roybal,  
eng. G. Prosek, eng. N. Kopitkin



Sofia. 1908 Urban plan designed by arch. Mayer



Sofia. 1919 Urban plan.

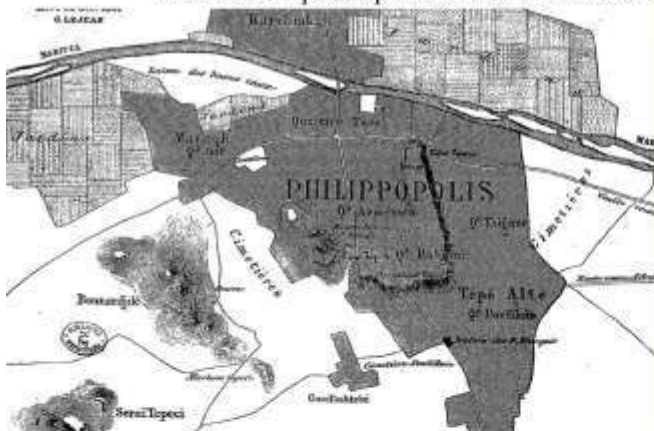


Sofia. 1927 Urban plan.



PLATE III

Urban development plans of Plovdiv and Rousse, illustrating their continuous expansion.



Plovdiv. 1867 Urban plan designed by Guillaume Lejan,  
Le Tour du monde, vol. 26, 1873



Plovdiv. 1942 Urban plan designed by  
prof. arch. Herman Jansen



Plovdiv. 1891 Urban plan designed by Arch. Josef Schnier



Plovdiv. 1908 Urban plan designed by arch. Meyer



Ruse. 1896 Urban pan designed by  
eng. Todor (Teodor) Tonev, arch. Franz Hasa



PLATE IV

Urban development plans of Varna, Bourgas and Stara Zagora



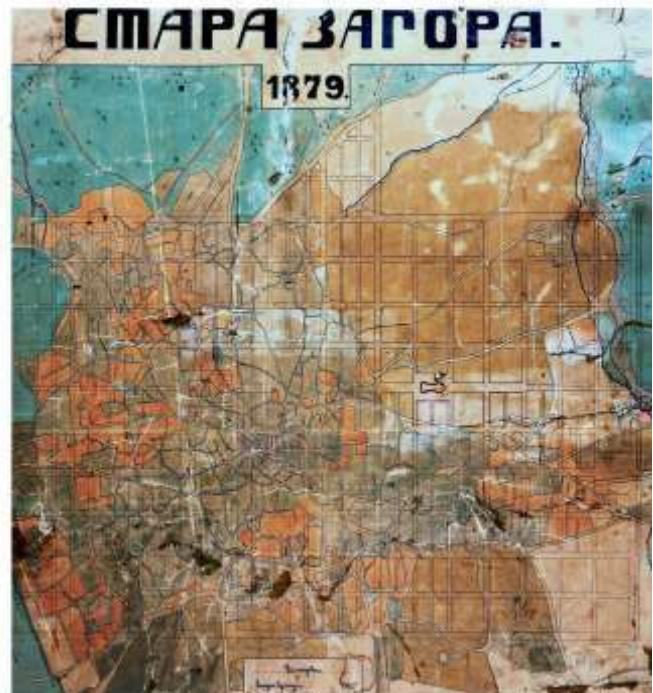
Varna, 1887 Urban plan designed by arch. Sava Dimitrievich, Krupka, Franz Hasa, Karl Trnka, Karakin Papazyan



Varna, 1923 Urban plan.



Bourgas, 1890 Urban plan designed by eng. Ananyan



Stara Zagora, 1879 Urban plan designed by arch. Lubor Bayer



This, however, created town-planning problems because of the small size of the plots planned for single-family detached homes, which now had to accommodate three- to eight-storey buildings that were too big for the surrounding environment. A new urban development crisis evolved, this time mainly related to volume and space, but also damaging the area of the green areas in the cities. The accumulation of urban collisions in Sofia led to the adoption of the Regulation for Construction of the Capital City in 1934, on the basis of which a General Urban Plan was adopted in 1938 (authored by German Prof. Arch. Adolf Mussmann) along with a new ordinance regulating its application. **(PLATE V)**

Initially, the building co-operative homes were designed by the association according to the requirements of each future co-owner. Construction was then assigned to a contractor selected by the General Meeting, but subsequently entrepreneurs started designing them with their own resources. Then they sold the flats as designed and in the construction process they met the requirements of their clients.<sup>5</sup>

Simplified were the procedures for the construction of buildings of flats for the needs of factory workers. They were mostly constructed after design by departmental design offices set up specifically to that purpose. They were built mostly in the outskirts of the cities, on cheap terrains, close to the factories they belonged to. **(PLATE VI)**

The greatest number of multi-family buildings of these two types sprung in Bulgaria in the periods 1921 - 1926 and 1928 - 1934. Not a few workers' dwellings were built after 1935, as well as in the 1940's and 1950's.<sup>6</sup>

There were also differences in the applied functional patterns. In the workers' dwellings, the corridor system was widely used: individual rooms, depending on their number, were located along a shorter or longer corridor.

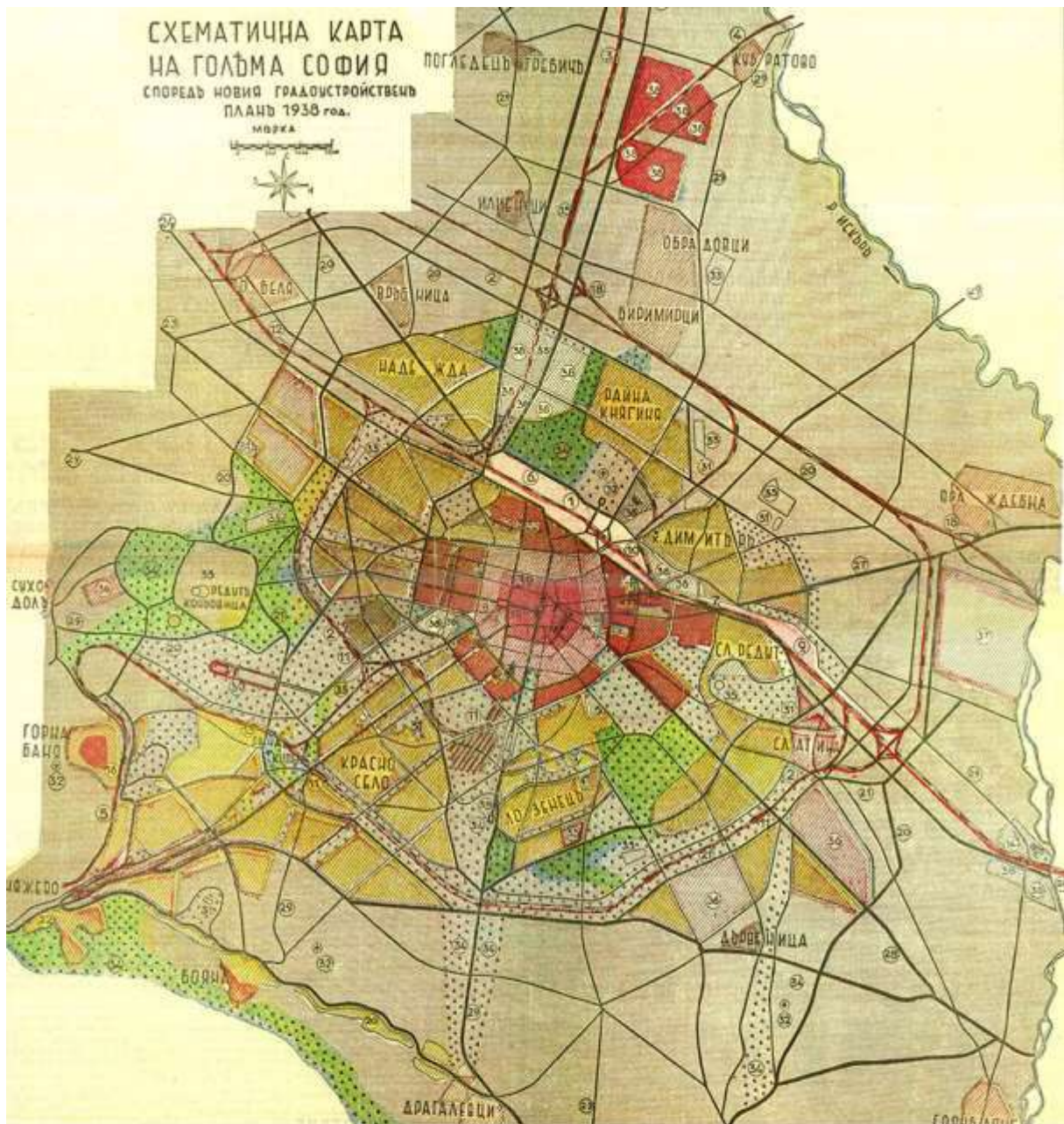
The requirements to the building co-operative homes were much higher and the flat layout design patterns changed fairly quickly. At first, the pattern was similar to the functionality of the single-family detached homes from the period up to the First World War, ie vestibule pattern with clearly zoned groups: **day-time** premises - a parlour and an attached extra premise designed for a study or a drawing room, **night-time** premises - two or more bedrooms, bathroom and possibly a closet, and a **housekeeping (economic)** part of the home - usually in the back of the building comprising a kitchen, a larder, a second bathroom and a room for the servants. Consequently, in addition to the vestibule pattern, flats were built in the so-called living-room pattern, where the vestibule was integrated with the parlour in a common premise already designated as a living room or a drawing room. The latter however, was transitory, usually to the night-time part of the flat. Both patterns provided for the existence of an entrance hall in front of the vestibule, or later in front of the living room, which had a direct connection to the housekeeping area. Often the latter had a separate entrance directly from the staircase, or even from a separate second servants' staircase. **(PLATE VII)**

However, it should be noted that the vestibule and the living-room functional systems existed in parallel for decades before the first of them was definitively abandoned.

As far as the architectural layout in the period between the two world wars goes, it should be noted that two mutually permeating stages can be outlined. Initially, buildings of a more pronounced eclectic appearance were constructed. The latter, however, was influenced by the developments all around Europe, including buildings from the first half of the mid-1920's and Art Deco-designed examples, like the very good architecture of the Musala building cooperative home in Sofia (1928 - 1930, architect Kosta Nikolov). **(PLATE VIII) (PLATE IX)**

Gradually, however, at the end of the decade, the architectural layout of the building co-operative homes began to outgrow the small decorative details and the influence of the classical modernism of the Bauhaus (*Hochschule für Bau und Gestaltung*), founded in 1919 in Weimar by architect Walter Gropius was evident. And this is perfectly understandable given that, according to a study published by BIAD, from the Liberation until 1935, 84 percent of the architects and 65 of the engineers in Bulgaria have completed their education in German-speaking countries.

PLATE V



Sofia. 1937 – 1938 Urban plan  
designed by  
prof. arch. Adolf Musman

1938. Signing the plan at  
Town Planning Directorate  
of Sofia Municipality.





PLATE VI

Workers' dwellings from the 30s and 40s of the twentieth century.



Pernik. The Workers' Homes of Pernik Mines, the late 30's



Mezdra. Workers' residential building of the Stanke Dimitrov factory, late 1940s



PLATE VII

Buildings from the 20s of the twentieth century, in eclectic styles.



Sofia, "Moskovska" Str. Building cooperative "St. Sofia",  
1923-1928, arch. Lazar Parashkevanov.



Sofia. Building cooperative "Globe".  
Beginning of the 1920s (architectural  
bureau Nachev-Fingov-Apostolov) with a  
built-in two-story building from 1912  
(arch. Nikola Lazarov).



Plovdiv. Buildings cooperative. Circa 1929



Sofia, the corner of "Knyaz Alexander Dondukov" Blvd  
and "Malko Tarnovo" Str. Eclectic. 1920s.



"Assenovets" Building cooperative, 1926



PLATE VIII

Buildings from the 20s of the twentieth century, in the style of Art Deco.



Sofia. A building in Art Deco style from the "Bezistena" complex against the courthouse (mid 20-s). Bul. "Vitosha" near the intersection with "Alabin" street. Circa 1930



Sofia. The corner of "Rakovski" Blvd. and "Iskar" Str. Building cooperative "Sun" in the style of Art Deco, 1929



Sofia. The corner of "Skobelev" Blvd. and "Kniaz Boris I" Str.



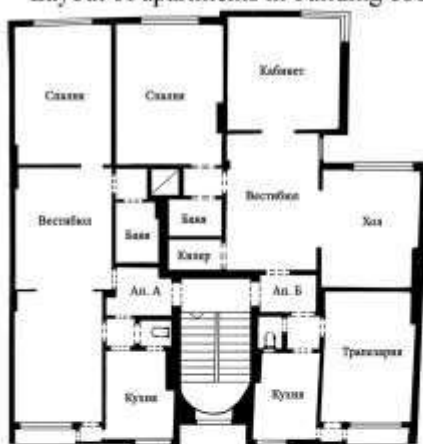
Plovdiv. Art Deco building cooperative, 1920s



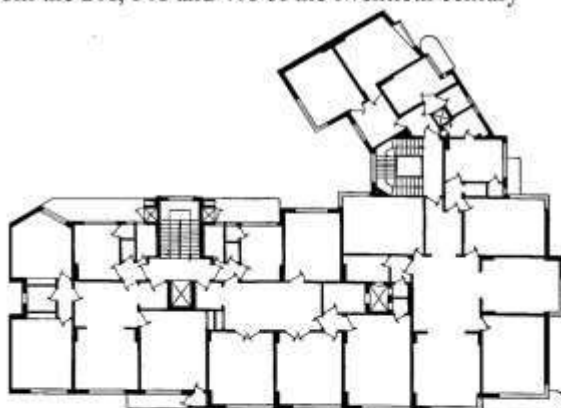
Sofia, the corner of "Serdika" Str. and "Triaditsa" Str. "Musala" building cooperative, 1925, Arch. Kosta Nikolov

PLATE IX

Layout of apartments in building cooperatives from the 20s, 30s and 40s of the twentieth century



Sofia, 19, "Ljuben Karavelov" Str.  
Building cooperative, 1940s,  
arch. Ivan Hranov.



Sofia, the corner of "Moskovska" Str. And "Malko Tarnovo" Str. Building cooperative "Karadzha", with passenger and kitchen elevators, 1828-1829.  
arch. Krastan Gechev and arch. Peter Karasimeonov.



Sofia, the corner of "Rakovski" Blvd and "Han Asparuh" Street. Building cooperative "Bakalov", 1934,  
arch. Slavi Angelov



Sofia, "Moskovska" Str.  
Building cooperative,  
apartment with servant entrance, 1937,  
arch. Victoria Angelova-Vinarova



Sofia, the corner of "Vasil Levski" Blvd. and "Graf Ignatiev" Blvd. Building cooperative, 1935.  
Arch. Ivan Vasilyov and arch. Dimitar Tsolov



Sofia, 8 "Kliment Ohridski" Str. Building cooperative, apartment with servant staircase, 1928-1929, arch. Georgi Ovcharov.



Of course, the transition to the new aesthetic concept went through a short phase of eclectic mixing with the established dogma, but yet the use of the European Art Deco style born in the early 1920's, had previously provided a ground for perception of certain views distinct from those of the old eclectic architecture at the end of the 19<sup>th</sup> and the first three decades of the 20<sup>th</sup> century. Indeed, Bauhaus's modern vision would appear in a fairly pure form at the very end of the 1920's, both in single-family detached home architecture and in the multi-family building co-operative homes. And it can be argued that by about 1946 - 1947 this would be the main architectural style to be used. **(PLATE X) (PLATE XI)**

One of the earliest examples of Modernism in Bulgaria is the building co-operative (co-operative chamber) Karadzhata, at 3 Moskovska St in Sofia (1928, architects Krastan Gechev and Petar Karasimeonov). The 1930's and the early 1940's will now be almost entirely dominated by the new modernist ideas in their pure form, applied mainly in the construction of multi-storey residential and public buildings, but also in some of the detached residential buildings. Good examples of this architectural style are several building co-operative homes on Vitosha Blvd in Sofia designed by architects Radoslav Radoslavov and Konstantin Dzhangozov built in 1937 - 1939; the building co-operative homes (late 1930's) at 12 Patriarh Evtimiy Blvd (1943, architect Vasko Vasilev), on the corner of Evlogi and Hristo Georgievi Blvd and Geneva St (1940's, architect Vasko Vassilev), to mention but a few in Sofia. There are also many examples of Modernism in the provincial cities, such as the Venedict Popov building cooperative home (today's Plazh, 1933, architect Stefan Venedict Popov) in Varna; the building co-operative homes at Aprilovska St in Gabrovo, 1 Nayden Gerov St (1934, architect Boyan Chinkov) in Plovdiv; on Aleko Bogoridi St in Burgas (mid-1930's); on Aprilovska St in Gabrovo (1930's); at the corner of Tsar Osvoboditel Blvd and Hadzhi Dimitar St in Sliven (1937, architect Zahari Iliev), to mention but a few.

In the mid-1930's, some building co-operative homes of a simpler appearance were built in the spirit of Modernism. They can be seen as precursors to the later constructivist manifestations of Functionalism. Such is, for example, the building co-operative home at the corner of Trapezitsa St and George Washington St (1935, architect Maria Sapareva) in Sofia, as well as many other examples from the late 1930's. These buildings lack the strip-like windows which were replaced by three- and two-winged windows. **(PLATE XII) (PLATE XIII)**

At the same time there will be a timid attempt towards architectonics in the spirit of the old Bulgarian architecture, developed in the spirit of either the Middle Ages or more often of the Bulgarian Revival Period. Examples of the first direction are several building co-operative brick-decorated homes the deed of architect Georgi Ovcharov (Shesti Septemvri St, 1928). The second direction is represented by the buildings of architect Veselin Yakovov, such as Mizia building co-operative homes on Ivan Vazov St, and another building that is not preserved, both from the period 1931 - 1933. Their appearance is characterized by broad eaves and small baroque pediments above a system of fa9ade avant - corps.

Brick decoration will appear in the late 1920's, but in the 1930's it will often be part of the large decoration forms of the otherwise constructively designed fa9ades. At that time Modernism would be quite common in the streets of the cities, now in a simpler and more impersonal form free of the detail typical of the pure style - pergolas supported by slender cylindrical columns, voluminous balconies, and tower-like accents. Fa9ades will often be finished with limestone slabs, or their plaster imitations.

PLATE X

Building Cooperatives of the 1920s and 1930s



Sofia, 3 "Moskovska" Str. Corner with "Malko Tarnovo" Str. Building cooperative "Karadzhata", 1928-1929, arch. Krastan Gechev and arch. Peter Karasimeonov.



Plovdiv, the corner of "Nikolay Kozhuharov" Str. and "Nayden Gerov" Str. Building cooperative, 1934 design by Arch. Boyan Chinkov



Varna, "Primorski" Blvd. Building cooperative "Venedig Popov" (Today's "Beach"), 1933, design by arch. Stefan Venedik Popov



Gabrovo, "General Radetski" Str. Building cooperative, 1930s



Sofia, 22 "Parchevitch" Str. corner with "Knyaz Boris I" Str. Building cooperative, 1930s.



Sofia, 38 "Vasil Levski" Blvd. Building cooperative, 1828-1829.



PLATE XI

Buildings Cooperative from the late 1930's and early 1940s



Sofia, 48, Vitosha Blvd. Building cooperative, 1937-1938, arch. Radoslav Radoslavov and arch. Konstantin Djangofov.



Sofia, 59, Vitosha Blvd. Building cooperative, 1938-1939, Arch. Radoslav Radoslavov and arch. Konstantin Djangofov.



Varna, 1 Knyaz Boris I Blvd. Building cooperative, 1938, Arch. Stefan Venedik Popov



Sofia, 17 San Stefano Str. Building cooperative, 1940, Arch. Angel Damianov



Sofia, 27, William Gladstone Str. Building cooperative, 1943 Arch. Vasko Vasilev



Sofia, 4 Oborishte Str. Building cooperative. Mid 40s of the 20th Century.



PLATE XII

Buildings cooperative with a simplified, constructivist vision, buildings with masonry and graphic decorations.



Sofia, the corner of "Trapezitsa" Street and "George Washington" Str. Building cooperative with a constructivist vision, 1935, arch. Maria Sapareva.



Sofia. Building cooperative with a constructivist vision, 1934.



Sofia. Building cooperative with a constructivist vision, mid 1940s



Sofia, "6-th September". Building cooperative with brick and sculpture decorations, 1928 arch. G. Ovcharov.



Sofia, "6-th September" Str. Building cooperative with brick decorations, 1929 arch. P. Tsvetkov and arch. T. Goranov.



(left) Sofia, "Karnigradska" Str. Building cooperative with ceramic decorations, 1930



(right) Sofia, "Evlogi and Hristo Georgiev" Blvd. Building cooperative with colorful decorations, 1930s.



PLATE XIII

Details from buildings cooperative. From the 1920s, 1930s and early 1940s.



Sofia, 51, "6-th September" Str. Building Cooperative with masonry decorations, 1929, fragment, design by arch. Pantalei Tsvetkov and arch. Theodor Goranov.



Sofia, 7 "William Gladstone" Str. Staircase of a building cooperative, early 1930s.



Sofia, "Georg W. Washington" Str. Staircase of a building cooperative, 1935



Sofia, 111 "Georgi Rakovski" Blvd. Stairs of building cooperative "Zadruga", 1938-1939, design by arch. Atanas Delibashev.



Sofia. 45 "General Gurko" Str. Entrance lobby and staircase of a building cooperative, 1939-1940



Sofia. 32 "Gladstone" Str. Staircase of building cooperative, 1943, design by arch. Vasko Vasilev



Sofia. 10 "Tsar Osvoboditel" Blvd. building cooperative with rooftop pool, 1939, design by arch. Vasko Vasilev

### 3. Multi-storey housing construction in the Stalin Period

After the occupation of Bulgaria by the Soviet army, Bauhaus's modernism will last 2 - 3 years (for example, the building co-operative home at 40 Vitosha Blvd (architect Yordanovich), or in fact until the time of the final taking of the country's rudder by the Communist Party and the nationalization in 1947 following the adoption of the new Dimitrov's Constitution. .  
**(PLATE XIV)**

In 1947, the Bulgarian Academy of Sciences and Arts (BANI) was abolished by unlawful and repressive laws and the Bulgarian Academy of Sciences (BAS) was set up after the Soviet model. Two years later, in 1949, the Institute of Urban Planning and Architecture (IGA) was established under the control of the party functionary, architect Alexander Obretenov (member of the Bulgarian Communist Party since 1931), one of the founders and conductors of the ideas of Marxist-Leninist aesthetics in Bulgaria. Obretenov set up the new structure of the Committee for Science, Art and Culture, which started enforcing the line of total Stalinist ideological and administrative control, centralized management and mass production of state cultural events, defined as 'folk culture' and work on the 'cultural front' according to Communist terminology.<sup>7</sup> His work *The Dialectical Materialism as a Worldview of Soviet Art* (1947) marked the beginning of the Stalinist line in arts, and architecture in particular. Categorical denial of the trends in architecture of the 'Western Capitalist world' followed in *Bourgeois and Socialist Culture* (1949), to finally evolve into concreteness in *Building a Nationalist Socialist Architecture* (1951), developed on Stalin's ideas that culture, hence architecture, should be Socialist in content, but also national in form. Evident was particular impact of the speech of Academician Todor Pavlov, presented in December 1950, at the opening of an IGA exhibition of architectural footprints of Koprivshitsa houses, when the prominent communist philosopher called for complying with this line as 'Soviet architects' were already doing.<sup>8</sup> The Academician openly instructed, **'...a fundamental and actual task is: the Socialist content of our culture, architecture respectively, to be given a national form.'** But since Stalin had indicated that **'the future merging of national cultures into a common culture with one common language'** was predetermined, then let there be **'a bloom of the cultures - national in form and socialist in content, under the dictatorship of the proletariat in one country towards their merging into a common socialist culture'**, until this universal socialist future becomes reality. Moreover, Pavlov forbade that **'as the future historian of our art has the right to consider the symbolic, futuristic and similar works of our pre-September [9<sup>th</sup> September 1944] artists for classical national Bulgarian art, so today's historian is obliged to distinguish between the architectural heritage of the past, which was the result of the then snobbism of various gaffers and wealthy merchants...'** Finally, he would heavily emphasize that **'our modern culture, hence architecture, is unthinkable without the study and use of the great experience of the Soviet culture - Socialist in its content, but national in its form.'**<sup>9</sup>

In architecture there was also direct intervention and control over the scientific production of the newly established scientific institute of the Bulgarian Academy of Sciences. In his extensive annual report for the first full year of operation (1950), its director Prof. Arch. Lyuben Tonev would have to write: **'... architecture must reflect concretely and eloquently the existence and the beauty of the Socialist era through works that are Socialist in content and national in form.'**<sup>10</sup> Gradually, Stalin's classicism penetrated in Bulgaria (aka Stalin's eclectics, or Stalin's empire style; in Russia the residential buildings in this style were referred to as *Stdmk*), in parallel with the denial of Bauhaus's ideas, by means of repressive measures as well. It goes to the fact that for a building which had been designed with only several three-winged windows on the main facade, the designer could have been accused of introducing 'prowestern influences' and suffer administrative problems.



PLATE XIV

Buildings Cooperative from 1943-1948.



Sofia, the corner of "Evl. And Hr. Georgievi" Blvd. and "Geneva" Str. Building Cooperative, 1943, design by arch. Vasko Vasilev.



Sofia, 40 "Vitosha" Blvd. Building Cooperative, 1946, Design by arch. Vasil Jordanovich.



Sofia, 63 "Vitosha" Blvd. Building cooperative, 1946, design by arch. Vasko Василев



Sofia, 53 "Graf Ignatiev" str. Residential building, 1948-1950, design by arch. Angel Damianov and arch. Maria Angelova.

The forcefully imposed new aesthetic system introduced by the Soviet Union abounded in voluminous constructions with small windows, balconies with balustrades and other classical facade decorations. In all Bulgarian towns preserved are buildings of this period, the largest ones being in the capital city and the bigger settlements. Out of the small towns only in Dimitrovgrad, founded in 1947 as an experimental project and developed in line with these aesthetic criteria in the period 1948 - 1950, there is large number of Stalinist classicist buildings. **(PLATE XV) (PLATE XVI)**

In parallel with this construction that was alien to Bulgarian culture, in line with Stalin's slogan 'National in form, Socialist in content', a new attempt was made to create an architectural vision that employed the principles of our historical heritage. This attempt was already more wide-scaled than the previous two: the National Romanism from the early 20<sup>th</sup> century and that from the late 1930's and the early 1940's. This time it started with employing the classical forms that were created around the country. In most cases, however, its manifestations were in the field of public works: stations, holiday homes, hotels, etc., although there were also residential buildings, including multifamily examples. **(PLATE XVII)**

By 1955, not a few multi-family residential buildings were built, forming whole micro-regions on the open terrains in the outskirts of the bigger towns and cities. In Sofia these are the zones at the end of Stamboliyski Blvd., the first group of block of flats of Lagera residential complex, a group of blocks of flats in Lozenets Quarter, the industrial residential groups in the vicinity of Iskar railway station and the large workers' residential quarter in Gorna Banya meeting the needs of the Bulgaria Factory. Construction of residential blocks of flats went further towards the downtown along the main boulevards Stamboliyski, Rakovski, Dondukov, Maria Louisa, Vitosha, Skobelev, to mention but a few.

The same process took place in other big cities in Bulgaria, and also in the towns, as well as in the newly-born settlements for the needs of industrial development: Dimitrovgrad, Rudozem, Madan, Pernik-Iztok/East, Devnya and others.

It should be noted that initially there was a rather lively argument between supporters and opponents of the idea of a quick and smooth absorption of the periphery urban terrains to build whole new residential neighbourhoods. Opponents stood on the idea of rebuilding the urban environment in a new 'socialist' way, eliminating all the 'decadent' buildings related to the Capitalist pre-war past so as to create, as proclaimed, a living environment worthy to meet the needs of the 'working class.' They stood for the elaboration of entirely new urban plans of the cities and villages developed on this ideological basis. They disputed even the excellent plan of Sofia by the 'German' Professor Mussmann and opened a wide competition for a new plan that was won by urbanist Prof. Lyuben Tonev. In spite of all the objections, in compliance with this plan entire zones of the downtown were demolished. This affected large portions of the streets Targovska, Dondukov, Maria Louisa and Alexander I Square (where some damages after the Anglo-American bomb raids were still seen). They used to abound in beautiful high-value buildings to be now replaced by the Largo ensemble dominated by the Party House. It was not until 1956, when the projects for new large residential complexes in Sofia were adopted thus putting an end of the dispute.



PLATE XV



Sofia, Rousse, Dimitrovgrad, Pernik, Plovdiv.  
Apartment buildings from the Stalinist period in  
architecture. 1947-1957



Sofia, 59 "Skobelev" Blvd. Plan of the corridor  
type apartments from a residential block.  
1952-1954



PLATE XVI

Dimitrovgrad, the first socialist city of Bulgaria.



Dimitrovgrad. Axonometry of the first urban plan, 1947, design by arch. Peter Tashev and arch. Lyuben Tonev and views of urban spaces on archival photos.



PLATE XVII

Architecture from 1947-1956 with traditional features in the design



Velingrad. Holiday home of trade unions  
(Today the Palace Hotel), 1949-1951



Varna, Palace Evksinograd.  
Ferdinand's House, approx. 1952-1954



Koprivshitsa, railway station, 1952



Bratsigovo. Chitalishte "Vasil Petleshkov", 1954-1956,  
arch. Zhelyazko Stoykov and arch. Vladimir Rangelov



Varna, resort Sunny Beach. The first hotel of  
Balkantourist, 1956-1957, arch. Georgi Ganev.

#### 4. Multi-storey housing construction in the period 1956 – 2013

After the April Plenum of the Central Committee of the Communist Party of 1956, when Todor Zhivkov became head of the party and gradually took the rudder of the state, in parallel with the political changes, architecture was partly released from Soviet ideology. But shortly before that and coincidentally right after the new Stalin's party guidelines in the Soviet Union of 1949 - 1950, Obretenov started adapting his views to them and probably to the statements of the Marxist - Leninist authority Todor Pavlov, he would write his summarizing work *On Building a National-in- Form Socialist Architecture* (1951). This was an attempt to impose a new approach to design as an overall lesson from Revival Period tradition. This was achieved specifically through the creation of some elements of Bulgarian historic architecture introduced in the form of changes based on heavy Stalin classicism as opposed to international Modernist trends. Later Tonev himself, too, in his analytical article *Ten Years of New Bulgarian Architecture* (1955), even though mentioning all the ideological cliches of the time, garnished with quotes by Lenin, Stalin, George Dimitrov, Vasil Kolarov, Valko Chervenkov, etc, would dare to criticize the implementation of Stalinist classicism in Bulgaria alongside listing some achievements. Criticism, of course, was already possible given the fact that after the death of Stalin in 1953, party criticism against decoration started in the USSR. In the same 1955 it would produce the regulation of the CP Central Committee and the Council of Ministers of the USSR on the elimination of redundancy in design and construction (*Об устранении излишеств в проектировании и строительстве*) which today is considered to be the end of the epoch of the monumental Stalinist classicism. So Tonev would safely write about **'mistakes in distribution; architecture has not yet acquired the image that the Bulgarian socialist home should have; the economic indicators and the level of the construction equipment are not yet the desired ones'**. He would also account for **'failures also in the search for the correct architectural layout of our home, going far as incorrect and uncritical 'transformation of classics', and decoration with 'classical' cornices, consoles, balconies, balustrades, etc.'** In his paper he also mentioned what was urgently needed in the development of architecture, constructions and construction technologies. In particular he discussed the **'storey number and the density of housing construction... the distribution system, a number of installation- and other household improvements... the nomenclature and serial design, the common and stream-like non-stop building process...'**<sup>11</sup> Further, in line with the national architectural rules, he would also criticize the fact that the designers **'have not enough learned lessons from the architecture, composition techniques and construction of the folk master of the past'** <sup>12</sup>

Therefore, this means that following the events in the Soviet Union, there was already some sort of attitude to change among the architectural guild, and when these changes were politically outlined in the plenum's decisions, their introduction started.

The changes were initially related to revisions of Stalinist classicism, which, without changing the heavy constructions, led to a sharp reduction in the amount of facade sculptural details and to the increase of the size of the window openings. The three-winged window was vindicated, even in Dimitrovgrad. The construction process, although waning, of buildings with national features in their architectural design continued.

But when in the Soviet Union there was a return to the Functionalism of Bauhaus and the theoretical ideas of Jacob Johannes Oud, transformed into the International style of the Modernist system, changes quite rapidly took place in Bulgaria where architects had in parallel adopted and were already working hard in this direction since the late 1920's until 1947. Especially distinct therein are Le Corbusier's five points of modern architecture: use of distinct geometric shapes, use of large solid boards of uniform material, flat roofs - usable, if possible, glazing of the facades in horizontal window strips and open-space design of the ground plan without supporting and partitioning walls to allow its use for public functions. Le Corbusier's



definitions grew very popular, **'The home is a machine for living'**, as well as Luis Sullivan's philosophy of style, **'Form is determined by function'**. Naturally, also focus was placed on the ideas of Constructivism as a significant Soviet phenomenon being a trend within Functionalism, building on the principles of Le Corbusier. Moreover, in 1922 Alexey Ghana would define Constructivism as a task of achieving a **'communist expression of material values'** and would add that **'tectonics, construction and texture are mobilizing material elements of industrial culture'**, thus defining the new culture of the Soviet Union as industrial in its essence.

Bulgarian Communist nomenclature stood not only behind the idea of socialist architecture but also behind industrialization as the activity of the leading 'working class', in its view, in line with the postulates of Marxism-Leninism. So, from a conceptual point of view, the transition to the new architectural aesthetics was generally a conflict-free process. And for a lot of Bulgarian architects this was fully-natural return in professional terms to the broken creative line of art before 1947.

Since the first year of IGA's existence, the question of the functional patterns in housing construction has been put to the fore. It was clear that the functional decisions in the building co-operative homes were too wasteful and differentiated servants' passes and premises, while the homes of the poorer people were quite primitive and suffered from hygienic deficiency. Attempts continued, with the initial abandonment of vestibules and transition to corridor-functional patterns providing direct access to each of the premises.

In 1956, the results were reported, and based on the reflections of Obretenov and Tonev and other authorities in the professional circles, it was proceeded to the creation of patterns that took into account the tradition of the 'folk house'. Paradoxically, as such was considered not the popular house of the poorer population with string-like and enfilade plans but the parlour- type homes of the richest gaffers, and the day-time living space was regarded as the centre of the composition.

Therefore, patterns with the living room as centre of the composition were developed (Sofproject), drawing on the experience of similar solutions created for building co-operative homes in the period 1935 - 1943. When designing Lenin residential complex (modern-day Yavorov complex), architect Vasil Valchanov will also apply the pattern of non-transitional living rooms, which would become more and more preferred in time.<sup>13</sup> **(PLATE XVIII)**

The first already large-scale residential complexes of Sofia: Deveti Septemvri (today's Slavia residential complex), Vladimir Zaimov, Lenin (today's Upper Yavorov), would follow these same patterns. The trend continued in the residential complexes designed in the period 1958 - 1959: Hipodruma, Zapaden Park, and Iztok.<sup>14</sup> Trends in the new housing construction developed and tested in Sofia would be later applied across the country. Suburban territories started being urbanized to host residential complexes as per the local needs.

Urban planning of these new residential agglomerations was in line with the modern concept of creating vast green spaces between the built-up terrains thus providing green areas, children's and sports grounds, and overall, more air and light. Over the next decades, however, further construction would be carried out in the already built-up territories and the newly designed complexes from the 1970's and 1980's would be in line with the changes in the regulations providing for more thick build-up development of the territory relative to the height of the residential blocks of flats. From the end of the 1950's, a multi-storey construction pattern will be sought instead of the 3- to 6-storey blocks of flats built up until then. In the 1960's, the number of floors grew up to high-rising 9- to 12- storey buildings, like the ones in the expansions of the residential complexes of Zapaden Park, Zaimov and Iztok, and even on the periphery of the low-rise well-developed Lenin (Yavorov) complex. **(PLATE XIX) (PLATE XX)**

PLATE XVIII



Sofia. Residential complex "Yavorov", formerly residential complex "Lenin", design by arch. Vasil Valchanov



Sofia. Residential complex "Yavorov", residential building of a mass design, design by arch. Vasil Valchanov



Sofia. Residential complex "Yavorov", formerly residential complex "Lenin", design by arch. Vasil Valchanov, late 1960s.



Sofia. Residential complex "Yavorov", coffee shop "The fairy", design by Arch. Vasil Valchan-



Construction of the residential complex "Lenin".  
The late 50s

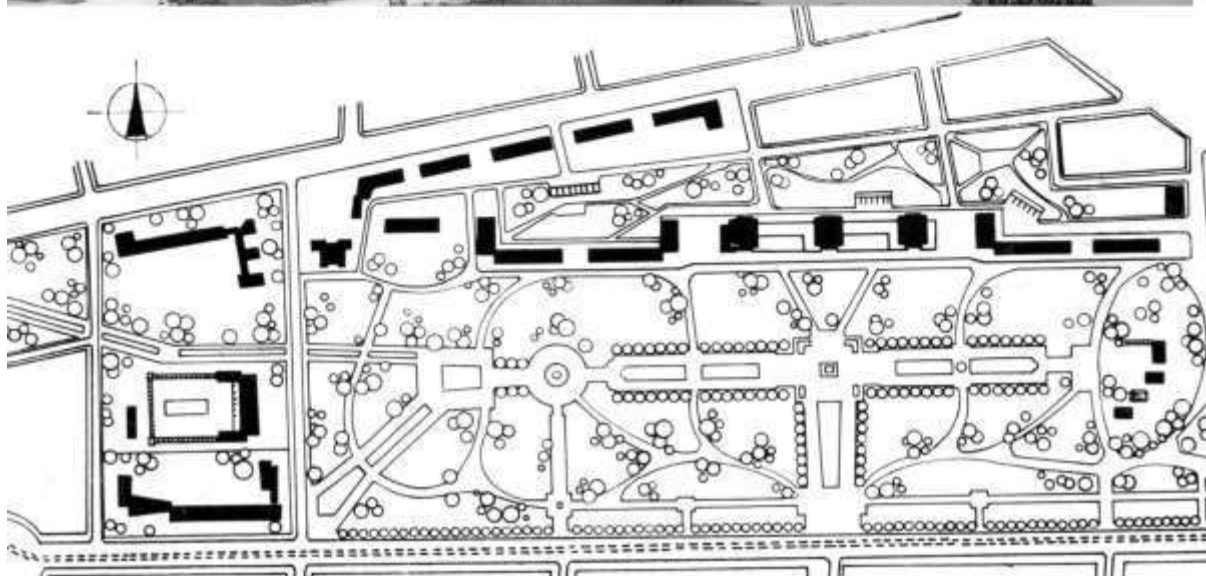


PLATE XIX

Early residential complexes in Sofia, 1950s - 1960s



Residential Complex "Western Park", 1956-1960,  
arch. Kiril Bosev and arch. Ivan Nikolov. Urban plan and views



Residential Complex "General Vladimir Zaimov", 1956-1959, Urban Plan, Buildings and Views

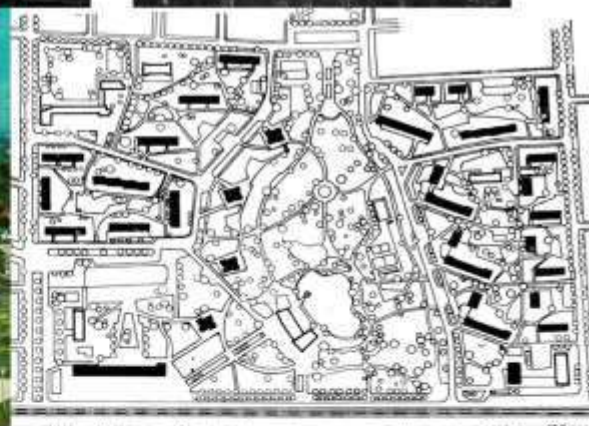


PLATE XX

Early residential complexes in Sofia, 1950s - 1960s



Sofia. Residential Complex "East", 1958-59, design by arch. Pancho Malezanov, Arch. Stefan Stefanov, Arch. Boyan Bondikov. Urban plan and views



Sofia. "Hipodroma" residential complex, 1958-1960, arch. Peter Tashev and arch. Zdravko Kolev (monolith-brick), the 80s and late 90s (monolithic).



Urban plan, distribution of residential buildings (architect Zdravko Kolev (left), architect Stefan Badjov (right) and views.

The new standards for spacing between buildings and their number of floors were complied with in design and construction until the 1990's all around the country, with the number of significant residential complexes in the cities growing up to several.

Since the beginning of the 1990's, the old built-up terrains were additionally and compactly built-up with new blocks of flats, and new small micro-regions of several buildings emerged, increasingly forming enclosed, guarded and fully maintained structures.

Functionally, the flats in the blocks of flats in the 1970's and 1980's were designed following the pattern of direct accessibility to the living room and the bedroom in the smaller ones, and in some cases with an indirect second bedroom in the big ones. Separate kitchens with a dining area were foreseen. Predominantly, two-room (one-bedroom) flats were built; fewer were the three-room (two-bedroom) flats, with the living room unlike the kitchen being considered as a room. Very few were four-room (three-bedroom) flats that served the needs of large families.

In the stylistics of Functionalism, flat roofs dominated in two variants - so-called warm and cold roofs: with one roof slab or with two roof slabs forming a low (about 80-cm high) space ventilated on both sides through ventilation openings. Low heating-energy prices predetermined the lack of thermal insulation, while 'sandwich' solutions by incorporating styrofoam which quickly sublimated and degraded into dust, were applied only on panel fa9ades. The fa9ades in the prevailing number of cases were plastered and the outer wooden joinery was mostly of two joined wings, subsequently mounted, mainly of coniferous wood imported from Komi ASSR and yielded by Bulgarians under a contract with the USSR effective from 1967 to 1994. The signing of this contract triggered large-scale construction in the country, which between 1950 and 1970 resulted in double growth of consumption of construction and industrial wood. However, the rapid growth of the coniferous species in the humid climate of Siberia made the wood friable and lacking joinery qualities. That is why the joinery of Komi wood - ie in all LPS buildings (large-panel system building, Plattenbau) - has long been amortized. Furthermore, the double joined wings did not allow good sealing.

In the 1960's, the construction of new residential quarters acquired large scale. Sofia residential complex Mladost-1 is an exemplar case. Originally called the Southwestern region, it was the subject of a large competition in 1964 won by the team of Architect Bogdan Tomalevski. A total of 1657 hectares were planned for construction, representing 10 percent of Sofia territory, divided into 5 micro-regions, each of about 7,500 inhabitants. Construction began immediately after the completion of planning. In the 1970's Mladost-2 and Mladost-3 quarters were built; in 1980 the plan of Mladost- 4 was approved, while the one of Mladost- 1A was approved at the end of 1981. So far, 440 large blocks of 16,494 flats are built there. Today, this massive construction project provides housing for about 120,000 people.

In the same period, similar large construction projects were implemented elsewhere, not only in Sofia (for example, Lyulin residential complex, planned in 1969, and constructed in 1973, about 121,000 inhabitants) but also in all district towns. These include Trakia residential complex in Plovdiv (1973 - 1976) comprising 256 blocks of flats distributed in 13 microregions, Meden Rudnik residential complex in Burgas, Vladislav Varnenchik residential complex in Varna (also dubbed Vladislavovo), Strogazia residential complex in Pleven, Zheleznik residential complex in Stara Zagora, to mention but a few. **(PLATE XX1) (PLATE XXII)**

It should be noted that in each of the larger cities the number of residential complexes reached 5- 8, and the population inhabiting the respective areas with large-scale blocks of flats made about 50 to even 70 percent of the total population.

In the 1970's and 1980's, some examples of monolithic construction with outstanding architectural qualities, designed for the ruling nomenclature, were also built in the country.



PLATE XXI

Residential complexes throughout the country, 1970s - 1980s



Kyustendil. Residential complex. Present day.



Pleven. Residential complex. 1976



Bourgas. Residential complex "Lazur" ("Tolbuhin") and the so-called "Cucumber".  
Circa 1970s



Stara Zagora. "Trite Chuchura" Residential  
Complex. Circa 1975



Haskovo. Residential complex "Orpheus". 1970s



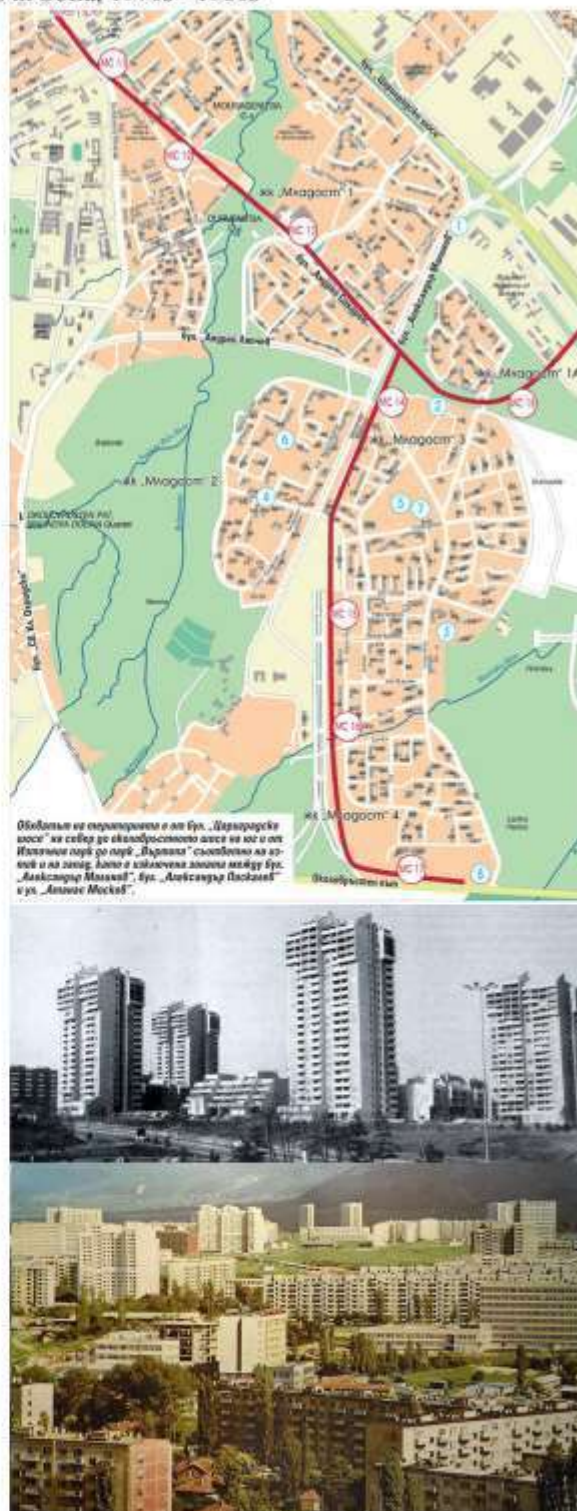
PLATE XXII

Large residential complexes in Sofia, 1970s - 1980s



Sofia. "Lyulin" residential complex.  
Construction lasted from 1971 until 1986.  
Model of the urban plan, 1966-1969,  
design by arch. Kiril Bochkov and  
Arch. Nedelcho Paskalev.

Views from 1972 and present day



Sofia. "Mladost" residential complex.  
Plan of present day neighborhoods with  
the metropolitan trails.

Partial views of the 1980s (so-called "blocks for  
young families") and present day.



PLATE XXI II

Brick-monoolithic apartment blocks from the 1970s and 1980s, designed to improve the massive residential architecture in Sofia and Kardzhali



Sofia. The corner of "San Stefano" Str. and "V. Tarnovo" Str



Sofia. "Jolio Curie" Str.  
Diplomatic apartment blocks



Sofia. "Sterju Atanasov Str"  
Residential complex "Izgrev"



Sofia. 10 "Veliko Tarnovo" Str.



Sofia. Group of experimental apartment blocks in  
residential complex "Motopista", 1987-2000



кв. Лозенец, ул. Кораб планина,  
т.нар. „Блок на Добри Джуров“



Kardzhali. Apartment terraced building, 1984-1986. Design by  
arch. Oleg Nikolov. Winner of the Grand award of of Interahr-1987





PLATE XX IV

Apartment type residential buildings throughout the country, 1990's - 2015



Sofia, "Trakia" str, 1996.  
Building with two attic floors



Sofia, 90s. Building with  
two attic floors



Bourgas, 90s



Sofia, Motopista,  
2005



Sofia, Residential complex  
"Iztok", 2006 year



Pleven, "Vasil Aprilov" str,  
2007



Pomorie, 2007



Sofia, "Bulgaria" Blvd.  
2008



Sofia, residential building,  
2016, floor plan



Sofia, residential building, 2013, floor plan



Novi han, residential building,  
2013, floor plan



Bansko, apartment hotel,  
2008, floor plan



Sofia, residential building, 2008, floor plan



Not a few of them were stone-clad. Towards the end of the 1980's, some experimental apartment buildings were also made in the search of new visions in residential architecture. The most significant of them include the experimental group in Motopista quarter and the terraced building of architect Oleg Nikolov in Kardzhali, winner of the 1987 Interarch Grand Prix. **(PLATE XXXTTT)**

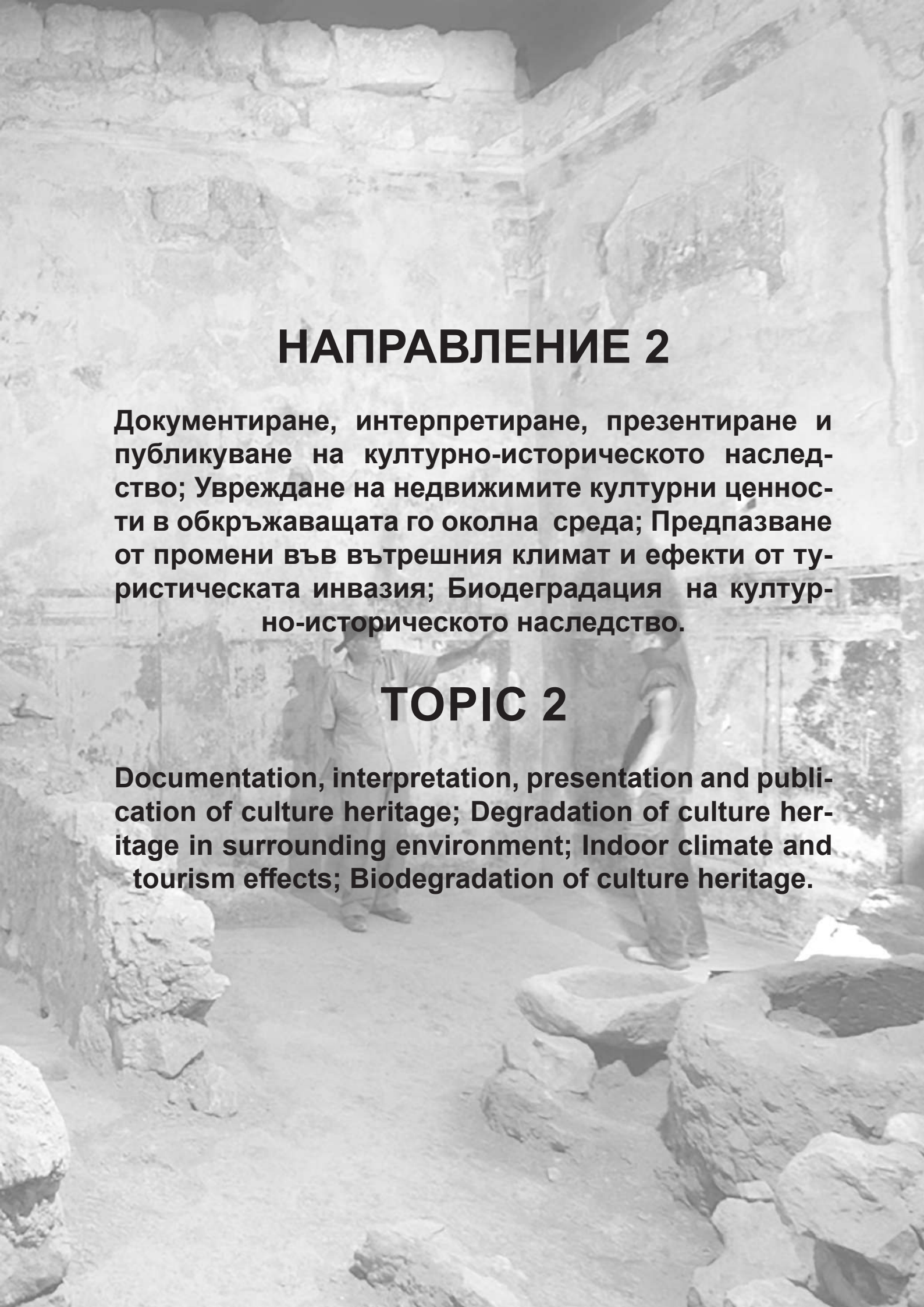
After 1990, there followed a period of construction of predominantly small-dimensional one- and two-room (one-bedroom) flats, where kitchens were replaced by a kitchenette, or more often they were designed as being part of the drawing-rooms. The use of the attic spaces, which initially accommodated two floors, led to the appearance of larger maisonette examples. After the amendments to the Spatial Planning Act, when the elevation over the cornice height up to the ridge height was defined to 4.50 m, the number of the attic floors was reduced to a maximum of one in the cases of height below 1.50 m between the slab and the cornice. After about 2005, the demand for small-size dwellings gradually dropped and an already larger number of three- (two-bedroom) and four- (three-bedroom) room flats were built, while the maisonettes disappeared almost entirely from the construction programmes.

During this period which is the last one in the study, the large-panel system building was discontinued as well as all related technologies of prefabricated construction works. The technologies for tunnel-type monolithic on-site cast concrete structures were also abandoned while some other mechanized systems such as slip forming, climbing formwork, etc, were rarely used. **(PLATE XXIV)**

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- [14] Пак там



The background image is a grayscale photograph of an ancient stone structure, possibly a ruin or a well. Two people are standing inside the structure, one pointing towards the wall. The structure is made of large, rough-hewn stones and has a circular opening at the top. The overall tone is historical and archaeological.

## **НАПРАВЛЕНИЕ 2**

**Документиране, интерпретиране, презентирание 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; Биodeградация 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.**





## ДВЕ ИЗГУБЕНИ СГРАДИ ОТ КРАЯ НА XIX В. И ТЯХНОТО ИСТОРИЧЕСКО МИНАЛО

Благовеста Иванова<sup>1</sup>

*Висше строително училище „Любен Каравелов“*

### **TWO LOST BUILDINGS OF THE END OF THE XIX<sup>th</sup> C. AND THEIR HISTORICAL PAST**

#### **Abstract:**

*The report presents an urban, architectural and cultural analysis of two buildings in the centre of Sofia and their neighbours.*

*The buildings of Petar Kanev Enterprise, in which the Drawing School in Sofia was located from the end of the 19<sup>th</sup> century until about 1907 are examined. Since 1910 they have been hired by Trapko Vassilev - a young and resourceful Bulgarian from Kostur region. In them he placed the First art gallery in Sofia, called the Permanent Art Gallery.*

*The aim of the study is to reconstruct the two buildings of the Trapkov Gallery and the adjacent buildings, located between Aksakov, Shesti Septemvri, Slavyanska and Dobrudja streets in terms of the urban planning and cultural aspects.*

*The personalities of Trapko Vassilev and his partner Petar Morozov are presented. The changes in the urban planning in the centre of Sofia are compared to the Ottoman urban planning. The appearance and development of the new districts, the state institutions located in them and the place of the Permanent Art Gallery in this part of the capital are analysed. Important cultural toposes, among which are public spaces and gathering places for intellectuals and bohemians are analysed.*

*The historical and cultural reconstruction resurrects details from the lost past of the memory of Sofia. The handris (handmade drawing) from 1894 was compared with the one from 1945. Thanks to them the changes of the housing construction in the district is established.*

*The study has a contribution to the development of the centre of Sofia in the period from the Liberation to 1945.*

**Key words:** *Permanent art gallery, Sofia's urban development after Liberation (1879), Trapko Vassilev, Petar Morozov, Drawing School*

#### **1. Увод**

През септември 1910 г. на картата на София се появява първата частна художествена галерия. Тя се намира в една неголяма сграда в зелените площи между улиците „Аксаков“ и „6 септември“ в развиващия се център на младата столица.

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Инициаторът на създаването ѝ Тръпко Василев открива нова страница в културния живот на града. С неуморната и упоритата си работа той се превръща в първия художествен предприемач и арт мениджър, а галерията става притегателен център за художниците, купувачите и любителите на изкуството.

Тръпко нарича своята галерия „Постоянна художествена галерия“. Известна още като „Тръпковата галерия“ и галерия „Аксаков“ тя затваря врати само по време на Първата световна война и през 30-те г. на XX в. при строителството на новите сгради на двете улици. Галерията прекратява своята дейност през 1944 г. Малко по-късно там се оформя ателие по скулптура за студентите от Художествената академия, така както е било преди Тръпко да наеме сградата.

Ще разгледаме двете сгради на „Предприятие Петър Кънев“, които за наети от Тръпко Василев и Петър Морозов за художествена галерия, а също така и останалите в квартала, разположени между улиците „Аксаков“, „6 септември“, „Славянска“ и „Добруджа“. Подобно изследване ще допринесе за възкресяване на паметта на Тръпковата галерия, за историята на сградите на Рисувалното училище и за реконструкцията на знаковия за съвременна София квартал.

## **2. Тръпко Василев и Петър Морозов. Градски исторически контекст.**

Желанието за работа и създаване на нови форми в културния живот на младите хора в началото на второто десетилетие на XX в. обединява Тръпко Василев и Петър Морозов. През 1910 г. те наемат сградата на „Предприятие Петър Кънев“ за художествена галерия. В нея до 1907 г. се помещава Рисувалното училище, преди да се премести в новата сграда на ул. „Шипка“. Причините за това са нейното добро осветление, както ще видим по-долу.



**Фиг. 1.** Портретна фотография на Тръпко Василев. Национална библиотека – Български исторически архив.



**Фиг. 2.** Петър Морозов. Автопортрет, 1903 г. Национална галерия.

Тръпко Василев е родом от село Личища до Костурското езеро. Роден през 1876 г. в семейството на Васил и Неделя Хамбарджиеви. Той посещава гръцко училище и от малък рисува с въглен по стените и оградите в родното си село, както



си спомня години по-късно. Останал полусирак и осъзнал себе си като човек, който към 11-14 годишна възраст може сам да изкарва прехраната си, той заминава за столицата на Османската империя Истанбул. Наема се на работа в млекарницата на дядо Шальо – човек от неговото землячество. Първоначално мие съдове, а още малко поотраснал започва да разнася гюмовете с млякото. Там го открива истанбулският художник от левантински произход Жавели, който взема момчето за свой помощник в изписването на интериори. По това време интериорната живопис със светски сюжети набира сили. Модната салонна живопис заема таваните и стените на сградите, които са украсени със стенописи с пейзажи, екзотични птици и растения. Някои от тях са оформени с постна боя с валяк, подобно на скъпите копринени тапети, които средното съсловие не може да си позволи.

Малкият Тръпко е помощник и изучава занаята – не само технологията, но и модерните за времето начини на оформяне на интериорите. През 1891 г. под въздействието на негов приятел от Загоричене, който по това време вече е в България, той решава на напусне Истанбул. Уговорките на неговия патрон и закрилник да остане в столицата на Османската империя, както и предложението за брак с една от неговите дъщери не бланзят младежа и той, вече пълнолетен получава като българин паспорт за отечеството.

Тръпко Василев пристига на пристанището в Бургас и се установява отново при негови земляци. В продължение на няколко години работи в Бургас, Ямбол и Стара Загора, където доказва своето майсторство в бояджийския занаят по начина, който вече е изучил. През 1898 г. воден от силното си желание да учи живопис той пристига в София. Установява се в евтиния хотел „Стара Загора“ на улиците „Аспарух“ и „Ангел Кънчев“<sup>2</sup>. Не след дълго попада на работа при влиятелни държавни мъже – министъра на образоването Димитър Пешев и бившия министър-председател Васил Радославов.

През есента на 1901 г. Тръпко вече е студент във втория випуск на Рисувалното училище. Житейските трудности не му позволяват да се справя с обучението и през 1903 г. той напуска училището. Четири години по-късно организира своята първа самостоятелна изложба в сградата на търговеца Тодор Балабанов на бул. „Цар Освободител“ и ул. „Раковски“<sup>3</sup>. Това е сградата, в която по-късно един българин от Македония Коста Стоянов, живял и лутал се в Америка, създава, по-късно превърналото се в култово, кафене „Цар Освободител“. По времето, когато Тръпко показва своите творби, помещението, което той наема е кръчма<sup>4</sup>.

Изложбата на Тръпко Василев е една от първите самостоятелни изложби в София. Тя вероятно е и една от първите, които са организирани извън държавна институция, тъй като е известно, че Иван Димитров и Димитър Добрович организират своите самостоятелни изложби в сградата на Народното събрание. Една година преди него бъдещият му съдружник Петър Морозов също организира самостоятелната си изложба пред софийската публика.

Изложбени зали по това време няма. Ето защо Тръпко Василев решава да създаде такава. За целта той кани за съдружник неговия състудент художника Петър Морозов.<sup>5</sup>

Петър Морозов (1880-1951) е син на воина от руската армия Петр Морозов, който след края на Руско-турската война се установява в Русе. Той завършва Рисувалното училище през 1904 г. и заминава да специализира графика в Париж.

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<sup>2</sup> Казасов, Д. Някогашните наши градове., изд. Отечествен фронт, С., 1975, с. 188.

<sup>3</sup> Константинов, К. Път през годините. С.1966, Български писател, с. 369.

<sup>4</sup> София XV-XX век. Спомени, впечатления, чужди пътеписи. съст. Анастасова, А., И. Донева и др. С., ОКИ „Столична библиотека“, 2009, с. 70 (спомен на Н. Танев).

<sup>5</sup> Започва да учи преди Тръпко Василев, през 1897 и завършва през 1905 г.

Завърнал се от там се запознава с частните художествени галерии и е наясно с начините на организиране на пространствата за експониране.

През 1910 г. двамата младежи наемат сградата или сградите на „Предприятие Петър Кънев“ и работят съвместно съвсем за кратко, тъй като Морозов се оттегля за да работи над своята специализация. Три години по-рано в тях се помещава Рисувалното училище, което преди построяването на проектираната от архитект Александър Смирнов сграда се разполага в частни помещения на няколко места в София.

В края на XIX в. повече от училищата са заемали частни сгради, които по план са тесни дълги паянтови съоръжения с централно разположени портици, коридор и странично от него разположени учебни помещения, каквато е и едната сграда между улиците „Аксаков“ и „6 септември“.

### 3. Градът и сградите на Постоянната художествена галерия

Мястото на новоучредената галерия в центъра на София в градоустройствен аспект е доста интересно. От Освобождението до 1910 г. центърът на младата столица е изцяло променен. Създаването на частен салон за изложби, почти едновременно с всички основни държавни институции означава, че културната общественост има нужда от такъв. Промяната на уличната мрежа при прехода от османско към модерно градоустройство е доста любопитен момент от гледна точка на сградата, която Тръпко Василев и Петър Морозов наемат за „Постоянна художествена галерия“, което ще проследим по градоустройствените планове.

На плана на София от 1878<sup>6</sup> г. (Фиг. 3) на старата улична мрежа с махалите са отбелязани новопостроените сгради. Сред тях са дворецът на княза, народният музей и църквата „Св. Крал“. Пред двореца е разположена градската градина. На изток от нея е „Алигина махала“. Сградите там не са обозначени, тъй като няма новопостроени такива, а старите са били паянтови и са събаряни. Такава е махалата, в която през 1910 г. Тръпко наема сградата на галерията. „Алигина махала“ граничи с бул. „Цар Освободител“<sup>7</sup> и „Топхане махала“, която е разположена южно от нея и достига до „Черната джамия“<sup>8</sup>. В тази част по това време все още е запазено османското градоустройство, улиците са разположени без ясен принцип, често пъти лъчеобразно и в плана те са отбелязани като свързващи се в широки пространства.



<sup>6</sup> Вж. Желева-Мартинс, Д. Биография на София. Исторически студии. С., изд. «Пространство и форма», 2006, с. 39-40, 56.

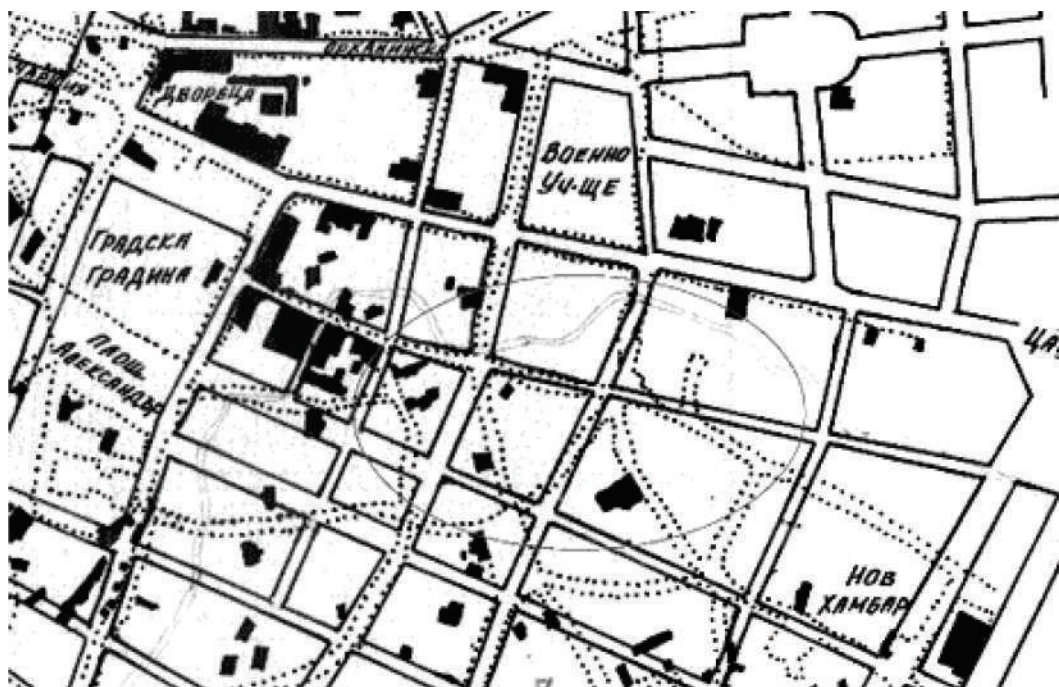
<sup>7</sup> Вж. Желева-Мартинс, Д., Ю. Фърков. История на българското градоустройство XIX-XX в. – част 2. Синхронни анализи на устройството на големите български градове. С., 2017, с. 95, 117..

<sup>8</sup> Сега църквата «Св. Седмочисленици».

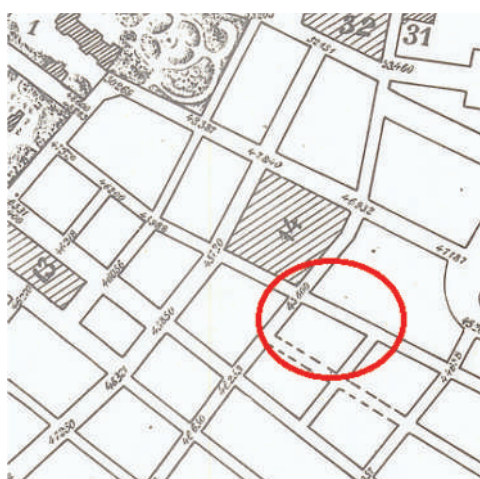


**Фиг. 3.** План на София 1878 г.<sup>9</sup>

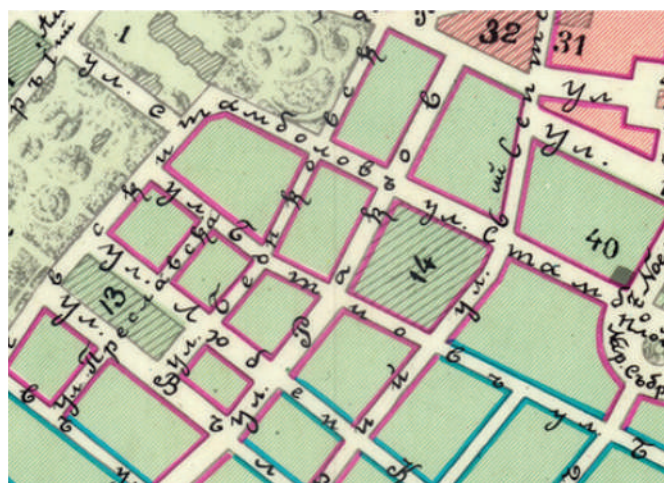
След Освобождението улиците в махалите все още следват османското градоустройство. На първия градоустройствения план на София от 1879 г. (Фиг. 4) върху старата улична мрежа е поставена новата регулация на улиците. Те са замислени по европейски образец и оформят правоъгълни квартали.



**Фиг. 4.** Детайл от градоустройствен план на София от 1879 (Батенбергов). С пунктир са обозначени старите квартали<sup>10</sup>



**Фиг. 5.** Градоустройствен план на София 1891 г. (детайл)



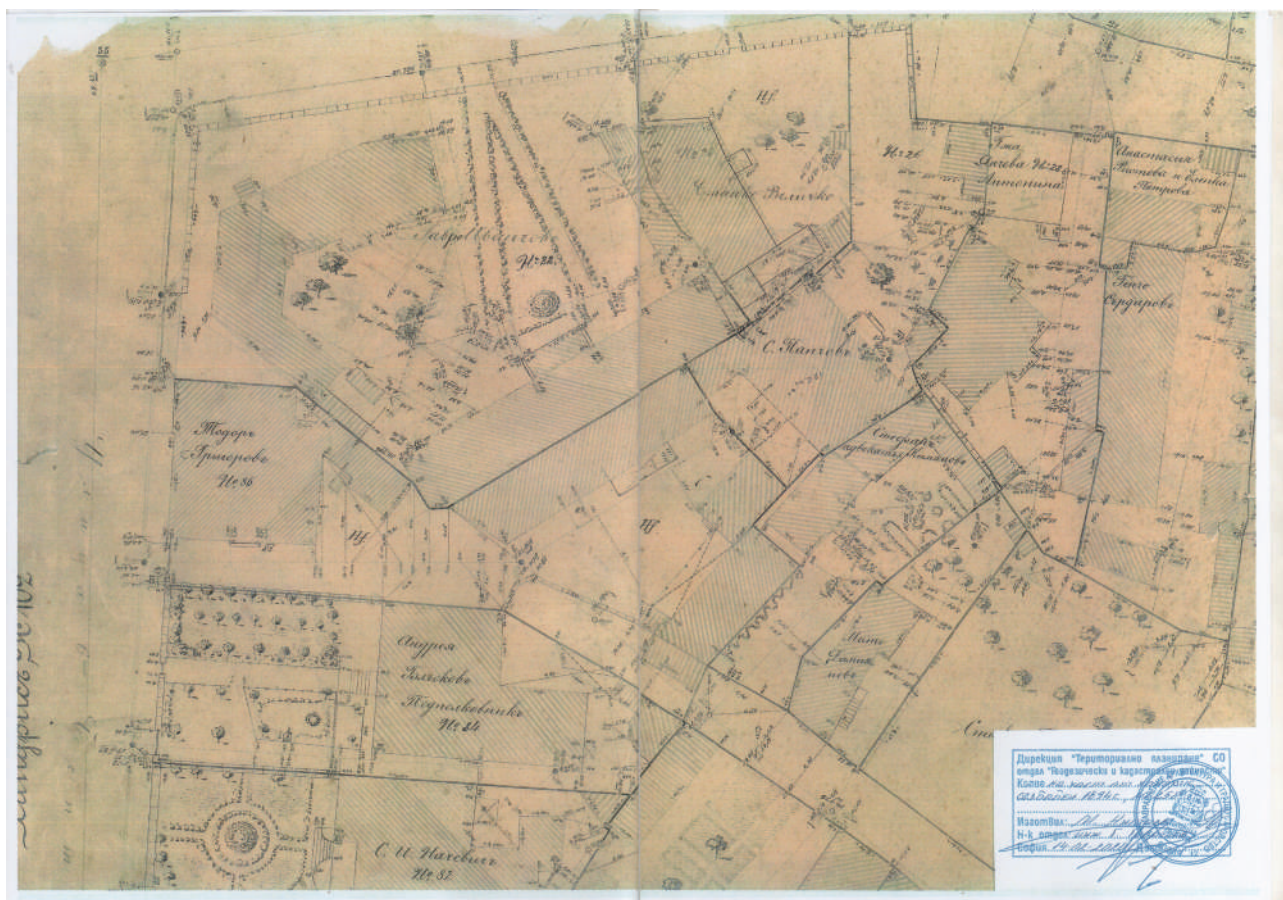
**Фиг. 6.** Градоустройствен план на София 1892 г. (детайл)<sup>11</sup> Сградата под № 14 е музейт, който се проектира. Постоянната художествена галерия е разположена южно от него.

<sup>9</sup> План на София в 1878 г. <http://stara-sofia.com/sofiemap1878.jpg>; <http://stara-sofia.com/karti.html>;

<sup>10</sup> Столична община, направление „Архитектура и градоустройство“, <https://www.sofia-agk.com/Pages/Render/990>



На градоустройствените планове на София от 1891 (Фиг. 5), и 1892 г. се вижда, че едно от централните места обозначено под № 14 (Фиг. 6), до което е близо бъдещата „Постоянна художествена галерия“, е предвидено за бъдещия музей на възраждането, който остава непостроен и в момента на това място има градинка.



Фиг. 7. Хандрис от 1894 г.<sup>12</sup>

На хандриса, ръчната скица (Фиг. 7), която представлява кадастрална снимка от 1894 г.<sup>13</sup> между улиците „6 септември“ и „Аксаков“ в бившата „Алигина махала“, личи запазването на две много характерни, подобни на катети на равностранен триъгълник алеи, които са улици от турско време. Те фигурират като стара улична мрежа и в плана от 1879 г. (Фиг. 4). На хандриса (Фиг. 7) личи, че в единия си край те се събират и опират в ул. „Аксаков“, а в другия водят до диагонално разположена навътре в парцела сграда. И тъй като тя е отбелязана на кадастралната снимка (Фиг. 7) като сграда в имота на Гавро Иванчов, това означава, че е съществувала по това време и по тази причина са запазени алеите, които водят до нея. Кварталът е с новозастроени сгради с градини между тях. Повечето от тях са разположени диагонално на парцела според логиката на разположението от османския период.

Интересуващата ни сграда е отбелязана и на плана на Бартел от 1897 г. (Фиг. 8). Получава се така, че характерната особеност на османската улична мрежа, която

<sup>11</sup> Столична община, направление „Архитектура и градоустройство“, <https://www.sofia-agk.com/Pages/Render/992>. В легендата специално вж. № 14.

<sup>12</sup> Направление Архитектура и градоустройство. Дирекция „Териториално планиране“ СО отдел „Геодетски и кадастрални дейности“.

<sup>13</sup> Направление Архитектура и градоустройство. Копие на част от хандрис създаден 1894 г. Дирекция „Териториално планиране“ СО отдел „Геодетски и кадастрални дейности“.



по природата си е хаотична, е запазена и при новото градоустройство поради плътната застройка, когато са създадени правоъгълни квартали и са планирани успоредни улици. Тя се вижда добре на плана от 1897 г. и поради тази причина кварталът не е разделен на две части в посока север-юг, както е планирано през 1891 г. (Фиг. 5).

От хандриса (Фиг. 7) личи, че дългата и тясна сграда е разположена на дълбочина над 14 м навътре от линията на улицата. През 1910 г. това е частната постройка, която първоначално използва Рисувалното училище, а след това е наета за „Постоянната художествена галерия“.

Планът на Бартел от 1897 г.<sup>14</sup> (Фиг. 8) видоизменя основно парцела, отбелязан на хандриса през 1894 г. като собственост на Гавро Иванчов (Фиг. 7), при граници със съседен имот на Станко Величко.

Към 1910 г. парцелът вече се ползва от „Предприятие Петър Кънев“, а след това става собственост на д-р Наум Николов Пасков. Около 30-те г. на XX в. архитектурното бюро на Иван Васильов и Димитър Цолов проектира жилищна кооперация на това място. Като цяло сградите вътре в парцела продължават да спазват диагонална ориентация, зададена от двете алеи или улици, за които стана въпрос до извършения от д-р Пасков строеж.



**Фиг. 8.** План на София от 1897 г. според кадастралната снимка на инж. Вилхелм Бартел, детайл.<sup>15</sup>

На плана на Бартел от 1897 г. (Фиг. 8) личи сградата, която на фотографията се вижда на преден план на ул. „Аксаков“ (фиг. 10). Става ясно, че тя е построена

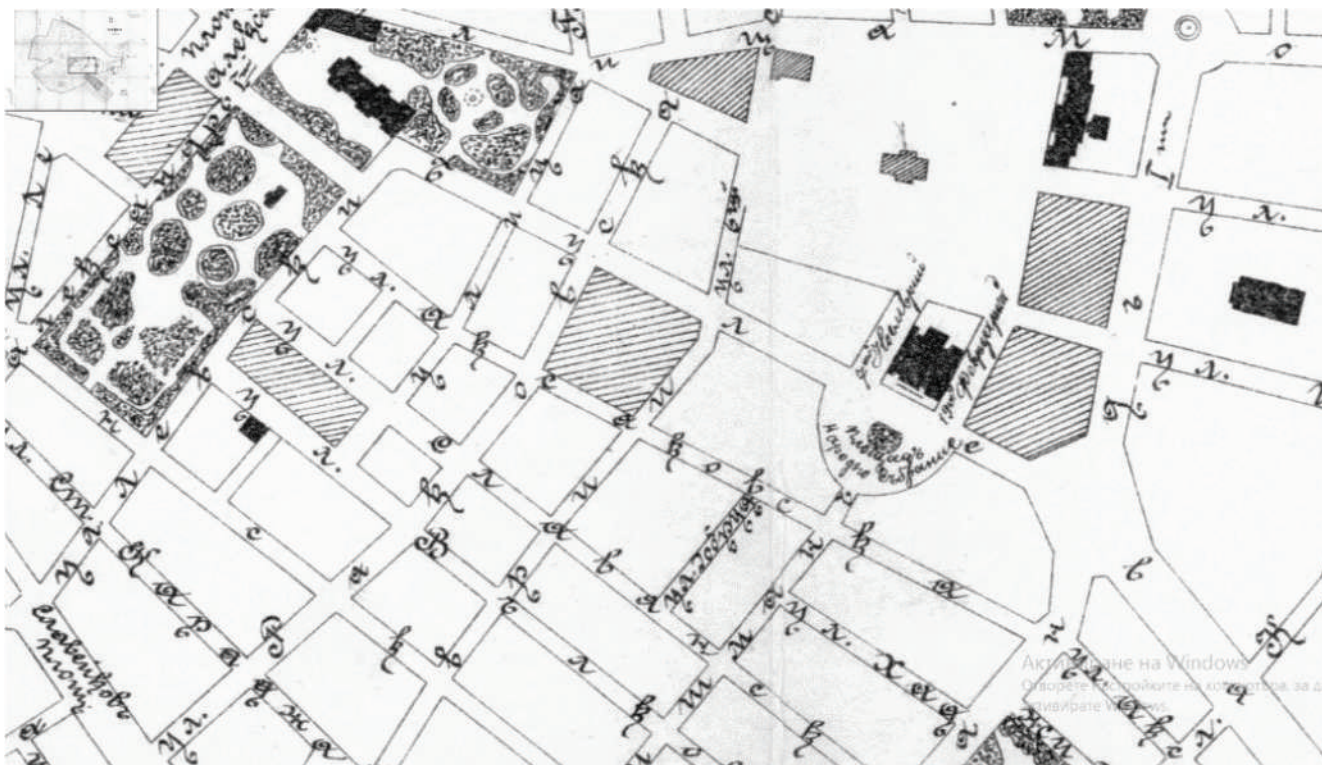
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<sup>14</sup> Столична община, направление „Архитектура и градоустройство“, <https://www.sofia-agk.com/Pages/Render/996>

<sup>15</sup> Столична община, направление „Архитектура и градоустройство“, <https://www.sofia-agk.com/Pages/Render/992>

след като е направена кадастралната снимка от 1894 и 1897 г. Тя е разположена на мястото на началото на алеите. Нейният вход е от към двора. Той е разположен така, че срещу него се пада входът на диагонално срещуположната му сграда, която се вижда на хандриса (Фиг. 7).

От тук се налага изводът, че по времето на новите градоустройствени промени в този парцел има една стара сграда от преди 1894 г. и една по-нова от ок. 1897. Техните входове са разположени един срещу друг. Вероятно и сградата, която е на линията на улицата също е била на „Предприятие Петър Кънев“ и е ползвана от Рисувалното училище.



Фиг. 9. Детайл от план на София от 1907 г. (Никола Лазаров).<sup>16</sup>

През 1907 г. улиците имат нова регулация.<sup>17</sup> Източно от градината пред двореца е улица „Левски“. „Алигина махала“ е регулирана. Изградена е уличната мрежа. В градината опира улица „Алабинска“, а от противоположната ѝ страна са улиците „Вълкович“, „Славянска“ и „Аксаковска“. От другата страна на ул. „Раковска“ „Аксаковска“ се пресича с улица „6 септември“, която води до „Черната джамия“.<sup>18</sup>

На плана са отбелязани градините, църквите, Народното събрание с площада, Царския манеж, кръглите площади с руския паметник и паметника на Васил Левски. Самият Тръпко Василев отбелязва, че галерията му се намира на ул. „Аксаков“ № 16.

<sup>16</sup> Блог „Стара София“, <http://stara-sofia.com/map1907.jpg> Столична община, направление „Архитектура и градоустройство“, <https://www.sofia-agk.com/Pages/Render/992>

<sup>17</sup> План на гр. София от 1907 г. Приложение към чл.92 от Закона за благоустройство на населените места. The world of maps and images, <https://maps.spaceschoolbg.eu/?p=238>

<sup>18</sup> План на град София 1907 с Подуени. Приложение към чл. 92 от закона за населените места. <http://stara-sofia.com/map1907.jpg>



В описа на инвентара на сградата той записва, че постройката е паянтова с марсилски керемиди<sup>19</sup> и платнен таван, 1 дирек и шест прозорци<sup>20, 21</sup>.

Описанието на Тръпко, спомените на Александър Божинов както и много други поставят въпроса била ли е Тръпковата галерия в сграда на Рисувалното училище, което както е известно, в началото след неговото създаване през 1896 г. се разполага в няколко частни сгради.<sup>22</sup>



**Фиг. 10.** Сградите на улиците «Аксаков» (вляво на фотографията) и на „6 септември“ (вдясно на фотографията).

Едно описание на Стефан Танев потвърждава написаното. За изгледа на галерията той пише на брат си Никола Танев, когато художникът завръщайки се от следването си в Париж, през 1912 г.<sup>23</sup> търси изложбена зала:

*„Има помещение на един бояджия, цялото гъбясало в бурения двор с мамули на ул. „Аксаков“, а бояджията Тръпко Василев ще го намериш във „Втора Шуменска“ [бирария]. Всички го познават. И той е нещо побъркан по изкуството, рисува и си харчи парите, за да поддържа онзи хамбар“.*<sup>24</sup>

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<sup>19</sup> Модерни за времето и навлизащи в покривните покрития по това време, съдейки по рекламите в периодичния печат.

<sup>20</sup> На горната снимка на сградата към улицата се виждат пет прозореца, а на сградата зад нея доста повече на брой.

<sup>21</sup> ЦДА, ф. 764 К, оп. 1, а. е. 20, л. 2.

<sup>22</sup> Юбилейно издание. Сто години Национална художествена академия. 1896-1996. С., НХА, с. 14-15.

<sup>23</sup> Божков, Ат. Никола Танев. С., 1956, „Български художник“, с. 16.

<sup>24</sup> Танев, Н. Тръпковата галерия. – в. Народна култура, орган на Министерството на просветата и културата, бр. 22, с. 6, 22. VI. 1957. Вж. и Никола Танев. Спомени (под ред. на Борис Делчев). С., 1971 „Български художник“, с. 115; Кюркчиев, В. Тръпковата галерия. Закъсняла признателност. Изрезка от вестник без издание и архивна страница. Пос. по Библиотека „Струмски“. ЦДА, ф. 1988 К, оп. 1, а. е. 39, л. 6.

Непретенциозният вид на сградата не притеснява Никола Танев, когато вижда, че галерията има горно осветление и решава, че ще направи там своята първа изложба в България.<sup>25</sup>

Връзката между Рисувалното училище и Тръпковата галерия по отношение на сградата се появява за пръв път в пресата. В едно интервю вече възрастният Ян Мърквичка при едно от последните си посещения в София е попитан за разговорите му с Константин Величков за създаването на Рисувалното училище. От разказа на Мърквичка журналистът прави следния извод:

*„Академията е била открита в старата Тръпкова галерия – същото място, което сега е галерия Аксаков. Персоналът броил само двамина Мърквичка за директор и бай Дойчо за разсилен и писар, доскорошен касиер на галерията“<sup>26</sup>*

В спомените си за Рисувалното училище Александър Божинов отбелязва:

*„1897 г., през април, членовете на „Дружеството на художниците в България“<sup>27</sup> нареди изложба в едно от помещенията на Рисувалното училище, което се намираше на ул. „Аксаков“. После, след като се построи специално здание за него, това помещение стана така наречената Тръпкова галерия“<sup>28</sup>.*

Божинов добавя интересна история, че противникът на създаването на Рисувалното училище Алеко Константинов посетил изложбата. Когато той си тръгнал, касиерът на галерията, след като не бил познал Щастливеца хукнал след него за да го види и го догонил, когато той вече бил преминал през двора и завил наляво към ул. „6 септември“ и „Раковски“. Този спомен съдържа две важни свидетелства. Сградата на Рисувалното училище е разположена в двор и е съвсем близо до кръстовището с ул. „6 септември“.

Тръпковата галерия е отбелязана и в спомените на Димо Казасов:

*„От Министерството на вътрешните работи на север по улица „Шести септември“, вдясно, при пресечката ѝ с ул. „Аксаков“, имаше празно място, в източната страна на което беше клекнал едноетажен паянтов салон. В него се помещаваше Рисувалното училище, чиито студенти при хубаво време рисуваха на двора. По-късно, когато бе издигната днешната сграда на Рисувалното училище, художникът Тръпко Василев преустрои това помещение в галерия, където години подред ставаха художествените изложби.“<sup>29</sup>*

В изданието, посветено на 100 годишнината на Художествената академия е отбелязано, че една от сградите на Рисувалното училище се намира на улиците „Аксаков“ и „6 септември“. На снимка 9 в същото издание е отбелязано, че сградата се намира на ул. „Аксаков“ № 22.<sup>30</sup> Този номер на улицата сега е близо до пресичането на улиците „Аксаков“ и „Цар Иван Шишман“. Документално твърдение за ул. „Аксаков“ № 22 на сградата има и в бележките на Антон Митов от времето, когато той е директор на училището:

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<sup>25</sup> Никола Танев. Спомени. (под ред. на Борис Делчев). С., 1971, „Български художник“, с. 116.

<sup>26</sup> Душков, Ат. Една забравена беседа с Иван Мърквичка. Неизвестни подробности за художническа България. Как бе създадена днешната Художествена академия. Заслугите на К. Величков и Мърквичка, първият директор. Художниците от първия випуск. Какво даде първият директор на своето второ отечество. Изрезка от вестник в архива на Тръпко Василев, изданието не е посочено, бр. 5671, ЦДА, ф. 764 К, оп. 1, а. е. 17, л. 134.

<sup>27</sup> Става въпрос за „Дружество за поддържане на изкуството в България“, основано през 1893 г. и неговата втора изложба.

<sup>28</sup> Божинов, Ал. Минали дни: издание, допълнено с непубликувани снимки. С., 2017, 2. доп. изд. Изток-Запад, с. 31 – 33.

<sup>29</sup> Казасов, Д. Някогашните наши градове., изд. Отечествен фронт, С., 1975, с. 166-167.

<sup>30</sup> Юбилейно издание. 100 години Национална художествена академия 1896 – 1996., С., 1996, с. 15.

*„Най-напред училището се помещаваше в частно здание на ул. Аксаков № 22 (днешната постоянна галерия на г. Тръпко Василев), после се разшири в зданието на д-р Миркович (на същата улица), а след това в зданието на ул. Сан-Стефано и Оборище (в зданието на Б. Станкович) от дете през 1906 г. се премести в собственото си здание на ул. Шипка № 1 – недовършено и сега.“<sup>31</sup>*

Очевидно номерацията между 1896 и 1910 г. е различна, защото Тръпко Василев посочва за своята галерия № 16 на същата улица.

Още информация от това време за мястото на Рисувалното училище откриваме и в „Първий илюстриран годишник на Държавното рисувално училище в София“ за учебната 1904-1905 г., където е отбелязано, че то се намира на „Аксаков“ и „6 септември“ и до 1905-1906 г. все още е в частните помещения. Първата сграда на новото място – ул. „Шипка“ № 1 е за „светливите картини“ – известните и привлекателни за софийските културни среди лекции по история на изкуството на Антон Митов.<sup>32</sup>



Фиг. 11. Скица на Тръпковата галерия от Никола Танев от 1919 г.

Съдейки по редица запазени фотографии, училищата в София ползват тесни, дълги едноетажни сгради, с двускатни покриви – такива, както е нарисувана от Никола Танев сградата на Тръпковата галерия (Фиг. 11).<sup>33</sup> На снимката (Фиг. 10) се вижда, че сградата на Рисувалното училище има централно разположен на фасадата ризалит, по-скоро напоявящ зазидан портик, такъв, какъвто се вижда и на хандриса и на плана на Бартел (Фиг. 7,8).

Александър Балабанов пише във вестник „Пряпорец“, че Тръпковата галерия е в сградата на редакцията на вестник „Балканска трибуна“.<sup>34</sup> Печатницата на

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<sup>31</sup> Пос. по Материали от юбилейните сборници на Художественото училище (1907 и 1927 г.), дигитализирани от Народната библиотека, проф. А. Митов, Директор, Общи бележки за развоя на нашето художествено училище (1927 г.). – Блог Стара София, 05/14/18, <http://stara-sofia.blogspot.com/2018/05/1927.html>.

<sup>32</sup> Първий илюстриран годишник на Държавното рисувално училище в София за учебната 1904-1905 г. с. 6-7.

<sup>33</sup> Например „Иван Денкоглу“, „Св. Седмочисленици“, „Патриарх Евтимий“, вж. ДА - София, ф. 5 К, оп. 1, а. е. 319, л. 163, 184, 225, 230.

<sup>34</sup> Кюркчиев, В. Тръпковата Галерия, изрезка от вестник в архива на Боян Мирчев, София, 1954 година. Библиотека Струмски. <http://strumski.com/biblioteka/?id=22>.



опозиционния вестник „Балканска трибуна“ е горяла<sup>35</sup> и е напълно възможно да е ползвала същата сграда след преместването на Рисувалното училище и преди наемането на сградата от Тръпко Василев. Спомените, че той все нещо майсторел по нея, издават недоброто ѝ състояние.

Мястото на Тръпковата галерия показва, че става въпрос за сграда, която Тръпко Василев наема, след като тя е била наемана от Рисувалното училище също под наем. Сградата е близо до Двореца, Военния клуб и до петното, което е учредено за строителство на непостроения музей на Българското възраждане. Местоположението ѝ е съвършено.

По това време населението в столицата е значително увеличено – 103 000 души спрямо 1880 г., когато в София живеят 20 500 души. В съседство с галерията е кафене „Алказар“ на бул. „Цар Освободител“, преди построяването на новия хотел „България“ (1935-1937 г.)<sup>36</sup> – място за сбирките на софийските интелектуалци, търговци, банкери, индустриалци, журналисти, адвокати и „млади социалдемократи“; на ъгъла на ул. „Аксаков“ и ул. „Левски“ до самата Борисова градина е кафене „България“, а според спомените на Димо Казасов в гостилница „Средна гора“ на ул. „Раковски“ отсядат българановците<sup>37</sup> Елин Пелин, Александър Божинов, Стефан Лазаров Костов и Андрей Протич.<sup>38</sup> По това време в София вече са построени Военния клуб, австрийското и италианското посолства. Срещу тях е фотографското ателие на Димитър Карастоянов. В съседство с него по бул. „Цар Освободител“ са жилищните сгради на Иван Евстратиев Гешов и д-р Димитър Моллов.<sup>39</sup> Галерията е в центъра на съвременния градски живот. Тя е в близост до Народното събрание, а депутати, банкери и индустриалци посещават изложбите и купуват произведения от организирания изложби. Можем да добавим още, че Тръпко Василев наема сградата, защото я познава, защото учи в нея по времето на краткото си следване.

В спомените си Константин Константинов описва по-съществените сгради в центъра на София и отбелязва, че на изток от Народното събрание се намира Рисувалното училище, на площада зад паметника (на цар Освободител) е Етнографският музей, а малко по-на запад на „Аксаков“ – първата „Постоянна художествена галерия“. Той отбелязва, че тези пет-шестнайсет хиляди жители на столицата по това време имат Тръпковата галерия, която някои от тях приемат за кокетно уредена сграда (според списание „Славянски глас“ от 1911 г.), а други я смятат за вечно рушаща се, тъй като Тръпко Василев извършва чести ремонти в нея.<sup>40</sup>

С наемането на сградата за художествени изложби Тръпко Василев е новатор. Тук може би е уместно да бъде направена аналогия със строителните дейности, които след Освобождението набират голяма скорост. Строителното предприемачество е новаторска дейност и художествено-декораторската работа на Тръпко Василев е част от нея. Познавайки този механизъм, той пръв започва да се

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<sup>35</sup> Вестник „Балканска трибуна“, Черната джамия и освиркването на княз Фердинанд. Из: Михалаки Георгиев. Съчинения, том III, „Български писател“, 1961 г. Под редакцията на Симеон Султанов, Иван Сестримски, Б. М. Георгиев, пос. по <https://beinsa.bg/izgrev.php?id=6465>

<sup>36</sup> По спомените на Константин Константинов. Вж. Константинов, К. Път през годините. С., изд. „Захарий Стоянов“, 2015, с. 352-353.

<sup>37</sup> Георгиева, М. Личности в българската карикатура - Александър Божинов като редактор на вестник „Българан“ (1904-1909). - Проблеми на изкуството, 2012, бр. 4, с. 50-59; Георгиева, М. Непознати имена на български художници във в. "Българан" (1904-1909) - опит за реконструкция и видимост. В: Изкуствоведски четения, 2011 (2012), с. 480-493.

<sup>38</sup> Казасов Д. Следи от минали дни. С., изд. „Отечествен фронт“, 1971, с. 262.

<sup>39</sup> Пак там, с. 356.

<sup>40</sup> Константинов, К. Път през годините. С., изд. „Захарий Стоянов“, С., 2015, с. 351.

изявява като „художествен предприемач“. Скоро неговият съдружник Петър Морозов се оттегля, тъй като започва специализация при Йозеф Питер (1911-1915 г.). Тръпко продължава сам като организира изложби и продава художествени произведения. Постепенно във времето Тръпко създава кръг художници, с които работи. В същото време неговата галерия е търсена като единствената в София.

Както уточнихме, собственик на сградата на ул. „Аксаков“ е „Предприятие Петър Кънев“ (“Pierre Kaneff, Sophia”). На 15. IX. 1910 г. той изпраща писмо, адресирано до художника-декоратор Тръпко Василев и художника Петър Морозов, с което изисква взетите под наем сгради да бъдат оградени за тяхна сметка от общия двор.

*„На имота с ограда и вход ще оставите от към ул. „Аксаков“ до самото здание с врата, най-много до 1,80 м широчина. Всичко направено от вас след изтичане на шестгодишния период, ако не се спогодим за продължение на наема, остава в мое ползване“.*<sup>41</sup>

На 18. IX. 1910 г. Петър Морозов изпраща отговор на горното писмо, в който пише:

*„Ремонтите направени от нас в сградите ви на Аксаков ще останат след изтичането на пет годишния срок във Ваше ползване, обаче това няма да се отнася и за новопостроеното ателие в дворът, както и за други, които бихме по-нататък построили в същия двор. Те са отделни от другите сгради, с подвижна конструкция и ние, ако не можем да се спогодим за продължение на наемането ще ги преместим от имотите Ви. Молим да се разпоредите да се направи тротоара на улицата около зданието и чеима в двора и нужник в сградата. П. Морозов“.*<sup>42</sup>

В заключение можем да отбележим, че на фотографията и на градоустройствените планове още от 1897 г. в парцела се виждат две сгради със срещуположни входове. Наемателите Тръпко Василев и Петър Морозов строят и ателие.

Официалното откриване на Постоянната художествена галерия е на 28 ноември 1910 г., според публикацията в списание „Художествена култура“<sup>43</sup>. В богато украсените салони се стича многолика публика. Според статията салоните са богато декорирани. На публикуваната фотография (Фиг. 12) се виждат стени, оформени в горната част с орнаменти, така както Тръпко е научен и може да работи. Интериорът издава вкус и премереност. На пода под витрината има килим. От тавана висят изящни осветителни тела. На всяко едно произведение е отредено достатъчно място. Съобразен е характерът на отделните творби и начинът на тяхното експониране. Оформени са отделни зони и кътове. Между експонатите са поставени тапицирани пейки и табуретки по европейски маниер.

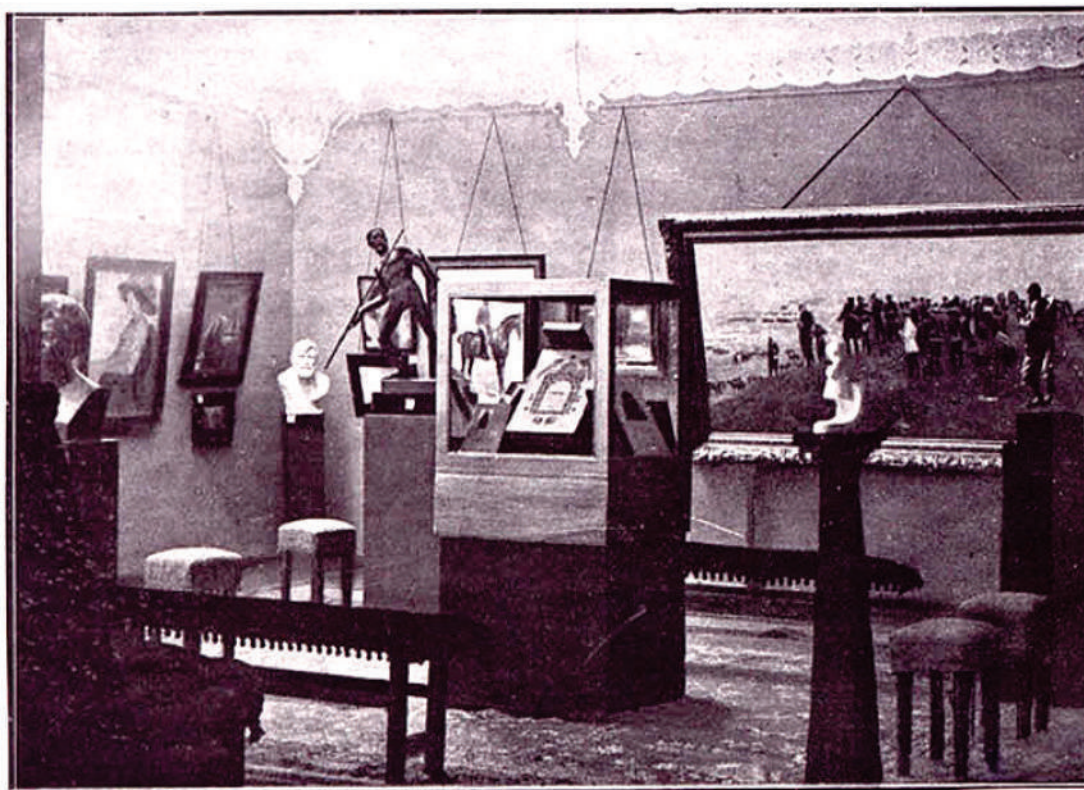
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<sup>41</sup> ЦДА, ф. 764 К, оп. 1, а. е. 21, л. 12.

<sup>42</sup> Пак там, л. 13.

<sup>43</sup> Автор не е посочен. Постоянната художествена галерия в столицата. – Художествена култура, 1911, януари-февруари, кн. 11-12, с. 36, 40.





Фиг. 12. Интериор от откриването на Постоянната художествена галерия.<sup>44</sup>



Фиг.13. Изложбата на дружество „Съвременно изкуство“ през 1897 г. в сградата на ул. „Аксаков“ 16.

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<sup>44</sup> Постоянната художествена галерия в столицата. – Художествена култура, 1911, март-април, кн. 13-14, с. 70.



В списание „Изкуство“ е публикувана репродукция от втората художествена изложба на Дружеството за поддържане на изкуството в България, организирана през април 1897 г.<sup>45</sup> (Фиг. 13). Изложбата е разположена в тясно и дълго помещение с горно осветление.



**Фиг. 14.** Интериор на Тръпковата галерия след нейното разширение.

Съдейки по запазените фотографии от архива на Тръпко Василев, между двете изложбени зали (Фиг. 13, 14) има множество прилики и няколко различия. Пространството на Тръпковата галерия също е тясно и дълго с обертхт само в едната половина. Обертхтът от долната си страна е снабден с въжета, на които хоризонтално са разпъвани платна за разсейване на светлината в залата. Това важно техническо средство личи от една архивна фотография с изложба, която не може да бъде отнесена към творчеството на един художник или на група художници.<sup>46</sup>

Допускам, че старата паянтова сграда на Рисувалното училище с обертхта, в която е показана изложбата на Дружеството на художниците в България, е разширена от Тръпко, който купува много строителни материали и той свързва с коридор срещуположните входи на двете сгради<sup>47</sup>. Поради тази причина в по-старата сграда се появяват стълбовете и колоните по надлъжната ос. Разширената част не е така солидна и заради това преустройство помещението е влажно и Тръпко постоянно се занимава с ремонти на пода или стените, както съвременниците

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<sup>45</sup> Изкуство, под ред. на Антон Митов и Иван Мърквичка 1897, кн. 1, с. 3 – 4, вж.

<http://82.147.128.134/prd/public/image>

<sup>46</sup> ЦДА, ф. 764 К, оп. 1, а. е. 28, л. 1. Любопитството подбуждат изложените произведения. На една от големите картини се виждат две дървета и седнала човешка фигура вляво от тях. На съседното голямо платно е изобразена църква с купол на висок барабан и още четири малки купола. Пред нея има човешка фигура. На друга снимка (Пак там, л. 2) вероятно от същата изложба се вижда каменист пейзаж с (гола?) женска фигура с дълга разпусната коса седнала върху камък, а вдясно от нея нахвърляни човешки тела (?).

<sup>47</sup> Тази логика запазва и вътрешният обем, построен в двора зад сградата на д-р Паяков около 1930 г. Към него се преминава от вход от улицата, който е развит под жилищната сграда.

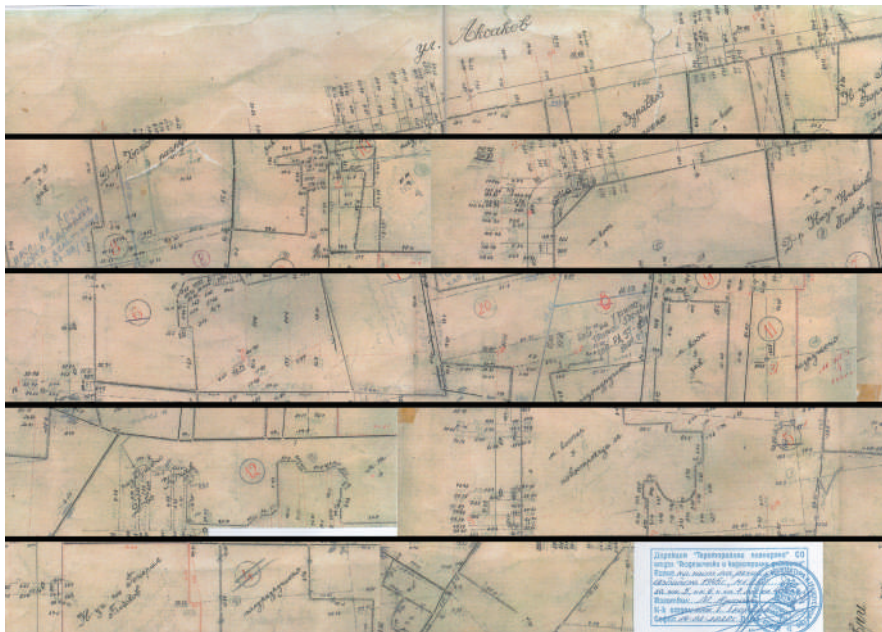
коментират. В спомените си той споделя: „За да направя салона годен за изложби аз похарчих осем хиляди тогавашни пари.“<sup>48</sup>

#### 4. Сградите в квартала

За останалите сгради в квартала между улиците „Аксаков“, „6 септември“, „Славянска“ и „Добруджа“ създава представа хандриса от 1894 г. (Фиг. 7) и още една ръчна скица от 1945 г. (Фиг. 15). На хандриса от 1894 г. личат градините, зелените площи и дворовете в квартала. На ул. „6 септември“ пред къщата на подполковник Андрея Блъсков има градина с алея в средата, която води до стълбището към сградата. В съседство с нея към ул. „Славянска“ е къщата на С. И. Начевич, също с градина. Тя е забележителна с централно разположените концентрични зелени площи и стълбището. Двете сгради са изтеглени назад от уличната линия. След войната сградата на подполковник Блъсков е полуразрушена, а на мястото на градината на Начевич е построена нова сграда, обозначена като сграда „Наследници на генерал Блъсков“. На дворното място на градината на Блъсков се строи нова жилищна сграда с „апендикс“ във формата на полудъга, която в днешно време е видима от страната на двора.

На мястото на едноетажната сграда на ул. „Аксаков“, където е била Тръпковата галерия (мястото обозначено на хандриса от 1894 г. като собственост на Гавро Иванчов), към 1930 г. е построена жилищната сграда на д-р Наум Николов Пасков. Вътрешната паянтова сграда е съборена и на нейно място е построена друга с размери 9,88 x 15 м, която е свързана с кооперацията посредством коридор и която след 1944 г. служи за ателие по скулптура на Художествената академия.

Новата жилищна сграда на ул. „Аксаков“ № 16 е с високи прозорци под полудъгвидни арки и някои фотографии показват, че след събарянето на двете стари сгради това пространство е използвано за галерия, но не от Тръпко Василев. Името „Тръпкова галерия“ остава, независимо от официалното галерия „Аксаков“, което личи от различни надписани от художници фотографии по повод на техни изложби.



**Фиг. 15.** Направление Архитектура и градоустройство. Дирекция „Териториално планиране“ СО отдел „Геодезически и кадастрални дейности“.

<sup>48</sup> ЦДА, ф. 1988 К, оп. 1, а. е. 39, л. 2. Пос. по библиотека „Струмски“, <http://strumski.com/biblioteka/?id=2218>





**Фиг. 16** Жилищната сграда на д-р Наум Николов Пасков на ул. „Аксаков“ 16.

#### **4. Заключение**

В днешно време ъгълът на улиците „Аксаков“ и „6 септември“ е особено оживен. На посетителите на съвременните заведения, разположени на мястото на Тръпковата галерия не са известни изнесените и анализирани факти. След осъществените две изложби и извършено обстойно изследване, предстоят дейности за постъпки пред Софийската община за поставяне на паметна плоча за да бъде запазена паметта на това значимо с богата си художествена дейност до 1944 г. място.



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Заслугите на К. Величков и Мърквичка, първият директор. Художниците от първия випуск. Какво даде първият директор на своето второ отечество. Изрезка от вестник в архива на Тръпко Василев, изданието не е посочено, бр. 5671, ЦДА, ф. 764 К, оп. 1, а. е. 17, л. 134.

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**THE EFFECT AND PROTECTION OF MISSIONARY RELIGIOUS  
BUILDINGS ON URBAN FORMATION – THE CASE OF AINTAB  
(GAZIANTEP)**

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*Daily routines of ethnic minorities, which are effective in determining the physical characteristics of cities, are reflected in the architectural products. Among the important components of the urban identity, buildings built for the purposes of religion, education, health and accommodation are intended to be used by large groups of people. These important cultural heritage assets reflect the physical expression of social thinking and cultural life of ethnic minorities,.*

*This paper is about city of Aintab (Gaziantep), which is located in the territory of the Ottoman Empire, for approximately 50-60 years in the 19th and 20th centuries. Several public buildings, such as schools, churches and hospitals were built in the city during that time as a result of missionary activities. The aim of this paper is to examine urban distribution and architectural properties of missionary religious buildings in the city of Gaziantep and to develop ways to protect them and to carry them into the future.*

*The method of the study includes the content analysis of the archival documents from Ottoman Empire, identification of buildings through fieldwork and gathering information from local historians via oral history research. A comparative analysis is undertaken by interpretation of the maps created for the study, where the establishment of religious buildings and their historical development is shown.*

*As a result of the study a set of criteria for ensuring the continuity of missionary religious buildings focusing on the determination of their social and cultural value is developed and a sustainable strategy for their protection is determined.*

**Key words:** *Gaziantep, Missionary Architecture, Religious buildings, Conservation and Restoration.*

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## 1. Introduction

Gaziantep (Aintab) in Turkey where was founded in the Southeastern Anatolia region that suitable for settlement, history goes back to the Paleolithic Age [1]. Aintab, which has hosted many civilizations, is located on the historical Silk Road route that continues in the east-west direction.

The first settlement is Doliche (Dülük) Ancient City, which is 12 km north of the city [6]. This area has developed around the castle and the walls in the Seferpaşa Neighborhood where suitable agricultural lands are located, close to the Alleben stream (Fig 1). This development initially continued inside the city walls, as the population increased over time and security problems disappeared, it continued to grow outside the walls.

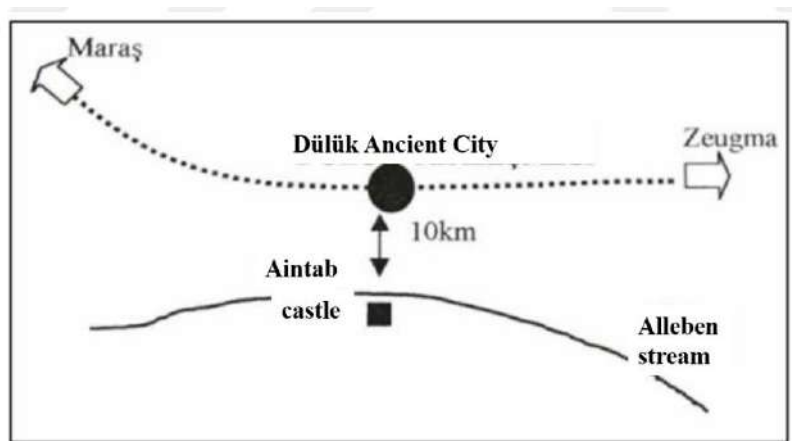


**Fig 1.** Relationship of Aintab-Dülük settlements in BC [15]

This urban formation in Aintab was realized as in the examples of the Turkish-Islamic city; The Friday mosque, neighborhood relations, and bazaar structures are the main elements that shape the city. Urban growth occurs with the mosque in the centerline that creates urban identity. The areas where the mosque was built have also enabled the city to determine in which direction it will develop [5].

Over time, the climatic-topographic features, the development of trade, the Ottoman State settlement policies, the westernization movements that started as a result of the Tanzimat (1838) and Reform (1856). Edicts and the buildings formed as a result of missionary activities have also shaped the economic, human, and physical development of the city and created a new urban, cultural identity.

Aintab city was shaped near the Alleben stream, the location of the castle and the Aleppo commercial road axis [14] (Fig. 2). The traditional city shape, which has been going on for many years, has changed very rapidly due to missionary activities.

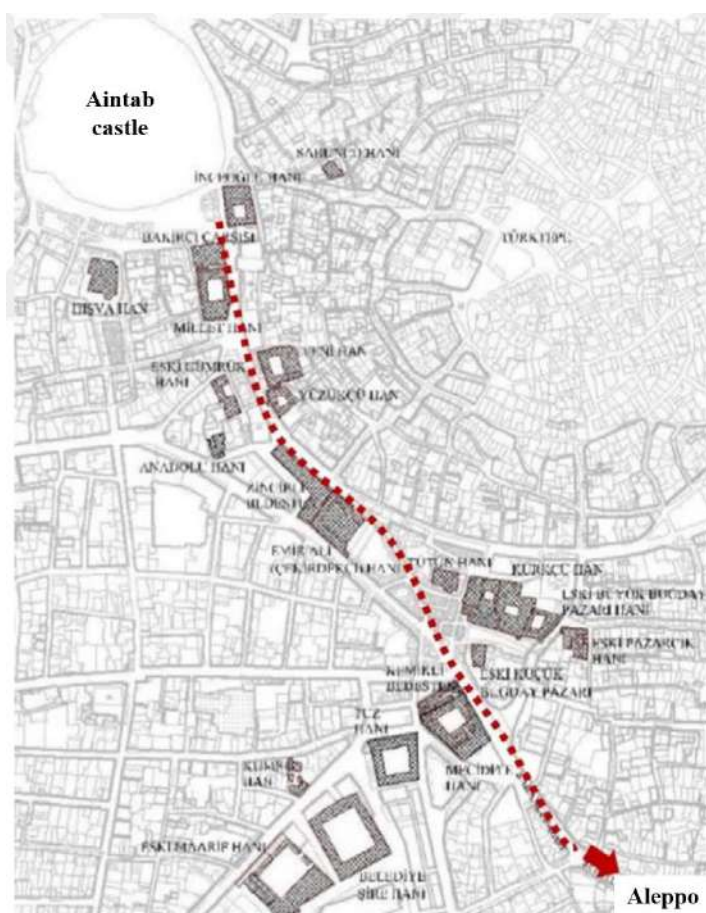


**Fig. 2.** Trade axis from the Aintab castle to Aleppo [19]

Especially the arrival of the Missionaries to Aintab in 1818 and the acceleration of the activities of ABCFM organization at 1848, caused the missionary structures in the city to increase rapidly [9]. These important buildings in terms of changing the identity of the city are various educational institutions, hospitals, soup kitchens, orphanages, and religious buildings.

This study is limited to the investigation of the effect of missionary religious structures on the city development of Aintab.

## **2. Urban Development of Aintab (Gaziantep)**

It is possible to observe the missionary activities of American-Protestants, Mormons and French-Catholics in the 19th century in Aintab [8]. The presence of people from different ethnic and religious communities is effective in the development of the city.

Muslims and Jews settled in the plains but the Christian communities settled on the high hills of the city. The Muslim population settled in the north and east of the city (in the Karagöz, Tişlaki, Şekeroğlu and Çukur districts), the Armenian population was in the Bey and Akyol districts in the west, and the Jews were in the Dugmeci neighborhood [16] (Fig. 3)





behavior of the masses over time and have also been reflected in the physical form of the cities.

In the 19th century, Turkey was described as " Bible Country " also it has been seen as a gateway to Asia. Turkey, in the borders of the Ottoman Empire, has been a valuable geopolitical location of was therefore at the focus of many missionary organizations activities [18].

The non-Muslim population which besides the Muslim population was also quite high in Aintab mostly Armenians. There were also Jews and Greek communities. [4]. Missionary religious buildings built in the city served these communities of ethnic and religious origin.

Six of the seven churches designed for missionary activities were built in the city in the 19th century. These structures were a Gregorian, an Armenian Catholic, a Latin Catholic, three Protestants, and an Anglican Church [2] (Fig. 5, Fig. 6)



**Fig. 5.** Panoramic silhouette from Kürdtepe of Aintab (1.Latin Catholic church, 2. First Protestant church, 3. Gregorian church, 4. Second Protestant church, 5. Anglican church) [Sitki Severoğlu photo collection retrieved from 2].

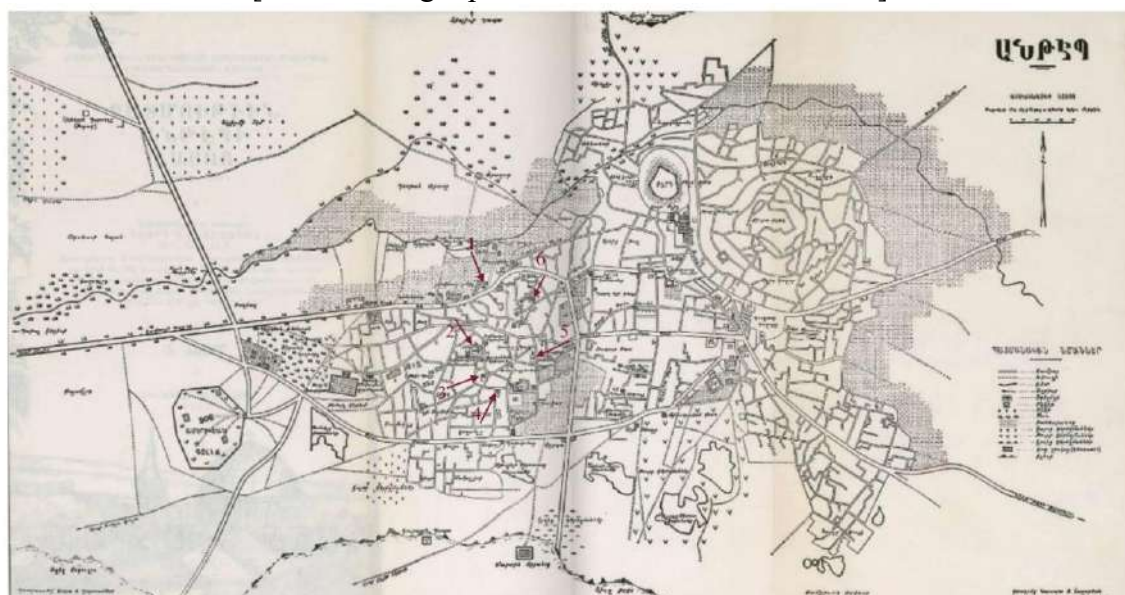


**Fig. 6.** Map of Aintab at 1920 (1.Latin Catholic church, 2. Gregorian church, 3. Second Protestant church, 4. Armenian Catholic church, 5. Anglican church, 6. First Protestant church) [Murat Uçaner photo collection retrieved from 2].

### 3.1. Armenian Protestant Churches

#### 3.1.1. Kayacik Church (1855)

The church, which was created in 1848, continued its activities in Kayacık Church, also known as the First Protestant Armenian Church, built by ABCFM Missionaries in 1855 in Kayacık Street. It was the biggest Protestant church building in the country. There were a primary school and a secondary school for girls and boys in the church. The building, which was later used as a cinema, was disappeared by fire [4]. (Fig. 7)



**Fig. 7.** Kayacık First Protestant Armenian Church , [11,no: 90435.29].

#### 3.1.2. Hayik Church (Aziz Bedros-1865)

Also known as the Second Protestant Armenian Church, it was opened for worship by ABCFM Missionaries in 1865 in the Tepebaşı neighborhood.

It was used as the warehouse of the spinning mill belonging to the Alevli family, which started operating in 1937/1938.

The church, whose restoration was completed in 2008, is called Ömer Ersoy Cultural Center and is used as a meeting and concert venue [4].

#### 3.1.3. Third Protestan Church (Alay Bey & İbni Eyüp-1879)

Durma states that the building belongs to ABCFM Missionaries, it was built in 1879 in the Alay Bey neighborhood and continued its activities until 1915 [4]. But Cephaneigil mentions that this structure was never built, based on Sarafian's documents [2].



### 3.1.4. Protestant Church of Eyüboğlu Neighborhood (1906)

It was opened to worship in Eyüboğlu Neighborhood in 1906, by ABCFM Missionaries. But the structure has disappeared. [7].



Fig. 8. Protestant Church of Eyüboğlu Neighborhood [Uğuluer, 2008 retrieved from; 7]

### 3.1.5. Protestant Church of Kozanlı Neighborhood

It was opened to worship in Kozanlı Neighborhood in 1906 by ABCFM Missionaries. But the structure has disappeared [7].

### 3.2. Armenian Orthodox Churches

#### 3.2.1. St Mary Church (Surp Asdvadzadzin Cathedral, 1873-1892)

It was the largest Gregorian Church in the city. The community, which previously used a cave as a church, worshiped in this church in Tepebaşı District after 1892. Designed by Armenian Architect Sarkis Bey Balyan, inspired by Hagia Sophia Church [12] (Fig. 9, Fig. 10) The crosses of the church are placed higher than the tallest minaret in the city [9]. The building, which was also used as a prison, has been used by the Muslim community as the called Kurtuluş Mosque since 1988.



a)



b)

Fig. 9. a) St Mary Church [11; 90435.34]. b) St Mary Church North facade (photo by Bozgeyik, 2019)

### 3.3. Latin Catholic Churches

#### 3.3.1. Kendirli Church and Latin School (1898-1903)

The Franciscan Monastery was built in Tepebaşı neighborhood in 1860, by the French Missionaries. Later, Kendirli Church (the Second Catholic Church) was opened in 1903, on Atatürk Boulevard in Bey neighborhood [4]. School and Church buildings are placed perpendicular to each other, forming the L form (Fig. 10a).

The Church was used as a Community Center, Teachers School, Hotel Management High School, and Gazi Cultural Center. The building, whose restoration work continues today, will be used as the Anatolian Archeology and Cultural Heritage Institute with the support of the European Union (Fig. 10b).



a)



b)

**Fig. 10.** a)Kendirli Church Situation Plan [2]; b) Urban Location in 1944 [Tatlıgil, 2005 retrieved from;10].

#### 3.4. Anglican Church

The church, thought to have been built by British missionaries in the 19th century, was built in Eyüpoğlu Neighborhood for the Armenian community of the Anglican sect. But the structure has disappeared [10].

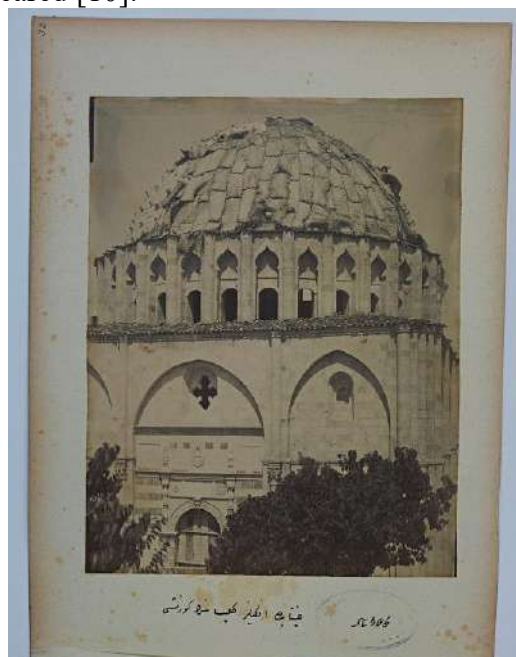


**Fig. 11.** Anglikan Church, [11; 90435.30]

#### 4. Conclusion

In the 19th century, many organizations carried out missionary work in the territory of the Ottoman Empire. These activities expanded the borders of the city by shifting to the city's west with the effect of its ethnic and religious origins. Academic studies and information about these structures are very limited.

Other types of buildings together with religious structures have also changed the silhouette of the city. Accessibility and continuity of access to religious buildings are important in Christian cities.

It is important that the churches have an imposing mass and the cross symbol on it is located higher than the highest mosque realm in the city. In this way, it was aimed to make Christianity dominant in the city skyline compared to other religions. All church buildings in the city were built together on a single road axis (Adil Özbek Street) and close to the congregation areas in the west of the city.

The aim of this study is that belongs to 19th-century missionary religious buildings that affect the urban formation, are one of the cultural assets that need to be protected. When the urban space design is observed, it is understood that these structures are important landmarks of the city.

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## BUILDING № 1 FROM THE MARBLE CITY LYUTITSA

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**Abstract:** *Lyutitsa is one of the most preserved examples of fortification found on the territory of modern days Bulgaria. Its location is less than 10 km from the Municipality of Ivaylovgrad. Its remains cover an area of more than 26 decares. Although that the fortress walls and part of the battle towers still rise to a height of more than 10 meters, unfortunately its urban architecture does not have the same destiny. The terrain inside of the fortified territory was levelled in the 16th-19th centuries. Moreover, the monumental buildings dated to the 9th-13th centuries were destroyed by the end of the 13th century. Subsequently, later on the architectural debris were reused in the repair of the 14th century fortification.*

*During the last nine years a monumental building numb.1 has been investigated, situated next to the only registered entrance of west site of the city. Its remains cover an area of over 220 sq. m., the thickness of the preserved layer is from 0.10 up to 1.20 m and the walls width is around 0.83 m. The construction is made by quarry stones welded with mortar containing small ceramic admixture. The foundations are dug in and laid on the rock. On the rock were observed removal traces linked with the extraction of construction material. This publication aims to present the results of the archaeological investigations from 2011 - 2020 in terms of architecture. Along with the history of the research, a summarize of the main chronological frames, based on more than 1300 artefacts found during the last nine archaeological expeditions, linked with the functioning of the Building, will be presented.*

**Key words:** *Lyutitsa, Marble city, Medieval Archaeology, Constantine VII Flavius Porphyrogenitus, urban architecture*

### 1. Introduction

Lyutitsa is located at the most eastern mountain ranges of Rhodope Mountains on about 8 km northwest of the town of Ivaylovgrad neighborhood Ladzha, on 6 km east of the village of Plevun and 1.5 km from the our days abandoned village of Rogozovo. It is situated on a plateau which is a part of the last chain of the mountain hills enters of the Aegean plain. Until 2002, among the inhabitants of the surrounding villages, the area was known as

"Gradishteto". The city remains cover an area of more than 26 decares. Although that the fortress walls and part of the battle towers still rise to a height of more than 10 meters, unfortunately its urban architecture does not have the same destiny. The terrain inside of the fortified territory was levelled in the 16th-19th centuries. Moreover, the monumental buildings dated to the 9th-13th centuries were destroyed by the end of the 13th century. Subsequently, later on the architectural debris were reused in the repair of the 14th century fortification. The Fortress walls and towers are built by predominantly marble stones with processed front side surface soldered with white and in some parts pink mortar. The inside part of the wall's construction is filled with an implektum of crushed stones and white and pink mortar. The abundance of marble raw material is the reason the researchers to call Lyutitsa to called the "Marble City" (Петрунова, Петрунов 2018).

## **2. Research of the building 1**

During the last nine years a monumental Building numb.1 (Fig. 1) has been investigated, situated next to the only registered entrance of west site of the city. Its remains cover an area of 197 sq. m., the thickness of the preserved layer is from 0.10 up to 1.20 m and the walls width is between 0.80 and 0.83 m. The building foundations are dug in and laid on the rock. The construction is made by quarry stones welded with mortar containing small ceramic admixture at some parts the grouting is up to 1 m. The foundations are very roughly grunted, as the plinth is shaped only in the north part of wall D of the central room (Fig. 2). In the center part of the building, the extraction rubbles are stepped shaped and reach a depth of 0.90 m. The rock itself has preserved traces of raw material extraction probably used for the buildings. Later on during the formation of the substructure, the space was filled with gravel and rammed. During the last nine field seasons on the territory of the building more than 1300 artefacts have been discovered. The discovered archaeological material display the rich history and long-standing use of Building 1.

The research of the Large Building situated next to the entrance of the Medieval City started in 2011 when Wall "A" has been found. During the following field seasons Wall "A" was completely uncovered and it was established that its total length was 14.65 m and the average width was up to 0.83 m (Fig. 3). The reason to start research at this place was that through the cleaning of the road, the presence of archeological structures have been found. The initial trench was placed 17 meters east of the Wall in continuation towards the south side of the Gate. On the surface many large stone blocks, slabs and stone debris, building and household ceramics, as well as a stone base for a wooden column were scattered. During the cleaning of this deposited and destroyed subtractions the Wall was uncovered. After the detection of the inner side of the Wall it was established that is with 0.80 m width and its length was traced up to 4 m. At the northern tip of the wall, next to the profile, the inner corner of the building (northeast corner of the room) was revealed. In the soil deposited between the destructions a high concentration of red tiles and archaeological materials mainly dated to the 13th century was found (Петрунова 2011, 450-452).

During 2013 and 2014, walls B1 and B3 were partially registered as well as part of wall B2 and the passage between B1 and B2 have been cleared. The aim was to expand the space



designated as Trench 2 and to trace the walls found in it. The total size of the studied area was approximately 70 m<sup>2</sup>, reaching the mainland rock in the northeast corner. At this stage of research, it could be argued that walls A, Б - east and Б1 - west, belong to the same building, which was most likely built in 10th-11th centuries. Its foundations are very deep and at the point where they reach substrate (the base of the wall) it is visible that they lie down directly on the rock. The Building has undergone at least one reconstruction, which is registered in the area under study and the discovered substructures. Above this level, layers saturated with debris and materials dated to the XII - XIII century were found. It has been certainly established that after the end of the existence of this "Later Building" other corrections were made, the constructions of which differ significantly from the main walls. These are registered in the southeastern and northwestern part of the studied sector (wall В, which belong to XIV century and was dismantled, and in its place in the numbering in 2020 was introduced buttress, structurally connected to wall 31; wall Г, which makes an angle with В2 to the north, it is still preserved. Wall Г is shallowly founded, 0.28 m above the level of the plinth of wall Д (Fig.4) (Петрунова, Василева, Димитрова – Тасева 2014, 501–504; Петрунова, Петрунов 2015, 663-665).

Another important fact is that above the presumed floor level, formed in the second stage of the building's existence, the layer of rubble has a very strong concentration of ceramics and finds. The thickness of the layer is around 0.20 - 0.25 m. The main reason for distinguishing these two stages is based on the discovered coins from the 11th and 13th centuries. Special attention should be paid as well to the open Entrance between walls В - east and В - west, which by end of this excavations season is not completely cleared yet. However the threshold which have been found marks that the floor level of the building is from the 12th-13th century (Петрунова, Петрунов 2015, 663-664). In 2015 and 2016, the layout of the entire building was tracked via trenches, as well as part of the profiles left for check out of the stratigraphy were dismantled. The plan of the building is outlined, and the total built-up area is established on 197 m<sup>2</sup>. (Петрунов, Кирякова, Стоилова 2016, 697-699; Петрунов, Кирякова, Райкова 2017, 566). In 2020, the central part of the Large Building was fully excavated, where Wall И was found, through which a central room was differentiated. Thus way, for six archeological seasons, the Large Building next to the Entrance to Lyutitsa is fully studied, as well as all the cultural deposits in it. The chronology of functioning and the stages of stratification are determined. After the end of the 13th century, the Building was a source of building materials for the repair of the fortress wall in the 14th century, which led to the complete erasure of the entrance walls from the south and the almost complete dismantling of wall И. However, out of the few preserved stones found in place, the space of the Entrance can be outlined, which is around 1.40 m in size. This conclusion is based on the preserved front side stones of wall И and wall Д, which flanked an internal passage of the same dimensions.

From the time being of the construction of the building to its destruction, coins of almost all representatives of the throne of Constantinople were discovered dated back to the beginning of the 10th century to the end of the 12th century. From Constantine VII Flavius Porphyrogenitus (912-963) to Isaac II Angelos (1185-1185), through the coins of the Latin

Empire and the Athenian Duchy and Latin and Bulgarian imitations to the restored Byzantine Empire after 1261 and the Byzantine and Bulgarian rulers at the end of the 10th century the first half of the fourteenth century were found. Here, it should be pointed out that Lyutitsa was mentioned for the first time in the Constantinople diocesan lists from the middle and end of the 9th century (ГИБИ 1961,146–165).

### **3. Conclusion**

The beginning of the construction must be seen in the context of the establishment of the entire urban ensemble or by the beginning of the second decade of the 10th century at latest. On one hand because the walls with a width of 0.83 m are massively dug out in the rock. On the other hand because of the size and the rectangular plan, showing the presence of monumental urban architecture, typical of the spiritual centers of this era display a synchronous character of the building and fortification. In any case, at the beginning of the 10th century the Building was already a fact and the life in it continued almost without interruption until the end of the 13th century. The number of the discovered artifact more than 1,300 objects are indisputable proof of the rich spiritual and material culture of its inhabitants and will be the subject of a future study.

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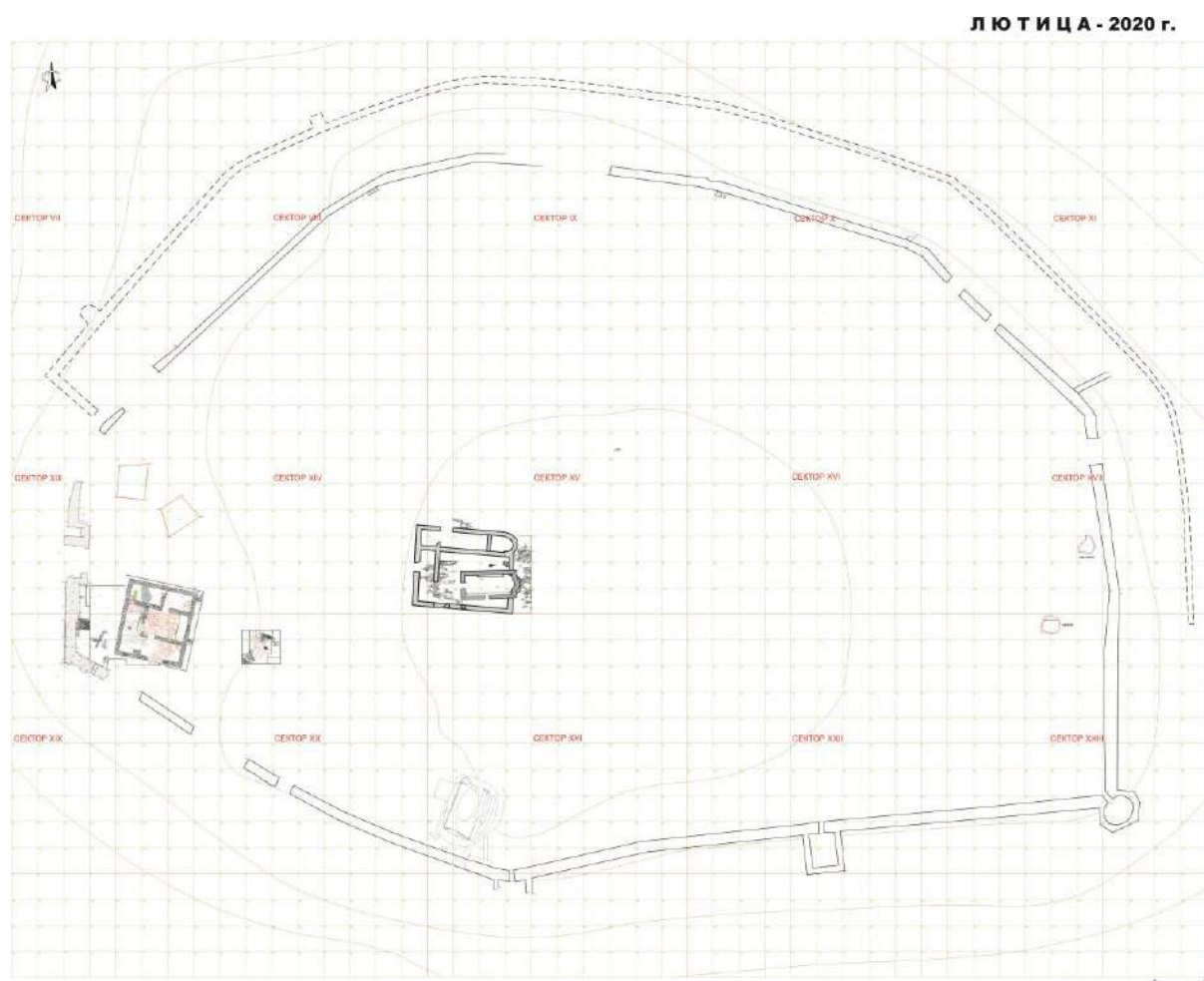


Fig. 1 General plan of the Medieval town of Lyutitsa



Лютица - 2020 г.

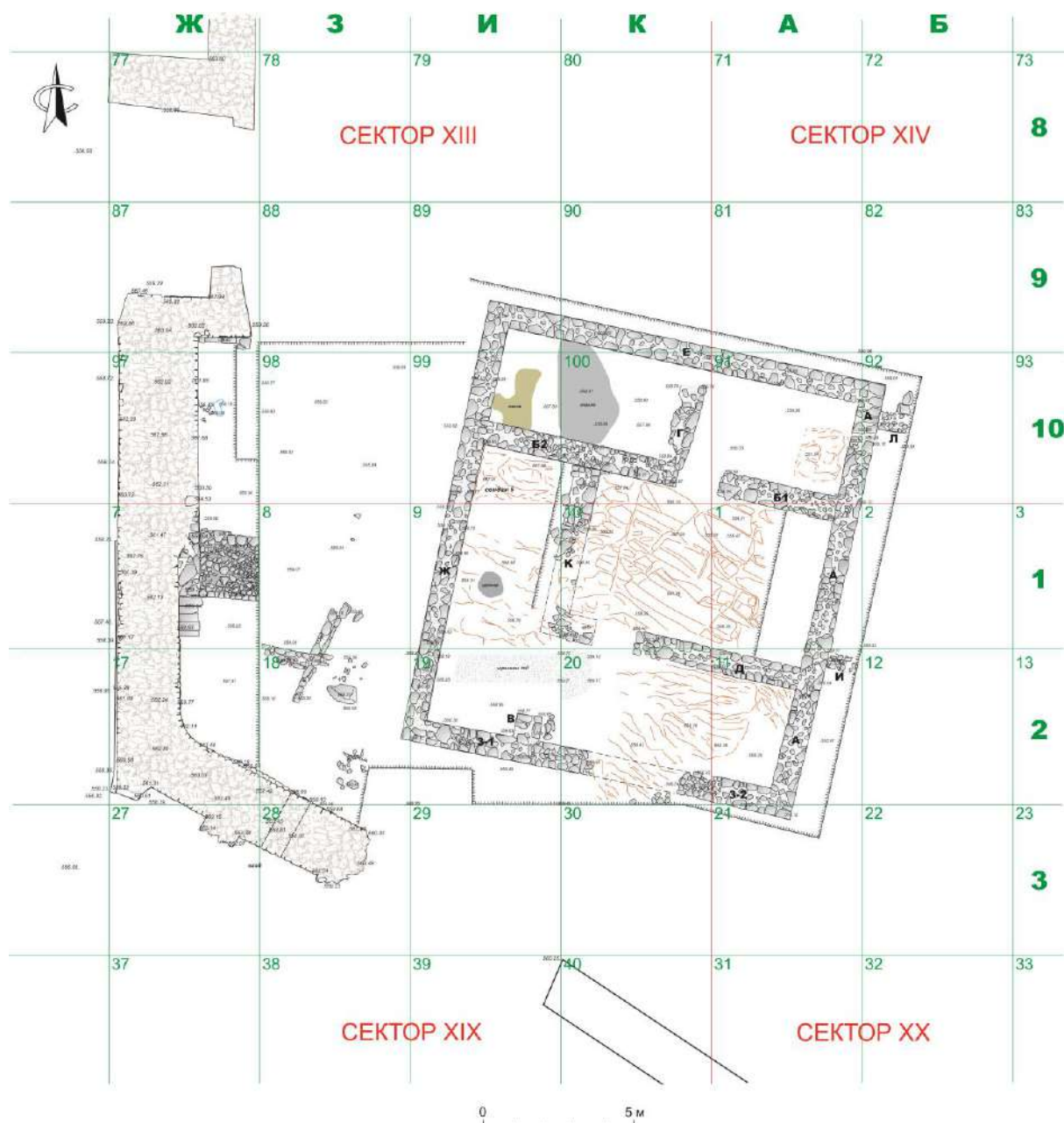


Fig. 2 Plan of Building 1 next to the entrance of the fortification

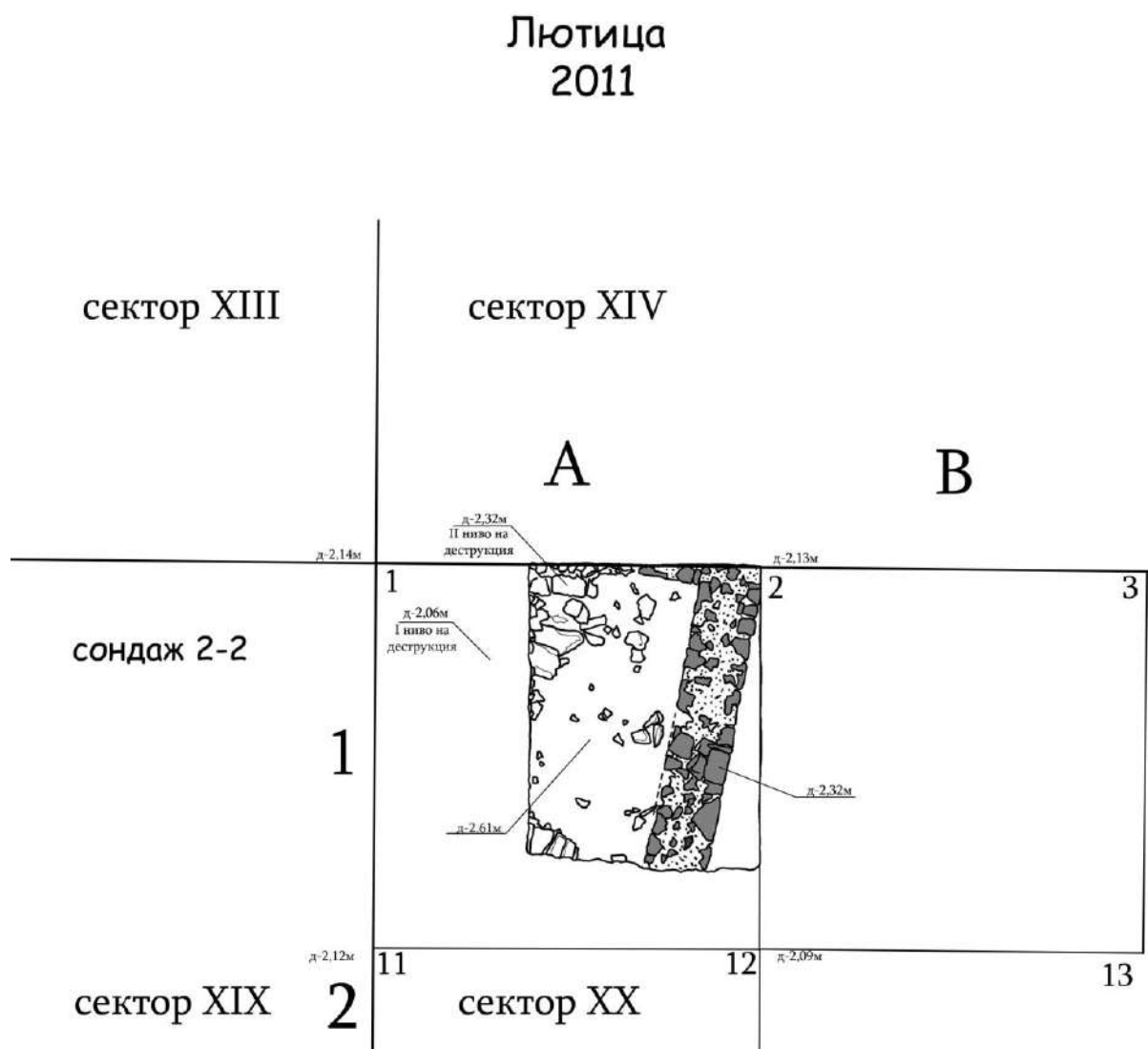


Fig. 3 Initial plan of the construction found in 2011

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Археологически обект ЛЮТИЦА

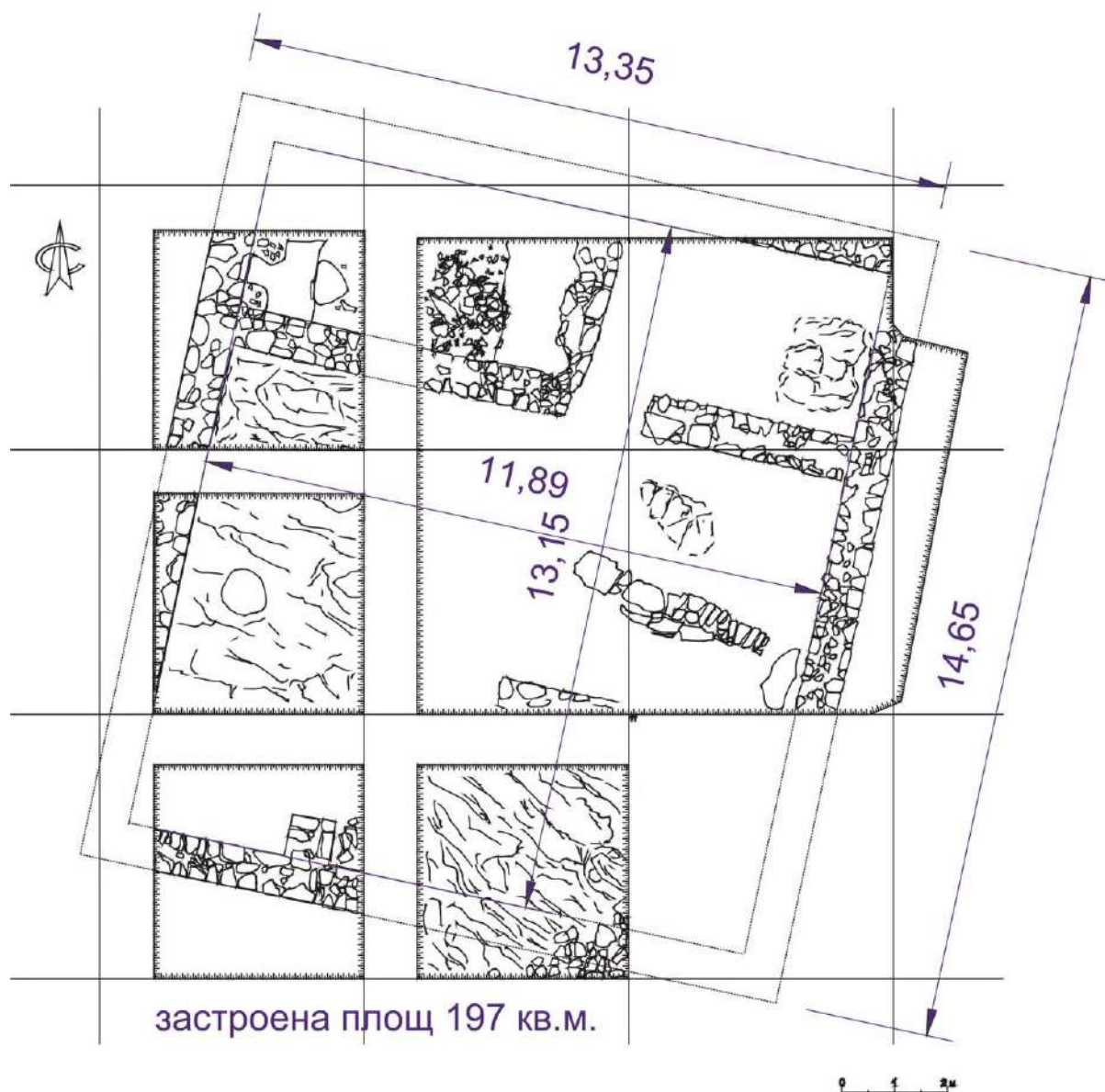


Fig. 5 The architectural remains and dimensions found in 2016.

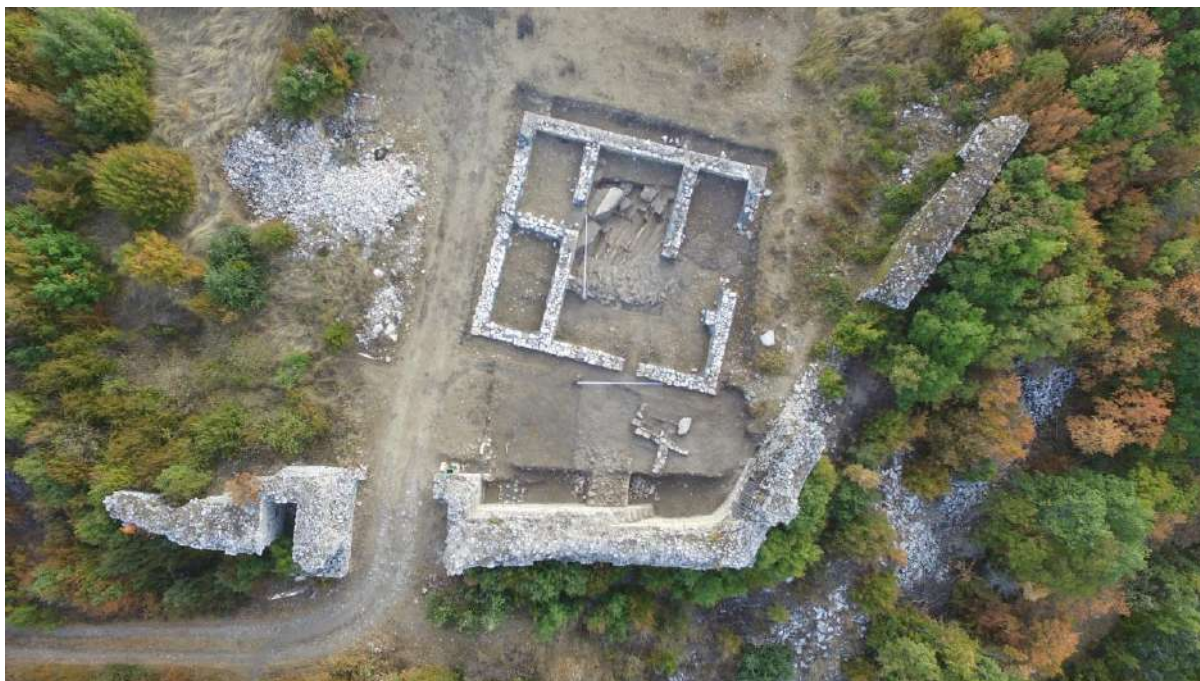


Fig. 6 Aerial photography



Fig. 7 Plinth on wall Д





Fig. 8 The central part of Building 1 and the carvings in the rock.





## THE NEUFERT BUILDING IN WEISSWASSER - AN EXAMPLE OF A BAUHAUS STEEL CONSTRUCTION

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**Abstract:** *In 1919 Walter Gropius founded Bauhaus, one of the most influential design schools of all times. Numerous buildings (and artefacts) still exist today and some of them are considered heritage, submitted to architectural and urban conservation. Within Bauhaus concept, which seem to be initiated already with Fagus factory (designed between 1911 and 1913), there was, inter alia, an emphasis on industrial building design and standardization. In this paper, we revised industrial building complex designed by architect Ernst Neufert in Weißwasser (North Saxony – 140 km south of Berlin) regarding challenges of structural conservation. It consists of six-story warehouse and a single-story loading bay. The warehouse and dispatch centre (which were originally used by glass manufacturer), have both a load-bearing steel frame and a brick infill. The visible steel frame gives a rudiment rhythm and order to the façade. The interior is shaped by pattern defined by steel columns, resting on the beams with intervening strained system ceilings out of stone iron. We discussed the state of this industrial complex, a project of renovation and the form in the context of its relevance for the legacy of its author who is widely known for his book Architects' Data and for legacy of Bauhaus, in general.*

**Key words:** *Neufert, industrial heritage, Bauhaus, steel building, standardization, normative rules*

### 1. Introduction

The industrial complex, warehouse and dispatch centre, in Weißwasser (Fig. 1) was designed by architect Ernst Neufert in the period while he was working on the book Architects' Data for which he is widely known. The complex is located in the country-side of North Saxony, 140 km south of Berlin, in a town named Weißwasser, close to the current border with Poland. The building was designed and built in 1936 for glass manufacturer Vereinigte Lausitzer Glaswerke (VLG, in English: United Lusatia Glassworks). Neufert became the resident architect there in 1935/1936, and in the same year came out the first of many editions of his Architects' Data, which has been published since that time worldwide, translated into numerous languages. This book is de facto accepted as most comprehensive and best known normative book for architects. Neufert was educated

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in Bauhaus, which was apparently of great importance for his professional development and legacy.



**Fig. 1.** The Neufert building in Weißwasser

## **2. The architect Ernst Neufert and normative rules s**

Ernst Neufert (1900-1986) was born in Freyburg, on the Unstrut, near Weimar. He was educated in Baugewerksschule in Weimar (1918-1919) and immediately after in Bauhaus first generation of students (1919-1920). Before that, during his teenage years, Neufert worked as bricklayer apprentice, which seems to be important for his interest in contemporary architectural standardization and normative rules. Neufert came to VLG upon invitation of the former fellow-student from Bauhaus Wilhelm Wagenfeld who became the artistic director of VLG in 1935. During his Weißwasser period, Neufert designed many buildings (various housing, office, and factory buildings) in the town itself and in nearby Tschernitz and Kamenz.

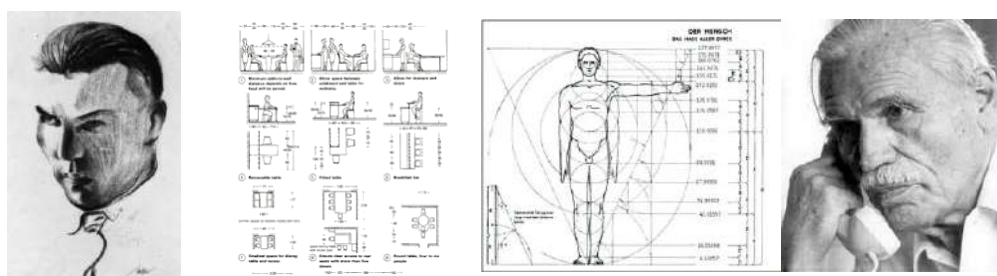


**Fig. 2.** Ernst Neufert as young man (left) author: Gyula Pap, 1930, samples of normative rules from his book Architects' Data (middle) [1], and photo of himself as elderly man (right)

After the Bauhaus was closed, as well as two other schools where he was employed (Bauhochschule, known as “the other Bauhaus”, also in Weimar, where he worked 1926-1928, and private school for art and architecture founded by Johannes Itten in Berlin where he worked in 1933-1934), being a family man, Neufert chose a safe job in the countryside



in those politically turbulent times. Neufert used that time wisely: he worked as architectural practitioner and on shaping research on normative rules in architecture into a book. The book made him famous both in Europe and in USA already after the first edition (Fig. 2). From 1939, Neufert worked with Albert Speer, Hitler's General Building Inspector on typology, standardization and rationalization of Berlin's housing.



**Fig. 3.** Examples of Neufert's work in cooperation with his son Peter [2]

In the last phase of his career, Ernst Neufert was professor of architecture at the Technische Hochschule Darmstadt (1945-1965) and, at the same time, he kept working as freelance architecture practitioner. In this period, with his son Peter he designed several industrial buildings (Fig. 3). In his entire career, industrial building design, standardization and normative rules remained main foci.

### 3. Impact of Bauhaus

Bauhaus was founded the same year as German Workers' Party (soon after renamed into National Socialist Party) i.e. they were both established immediately after the World War I, the biggest and the most devastating war the World had seen by then. Germany was heavily impoverished, and in response to lack of resources, famine, un-der employment, in need to meet basic needs of population, minimum standards be-came one of imperatives of Bauhaus. A year after the National Socialist came to power, in 1933 they established one-party rule by changing Weimar Constitution, the Constitution of German Reich according to which Germany was democratic parliamentary republic, and that marked the end of what had name Weimar Republic (1919-1933). Bauhaus was not at any point isolated from those events, not the least for being originally located in Weimar, which, has also an obvious symbolic connotation.



**Fig. 4.** AEG Turbine Factory in Berlin by Peter Behrens studio (left) Doris Antony, Berlin, Germany; Fagus pencil case Factory in Alfeld on the Leine by W. Gropius/A. Meyer in (middle) Carsten Janssen [3], [4]: and Van Nellefabriek in Rotterdam by Johannes-Andreas Brinkman and Leendert-Cornelis van der Vlugt.

Gropius and his partner in architectural studio Adolf Meyer were apprentices of Peter Behrens, and it is generally considered that “all started” from the design for AEG Turbine factory (Fig. 4). Industrial building design was among main foci in Bauhaus. When Walter Gropius changed location of Bauhaus in 1925, he chose Dessau which, by coincidence or not, had reputation of industrial town. While Bauhaus was led by Hannes Meyer, a Marxist, in 1928-1930, social ideals including minimum standards and affordable housing became prevailingly important over aesthetics. However, need for standardization was widely recognized. Ernst May's huge projects in Russia (20 cities in 3 years, the utopian city for 1 million citizens and more) were, de facto, standardization in practice. Standardization of kitchen by Margarete Schutte-Lihotzky (who was part of May's team and friend of Bruno Taut) has had enormous impact on home design ever since. The Dessau period ended in 1932, and for one semester Bauhaus existed in Berlin – situated in an old telephone factory (again a factory) in Berlin-Steglitz, but it was ultimately closed on the August 10th 1933, while directed by Mies van der Rohe who was, by common opinion, mostly interested in aesthetic. It all proves what is commonly known: industrial architecture was highly relevant for Bauhaus. Further, it implicated preferences in materials used for designs, among which metal was often favoured, and there are numerous proofs of that from design of artefacts to architecture. Last, but not least among key impacts of Bauhaus to Neufert's work, due to Bauhaus Neufert was acquainted with many prominent architects and historical figures of his time and he successfully cooperated with number of them: Walter Gropius, Adolf Meyer, (1919-1920, 1921-1925), Ricardo Magdalena (1920-1921), Paul Klee, Wassillie Kandinsky, Georg Muehe, Otto Bartning (1926), Frenk Loyd Wright (1936), Albert Speer (1939-1944) and more. Neufert is said to have met Antonio Gaudi in 1921, during his sabbatical in Spain as well as to be impressed by his approach to architecture. It indicates that Neufert was driven by creative individuals and had interest in diversity.

#### **4. The Industrial complex**

##### **4.1 Specifications**

The building complex consists of a six-story warehouse and a single-story loading bay. The building construction (Fig. 5) is essentially formed by a steel structure and brick-work infills (12 cm wall thickness) in the outer walls, which is still largely original to-day, as well as between the cross bars exciting steel stone ceilings. The steel structure is composed of columns arranged in a regular pattern (HEA profiles) and floor-to-ceiling hinges connected to them as floor-to-floor units. The individual ceiling panel of the steel stone ceilings are resting on the lower flange of the steel cross bars. The warehouse is accessed by a staircase and two larger freight elevators (in practice out of service) and has about 700 m<sup>2</sup> of storage space per floor.

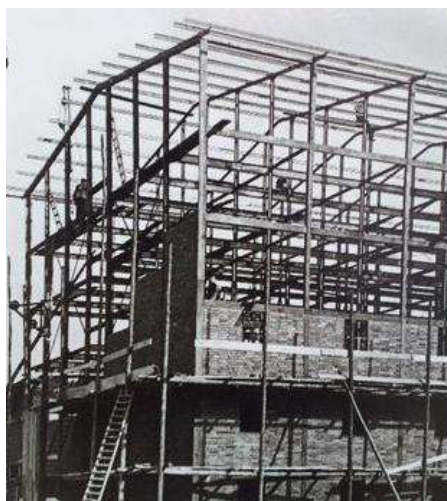


**Fig. 5.** Warehouse during assembly [5]

The functionality of the supporting steel skeleton is not only visible in the façade, but also serves as a basic system of order. The interior is characterized by a steel column grid, on which beams with interlocked system ceilings ("stone-iron ceilings") rest with shingled brick formwork. The steel columns are tapered - for reasons of rationality - visible on the upper floors the steel frames of the east and west façades were hung so that the building could actually be "pulled up" like a construction kit. The loading hall, which adjoins the warehouse at right angles, has a direct functional connection and takes up the formal idiom of the warehouse. Noteworthy is the off-center column, which increases the functionality of the hall and which found its way into Neufert's Architects Data. From the loading hall there is direct access to the two freight elevators, here are loading ramps for the truck delivery of glassware and a loading ramp for shipping by rail.

## 4.2 Current state

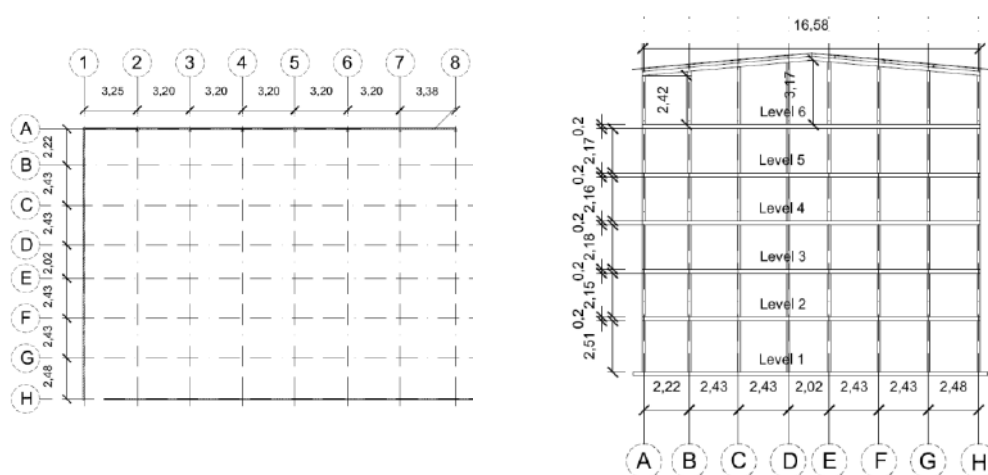


**Fig. 6.** Ground floor and cross-section [6]

The building was erected as a warehouse in the years 1935 to 1937. It was partly damaged in 1945. Those war damages affected the building in axes 1-5 / C-H in levels 3-6 (Fig. 6) and they were repaired in 1954 /1955. The outer brick walls and the ceiling,



including the steel structure, were completely rebuilt. Being vacant for years and neglected by changing owners coupled with the widespread image of "obsolete" (simple storage without purpose, in addition to bad condition). The current state of the construction requires urgent action: the roof and the roof drainage have been mostly destroyed which leads to large-scale drenching on all floors. Part of the vertical glass window band on the south facade has collapsed, many windows on the east and west front are broken. Some props also lack the force-fit, so that now there is a risk that parts of the building go out.

#### 4.3 Renovation

The destroyed roof and the windows have to be fixed urgently to keep moisture and other weather conditions outside. After fixing of the outer cover, the whole steel skeleton structure is to be checked. A damage from corrosion is largely visibly (Fig. 7) and it will continue to progress unless counter action is provided. Most columns and bars will need a surface blasting with new coating.



**Fig. 7.** Outer steel skeleton with corrosion damage



**Fig. 8.** Column in the building with corrosion damage and its rehabilitation

The connections, which have been already damaged due to the expansion of corrosion, should be completely rebuilt (Fig. 8). Those damaged connections already cause many problems in other parts of the building like the façade. The bolted joints are no

longer effective and will jeopardize the carrying capacity. The bolts will get additionally to the planed tensile and shear forces also bending forces which weakens the connection and changes the original transfer of loads. To refurbish the inner columns joints, it is not possible to have an outer scaffold construction. For this reason, an inner auxiliary construction is necessary and should be arranged among each other in every floor to support the ceiling. To take the load off the columns, a counterweight presses the ceiling beams upwards is required, while fixing the joints (Fig. 9).

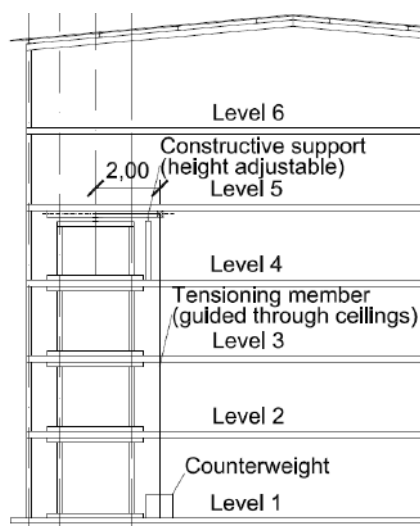


**Fig. 9.** Cross section with counterweight presses and supports during fixing

The preservation requires comprehensive intervention and finding new purpose for the building. Providing new purpose, which could be most efficient preservation, is action constrained by very small story height (2.15m) and small distance between columns (2.02-2.43m), according to contemporary standards. Windows are immediately under the ceiling, and parapet is high and in disproportion of interior, serving the rhythm of the façade. Some efforts for using building with new purpose were made in the past. Behind the building, a swimming pool and other details were added, in attempt to change the industrial complex into cultural hub.

### **5. Outlook, heritage value and preservation prospects**

Currently, an association - Neufert-Bau Weißwasser e.V. - is collecting funds for refurbishment, therefore the future use is not clear yet. There are several evident problems which make reuse difficult: ceiling is too low for contemporary standards; windows are too small and position of windows seem to be in favour of façade, shaped by means which are as modest as possible. The building itself has primarily historical value because one of the chapters of the epic story of Bauhaus and its students was related to it. A small town Weißwasser in Germany is to be remembered as the place where one of the best known architectural books was written. It certainly has memorial value (regarding history of construction for early contemporary application of modular system), scientific value (for a constant application of normative rules) and more. In ideal case, the building would be reused with original purpose – as a ware-house. It could be beneficial to keep part of the building open for public and present it as museum. It could bring additional means which could cover at least some maintenance cost. Such model of management (which was promoted by its Canadian authors under the name Economuseum) may give good results in a countryside as such, far from areas which grossly attract tourists.

## 6. Discussion

Bauhaus gave enormous contribution to analyses of minimum standards in architectural design from its beginning. Under Hannes Meyer, social aspects, standardization and affordable housing became even more important. Concerns of social impact of how we built already had history in Germany, which can easily be traced (at least) to Deutscher Werkbund and Herman Muthesius. Having that in mind, Neufert had significant share in Bauhaus legacy. In the Weißwasser phase, it seems that Ernst Neufert experimented with application of his ergonomic normative rules and possibilities of different material. The design of the factory proves dedication to minimum standards as well as to Bauhaus design principles within low budget and using simple materials, which were at disposal locally. Besides brick and concrete, Neufert used metal and glass. His design keeps promoting genuine values and preferences thought at Bauhaus. We can easily recognize order, rationality, outlook determined by structural elements, respect for functionality, use of materials suitable for industrial production etc. Generalizing from historical distance, one can notice two streams of architects within Bauhaus according to their interest in normative rules vs. aesthetic, and Neufert definitely belongs among architects who rely on standardization and typology in architecture. However, compared to some other architects of the 20th c, Neufert is more into standardization than typology, and between standardization and optimization, more into the second. Neufert's approach brings greatness by modesty; he does not seem to be imposing or overambitious, but rather comprehensive, practical and in-to optimization. It seems that in his case, normative rules are there to assist and serve, not to lead or constrain the individuality, or to falsely simplify architectural design. As consequence, Neufert's own designs differ among each other and reveal personal development and permanent search, which deserves full respect from practitioners, in addition to gratitude from countless generations of architectural students who used his Architects' Data to enter the world of Architecture. Therefore, the didactic value of Architects' Data for architectural students can be hardly overestimated, or its value as reference for practitioners.

## 7. Conclusion

The Neufert building in Weißwasser is an industrial building in the formal language of the "classical" modern response of the New Objectivity of the 1930s. It was planned and built as a central warehouse and logistics centre of the "Vereinigte Lausitzer Glaswerke" (VLG), for the storage of glass products, among other designs by Wilhelm Wagenfeld - famous for the Wagenfeld lamp, as well as to the envelope production warehouse rail. Bauhaus, was founded in 1919 by Walter Gropius in Weimar as art and craft school. Today it is celebrated worldwide as alma mater of European 20th c avant-garde design. Many Bauhaus designs, from households to industrial buildings, are considered contemporary design classics, and such achievement of Bauhaus is most surprising considering its short existence (all three phases of its existence together lasted less than 15 years). Neufert gave exceptional contribution to dissemination of knowledge regarding minimal standards based on architectural analyses. His Architects' Data set reference for all normative rules in 20c. His industrial complex design in Weißwasser proves that his written works and his architectural design were consistent. Therefore, we conclude the world of architecture would be impoverished with-out legacy of Ernst Neufert and the effort should have made to preserve his industrial complex in Weißwasser as standing testimony of that.

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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.**





## UPGRADING BY TIES OF HISTORIC RC BUILDINGS UNDER SEISMIC SEQUENCES CONSIDERING AXIAL FORCES EFFECTS

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### **Abstract:**

*The seismic strengthening by using cable elements (tension-ties) of existing historic industrial reinforced concrete (RC) structures, which are elements of the recent Cultural Heritage, is numerically investigated. Special attention is given to compute the response under environmental effects and seismic sequences by considering the effects of alternating axial forces. 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, the case of multi-storey old RC frames under multiple earthquakes is treated numerically in a typical example.*

**Key words:** Historic RC Structures, Strengthening by Cable-braces, Multiple Earthquakes, Alternating Axial Forces.

### **1. Introduction**

The recent Cultural Heritage has as elements, among others, old existing industrial reinforced concrete (RC) structures [1]. These historic RC structures are often subjected to non-usual extremal actions, e.g. seismic, environmental etc., and so they appear a significant strength degradation and damages. To overcome such strength degradation effects, various repairing and strengthening procedures can be used for the seismic upgrading of existing RC buildings [1-3].

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For the seismic analysis of such historic RC structures, an estimation of the effective flexural stiffness must be realized taking into account cracking on columns and beams [1-4]. Moreover, the effects of alternating axial forces on the available cyclic shear strength must be taken into account.

Indeed, due to alternating environmental actions, the response of reinforced concrete (RC) framed structures can appear variations in axial forces. For example, in multi-floor RC buildings, the axial loads of the columns are continuously changing during a seismic excitation. These variations in axial force can influence strength, stiffness, bending and shear behaviour, and deformation capacity of RC columns [4-6]. In low-rise RC buildings, that do not have additional vertical loads, the above changing axial forces do not always remain as compressive ones during a seismic excitation. So, axial forces can be alternated from compressive to tensile ones, meaning that their sign switches from negative to positive ones.

Further, concerning the seismic upgrading methods, cable-like members (ties) can be used as a first strengthening and repairing procedure [5-7]. 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 [8]. As concerns the numerical treatment, non-convex optimization algorithms are generally required [7-10].

The present study deals with a computational approach for the seismic analysis of historic RC frame-buildings, which have to be strengthened by cable elements and to be subjected to seismic sequences [11]. Special attention is given for the estimation of the effects of alternating axial forces on the response. Finally, an application it is presented for a simple typical example of a two-bay two-story old industrial RC frame strengthened by bracing ties under multiple earthquakes.

## 2. Method of analysis

### 2.1. Problem formulation and numerical treatment

Effective numerical methods for the problem formulation and treatment have been presented recently, see [7]. Briefly, the RC frame structural system is discretized in space by using finite elements. 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. All these characteristics can be expressed mathematically by non-convex relations of the general form:

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

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 [8].

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

$$M \Delta u + C \Delta \dot{u} + (K_T + G) \Delta u = -M \Delta u_g + T \Delta s \quad (2)$$

Here  $u$  and  $f$  are the displacement and the loading forces time dependent vectors, respectively. The damping and tangent-stiffness terms,  $C$  and  $K_T(u)$ , respectively, concern



the general non-linear case. By  $\mathbf{s}$  is denoted the cable elements stress vector and  $\mathbf{T}$  is a transformation matrix.  $\mathbf{G}$  is the geometric stiffness matrix, by which P-Delta effects are taken into account. Dots over symbols denote derivatives with respect to time.

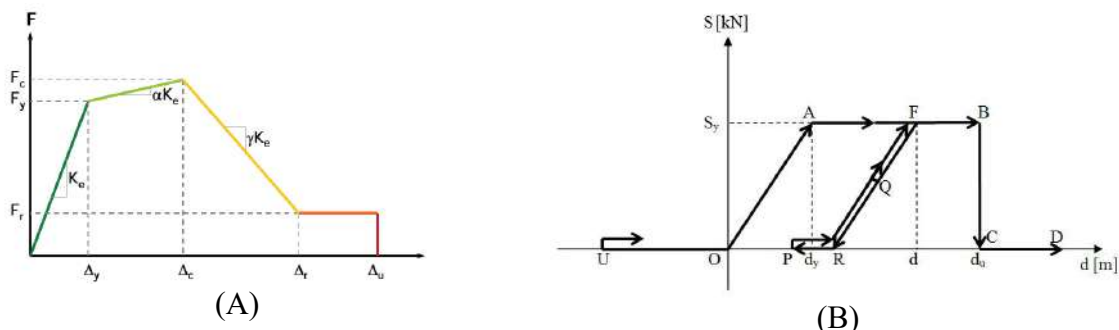


**Fig. 1.** The constitutive laws: (A) generalized force- generalized displacement piecewise linearized constitutive diagram (backbone), (B) unilateral behavior of the cable-elements.

The system of the above relations (1)-(2), combined with the initial conditions, provide the problem formulation, where, for given  $\mathbf{f}$  and/or  $\mathbf{x}_g$ , the vectors  $\mathbf{u}$  and  $\mathbf{s}$  have to be computed. Relevant computational approaches are described in [7-10]. After piecewise linearization of the above constitutive relations (2.1), in each time-step  $\Delta t$  a relevant non-convex linear complementarity problem of the following matrix form is eventually solved :

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

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

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

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 [7,11,13]. After Park/Ang [13], the *local* damage index  $DI_L$  is computed by the following relation:

$$DI_L = \frac{\mu_m}{\mu_u} + \frac{\beta}{F_y d_u} E_T \quad (4)$$

where:  $\mu_m$  is the maximum ductility attained during the load history,  $\mu_u$  the ultimate ductility capacity of the section or element,  $\beta$  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:

$$DI_G = \frac{\sum_{i=1}^n DI_{Li} E_i}{\sum_{i=1}^n E_i} \quad (5)$$

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.

## 2.2. Consideration of the effects of alternating axial forces

As mentioned in Introduction, during a seismic excitation, axial forces in columns and beams are variable, and in some cases can be alternated from compressive to tensile ones. These axial force variations in an RC structural element, e.g. in an RC column, influence strength, stiffness, bending and shear behaviour. So, axial-shear-flexure interaction must be taken into account, resulting e.g. to well-known interaction M-N diagrams of bending moment  $M$  and axial force  $N$ , etc. [14]. For example, considering cracking, environmental and other effects, the effective stiffness  $E_c I_{eff}$  in RC columns can have a range from  $0.90 E_c I_g$  for columns under compression to  $0.20 E_c I_g$  for columns under tension [4,15]. By  $E_c$  is denoted the concrete elasticity modulus and  $I_g$  is the gross inertia moment of the cross-section.

Especially for the available cyclic shear strength  $V_R$ , corresponding to a considered RC member end, the effects of above alternating axial forces can become very significant. These effects are computed according to the following semi-empirical experimental expression given by Eq.(A.12) of Eurocode EC8, see [15,16]:

$$V_R = \frac{1}{\gamma_{el}} \cdot \left[ \frac{(h-x)\lambda_1}{2L_s} + (1-0.05\lambda_2) \cdot \left[ 0.16\lambda_3 (1-0.16\lambda_4) A_c \sqrt{f_{cm}/CF + V_w} \right] \right]. \quad (6)$$

For the meaning of various symbols in the above eq. (6) see [16]. Especially for the parameter  $\lambda_1$  in eq. (6), this is:

$$\lambda_1 = \min(N, 0.55 A_c f_{cm}/CF) \quad (7)$$

So, the parameter  $\lambda_1$  depends mainly on the axial force  $N$ , which is positive for compression, while when the axial force is tensional then  $N$  is taken zero.

## 3. Numerical example

### 3.1. Description of the considered historic RC structural system.

The old industrial reinforced concrete frame F0 of Fig. 2 is considered to be upgraded by ties and subjected to a multiple ground seismic excitation [11,17]. The estimated concrete class is C12/15 and the steel class is S220.

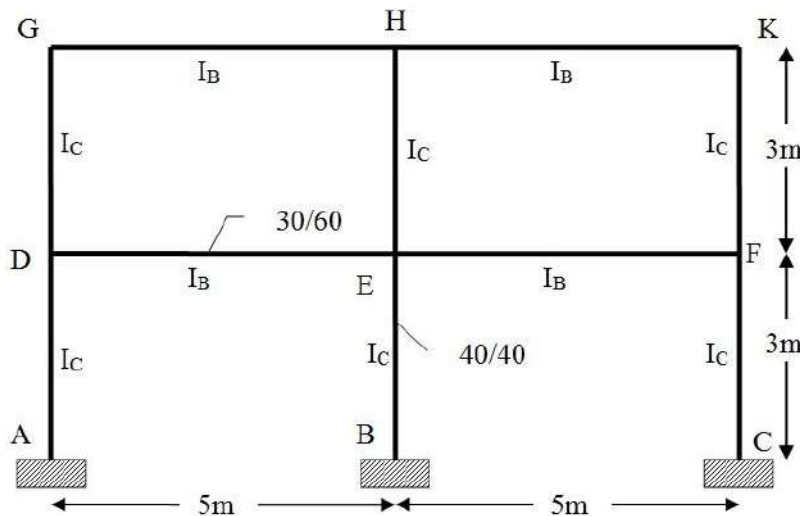


Fig. 2. System F0: the old RC frame without cable-strengthening

Due to various extreme actions (environmental etc.), corrosion and cracking has been taken place, which has caused a strength and stiffness degradation. So, a stiffness reduction due to cracking must be estimated. Taking into account the alternating axial force effects, the range of effective stiffness is considered to be  $(0.25 \div 0.60)E_c I_g$  for the two external columns,  $(0.40 \div 0.80)E_c I_g$  for the internal columns and  $(0.30 \div 0.50)E_c I_g$  for the beams. Using Ruaumoko software [12], the columns and the beams are modeled using prismatic frame elements. Nonlinearity at the two ends of RC members is idealized using one-component plastic hinge models, following the Takeda hysteresis rule [11,12]. Interaction curves (M-N) for the critical cross-sections of the examined RC frame have been computed.

The frame-system F0 of Fig. 2 was initially without cable-bracings. After the seismic assessment, it was decided the frame F0 to be strengthened by ties. The X-cable-bracings system, shown in Fig. 3, has been proposed as the optimal one in order the frame F0 to be seismically upgraded. The system of the frame with the X-bracing diagonal cable-tension only elements shown in Figure 3 is denoted as system F4.

The cable constitutive law, concerning the unilateral (slackness), hysteretic, fracturing, unloading-reloading etc. behavior, is depicted in Fig. 1(B). The cable elements have a cross-sectional area  $F_c = 18 \text{ cm}^2$  and they are of steel class S220 with elasticity modulus  $E_c = 200 \text{ GPa}$ .

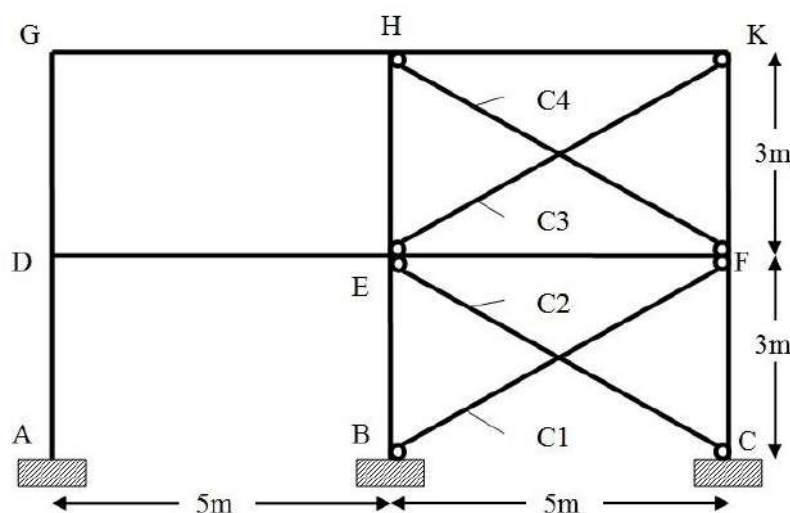


Fig. 3. System F4: the old RC frame cable-strengthened with X-bracings

### 3.2. Earthquakes Sequences Input

A list of multiple earthquakes, downloaded from the strong motion database of the Pacific Earthquake Engineering Research (PEER) Center [17], is appeared in Table 2.

### 3.3. Representative results

After application of the herein proposed computational approach, some representative results are shown in Table 3. These results concern the Coalinga case of the seismic sequence of Table 2.



**Table 2.** Multiple earthquakes data

| No | Seismic sequence | Date (Time)        | Magnitude (M <sub>L</sub> ) | Recorded PGA(g) | Normalized PGA(g) |
|----|------------------|--------------------|-----------------------------|-----------------|-------------------|
| 1  | Coalinga         | 1983/07/22 (02:39) | 6.0                         | 0.605           | 0.165             |
|    |                  | 1983/07/25 (22:31) | 5.3                         | 0.733           | 0.200             |
| 2  | Imperial Valley  | 1979/10/15 (23:16) | 6.6                         | 0.221           | 0.200             |
|    |                  | 1979/10/15 (23:19) | 5.2                         | 0.211           | 0.191             |
| 3  | Whittier Narrows | 1987/10/01 (14:42) | 5.9                         | 0.204           | 0.192             |
|    |                  | 1987/10/04 (10:59) | 5.3                         | 0.212           | 0.200             |

**Table 2.** Representative response quantities for the frame-systems F0 and F4

| FRAMES | EVENTS                                   | DI <sub>G</sub> | DI <sub>L</sub> | u <sub>top</sub> [10 <sup>2</sup> cm] |
|--------|------------------------------------------|-----------------|-----------------|---------------------------------------|
| (0)    | (1)                                      | (2)             | (3)             | (4)                                   |
| F0     | Event E <sub>1</sub>                     | 0.134           | 0.179           | 2.227                                 |
|        | Event E <sub>2</sub>                     | 0.301           | 0.474           | 3.398                                 |
|        | Event (E <sub>1</sub> + E <sub>2</sub> ) | 0.334           | 0.481           | 3.410                                 |
| F4     | Event E <sub>1</sub>                     | 0.068           | 0.007           | 1.126                                 |
|        | Event E <sub>2</sub>                     | 0.097           | 0.136           | 1.447                                 |
|        | Event (E <sub>1</sub> + E <sub>2</sub> ) | 0.108           | 0.154           | 1.471                                 |

In column (1) of the Table 2, the Event E<sub>1</sub> corresponds to Coalinga seismic event of 0.605g PGA, and Event E<sub>2</sub> to 0.733g PGA, ( $g=9.81\text{m/sec}^2$ ). The sequence of events E<sub>1</sub> and E<sub>2</sub> is denoted as Event (E<sub>1</sub>+ E<sub>2</sub>). By u<sub>top</sub> in column (4) is denoted the absolutely maximum horizontal roof displacement.

As the values in the Table 2 show, multiple earthquakes generally increase response quantities, especially the damage indices. On the other hand, the strengthening of the frame F0 by X-bracings improves the response behaviour.

#### 4. Concluding remarks

A numerical approach has been presented, which takes into account the effects of alternating axial forces on the seismic analysis of historic RC structures. These structures are seismically upgraded by using cable-bracings. Emphasis is given to their response under seismic sequences. As the results of a numerical example have shown, the strengthening by cable-bracings can reduce damage indices. Thus, the proposed methodology can be used for the effective investigation of the inelastic seismic behaviour of existing old RC building systems under seismic sequences.

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## INCREASING THE LOAD BEARING CAPACITY OF HISTORICALLY VALUABLE WOODEN BEAM CEILINGS UNSING THE EXAMPLE OF THE POLYCHROME CEILINGS OF PALACE SZTYNORT

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**Abstract:** *For centuries, wooden beam ceilings have been a typical and widely-used wooden ceiling construction. In order to guarantee new social usage requirements and the preservation of the historical substance, a methodology suitable for historic monuments was developed in 2017 at the Chair of Structural Design at the TU Dresden for the strengthening of wide-span wooden beam ceilings by means of additional cross-sectional expansion using structural bonding as a contemporary timber connection technique. As a result of these investigations, the cultural monument of Palace Sztytnort in Sztytnort, Republic of Poland (formerly East Prussia) is getting its significant and extensively polychrome painted wooden beam ceilings back, which are then also suitable for bearing greater loads from future usage requirements. The protection and strengthening of the wooden beam ceilings of Palace Sztytnort is an innovative combination of new engineering science, structural approaches and high social requirements for modern monument conservation to the benefit of the continuing existence of cultural heritage.*

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

### 1. Introduction

For centuries, wooden beam ceilings have been a typical and widely-used wooden ceiling construction. Wooden beam ceilings with large spans in particular rarely meet today's construction standards [3] and must be strengthened for their new use. Such strengthening is usually carried out by supplementing the typically rectangular beam cross-section with an added cross-sectional element. The connection of cross-sectional elements is usually achieved by means of pin-shaped metal fasteners which have the disadvantage of a flexible bond (i.e. slippage of nails and screw). On the other hand, the application of a structural bond between existing cross section and strengthening enables a firmly bonded, rigid connection. In building practice there are only a few examples of the use of structural bonding for structural additions to an existing building, but with an additional connection using pin-shaped fasteners [11]. Up to now, the application of structural bonding in existing buildings has not been possible because the production of a sufficiently precise bonding surface on old beams in-situ was only possible with a high technological effort, and the undefined environmental conditions on the construction site had a negative impact on the quality of bonding and load-bearing capacity.

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As part of scientific investigations [5] in 2017, the Chair of Structural Design at the Technical University of Dresden investigated a method for the strengthening of wide-span wooden beam ceilings in line with the requirements of monument conservation using additive cross-section expansions with the application of structural bonding under conditions similar to those on a building site and digital production technology to produce precisely-fitting compensation plank.

This method was examined with reference to the specific application example at Palace Sztynort (Fig. 1). The baroque wooden beam ceilings of the cultural monument are characterised by their unusually large spans of up to 11.00 m and extensive polychrome painting (Fig. 2).

New usage requirements are being defined with the renovation of Palace Sztynort by its new owner, the Polish-German Foundation [2]. The primary objectives of monument conservation are to preserve the original substance and to make it accessible to the public [6]. Public-use places a higher load on the ceiling construction, which in turn cannot be met by the historical, repaired ceiling structure. The span widths of 11.00 metres in the wooden beam ceilings are a constructive exception and require structural-efficient strengthening to meet the new requirements of public-use. In addition the basic load situation should not be changed if the existing foundations are to be retained [4]. In this case, only strengthening of the wooden beam ceilings with a low deadweight and with high requirements for the preservation of historical monuments can be applied [5].

## 2. Presentation of the site and heritage value

Palace Sztynort, as former manor house of the von Lehndorff family, is an important cultural monument of German-Polish history (Fig. 1). At the moment Palace Sztynort is a temporary construction site (Fig. 3) of the German-Polish Foundation for the Preservation of Cultural Monuments and the foundation Polsko-Niemiecka Fundacja Ochrony Zabytków Kultury, which has the role of proprietor. Geographically, Sztynort lies far to the east in the Republic of Poland, in the Warmian-Masurian voivodeship, right in the middle of the Masurian Lake District.



**Fig. 1** View of Palace Sztynort Source: Weichelt (2016)

The baroque core of the manor house was built in the year 1689 on the foundations of a previous building [1]. Extensive structural modifications in the 19th century expanded the basic baroque construction with two late classicist wings, and subsequently with 3 towers. The full integrity of the original baroque interiors in traditional Masurian craftsmanship is regionally as well as nationally unique. The polychrome design of the wooden beam

ceilings with ceiling boards between the beams reflects this tradition (Fig. 2). The polychromy comprised of tendrils and strings of fruits was produced by a painter from Rastenburg (today Kętrzyn), who is not listed by name in the historical building documents [1]. The still existing beams with its polychromy are unique in its size, originality and completeness and can complete with the original ceiling boards. As a site of resistance to National Socialism, Palace Sztynort attained great historical significance. In the year 1941 the close proximity to the Fuehrer's headquarters in Rastenburg lead to the seizure of the west wing of the manor by the minister of foreign affairs, Joachim von Ribbentrop. While it was occupied by von Ribbentrop, the von Lehndorff family remained in the manor, confining themselves to using only the east wing and parts of the core building. Heinrich Graf von Lehndorff, the owner of Palace Sztynort at the time, joined the resistance against National Socialism in 1941. Unnoticed by Joachim von Ribbentrop, the palace developed into an important location of information exchange for the military resistance. The fact that forces hostile to one another lived together under one roof in Palace Sztynort is unprecedented. This did not prevent Heinrich Graf von Lehndorff from playing an important role in the planning of the assassination attempt carried out against Hitler on July 20, 1944 in the nearby Wolfsschanze. But he paid for this attempt with his life. The historical value of Palace Sztynort as site of resistance to National Socialism and as home and domain of Heinrich Graf von Lehndorff is a unique feature of the monument with importance for world history.



**Fig. 2** View of polychrome ceilings, situation 2004  
Source: private archive: B. Kulczyńska-Nowak



**Fig. 3** View of polychrome ceilings, situation 2017, source: Meyer

### 3. Concept of strengthening

#### 3.1. Structural implementation

The idea for structural strengthening of the wooden beams is based on transformation of the load-bearing rectangular cross-section of the historical beam into a T-beam (Fig. 5). In this cross-section expansion, the individual cross-sections are not connected to each other by the conventional, metal connecting elements, but are joined together by structural bonding to form a high-performance composite cross-section. Here the historical wooden beams retain their load-bearing function and are supported in their load-bearing capacity by the wooden expansion. The possibility of transforming the ceiling into a T-beam is provided geometrically since the ceilings used to have a floor construction about 15 cm thick above the beams. The former filling of loose organic material is replaced by a glue laminated timber element (Fig. 4).



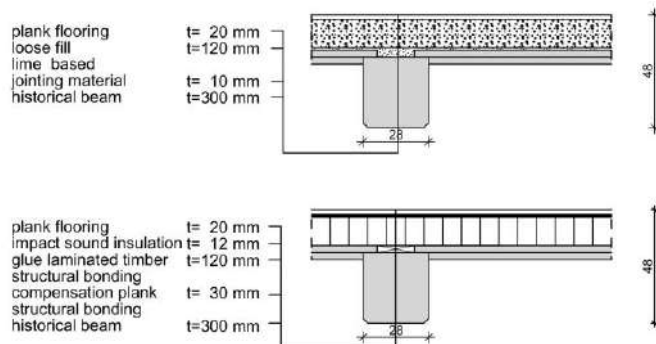


**Fig. 4** Historical ceiling structure as compared to the new ceiling structure, source: Meyer

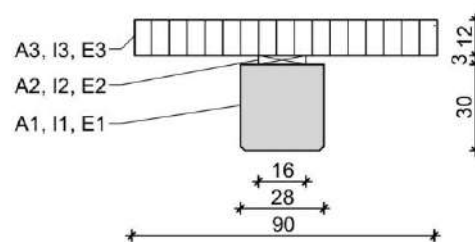


**Fig. 5** Schematic system of cross section, source: Meyer

**Table 1** material properties of the composite cross-section

| nr. | element            | height [m] | width [m] | material | property class | E-Module [N/mm <sup>2</sup> ] |
|-----|--------------------|------------|-----------|----------|----------------|-------------------------------|
| 1   | historical beam    | 0,30       | 0,28      | pine     | C24            | 11000                         |
| 2   | compensation plank | 0,06       | 0,16      | spruce   | C24            | 11000                         |
| 3   | compression zone   | 0,12       | 1,00      | spruce   | GL24h          | 11500                         |

By placing the compensation plank between the existing beam and the glue laminated timber element, it is possible to return the historical ceiling boards to its original position after renovation [7], [8]. In addition to the geometrical requirements, the compensation plank is the essential cross-sectional element for structural bonding.

When applying a glued joint, the difficulty lies in creating a precisely-fitting surface on the existing beams for complete and evenly-thin bonding joint. Mechanical, manual processing involving a guide device is suitable for well-preserved beam surfaces. If, however, local, superficial damage to the wooden beams is found, this must be removed to the load-bearing area. The application of digital production technology provides the possibility for producing a precisely-fitting compensation plank for this beam surface. To obtain production data for the CNC-supported manufacture of this compensation plank, a numerical point cloud of the beam surface is generated using the "Structure from Motion" method (three-dimensional photogrammetry). A negative of the existing beam surface is produced on the compensation plank on the basis of this point cloud. The necessary compression pressure for the structural bonding between the existing beam and compensation plank is generated using screw-press-bonding.

The technological feasibility and the static effectiveness were tested by the production of 3 test beams on a model scale of 1:2 and structural mechanical testing for load-bearing capacity and serviceability.

### 3.2. Structural bonding

As part of the investigation, three different adhesives were initially examined on the basis of their properties. A PUR wood adhesive, a 2-component epoxy resin adhesive and a melamine urea adhesive were available for selection.

The melamine-urea resin adhesive was selected as a suitable adhesive for bonding wood components. This adhesive is approved by the general building supervision for the production of bonded joints with thick bonded joints and offers suitable properties, especially for implementation under construction site conditions. Another essential aspect for the use of the adhesive is decades of experience in the production of glued laminated timber. This experience is important when using it in a heritage site.

Adhesives places high demands on the bonding surface. Care must be taken to ensure a bonding surface free of fibre breaks, loose fibres and dirt, especially when treating existing beams.

The mechanical properties of the adhesive depend on the bonding joint thickness.

Essentially, a change in the thickness of the adhesive influences the load-bearing capacity and the elongation at adhesive failure. The application of the melamine-urea resin adhesive under investigation required an bonding joint thickness of  $t_{f,max} \leq 1.5$  mm. Manufacture of the precisely-fitting compensation plank serves to ensure constant mechanical properties of the adhesive through a constant, thin bonding joint (Fig. 7) as a negative of the surface on the processed existing beam.

The pressure required for the use of the melanin-urea resin adhesive was generated in the test beams using screw-press-bonding [9]. The use of screw-press-bonding (Fig. 6) was investigated for this specific application by selecting special screw and screw spacing [5].

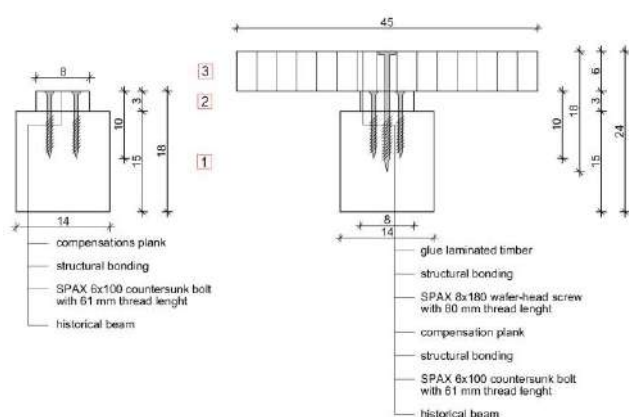


Fig. 6 Screw-press bonding – test beam, source: Meyer

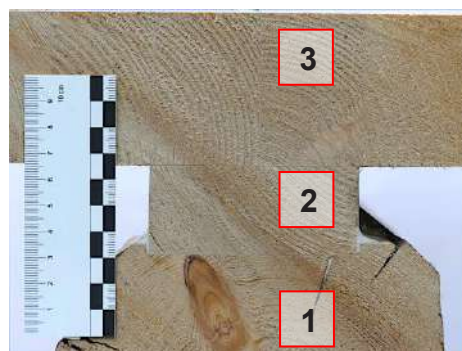


Fig. 7 Cross section-test beam, source: Meyer

The screws are used as a technological aid for securing the position of the precisely-fitting compensation plank and for generating the necessary compressive pressure over the unpainted upper surface of the wooden beam. These screws can be removed after the required minimum pressing time. Production of the test beams integrated the necessary screws for the screw-press-bonding in cross section. This is due to the small adhesive area per bonding joint on a model scale and the amount of adhesive sample available.

### 3.3. Surface measurements – Structure from motion (SfM)

When using structural bonding, adhesive-related requirements for the bonding surface must be observed. One of the essential requirements is the production of an evenly-thin glue joint over the entire length of the ceiling beam. For the production of a precisely fitting compensation plank using CNC-supported production, the existing and prepared adhesive surface must be digitally measured.

The Structure from Motion method (three-dimensional photogrammetry) is suitable for recording the adhesive surface on the existing beam [5] with the result of a numerical point cloud (Fig. 9). Only a calibrated reflex camera and, if necessary, reference standards for optimizing the precision of the photogrammetric images in-situ (Fig. 8) are required for this optical measurement method. In order to use CNC-supported production, the point cloud has to be converted into a facet surface. As a result, a precise compensation plank is produced as a counterpart to the beam surface.

After the point cloud has been cleaned of measurement errors, it can be converted into facet surfaces.

The SfM method is characterized by the simple implementation of the surface detection. Furthermore, no special measuring technology is necessary. The conversion of the photogrammetric recordings into a numerical point cloud is carried out using appropriate software solutions. When taking images for the SfM method, it is important to ensure consistent lighting conditions over the entire length of the beams. This can be done with additional spotlights.

As an alternative to generate a 3D surface model of the beam using the SFM method, a three-dimensional laser scan can be used. The application of this method also enables an exact measurement of the beam surface, but with significantly higher expenditure on technical equipment.



**Fig. 8** Reference standards (black bars with measurement points) for optimizing the SfM method source: Meyer

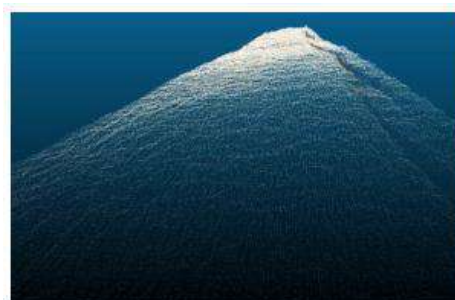


**Fig. 9** Point cloud of the beam surface from the SfM method source: Meyer

### 3.4. Experimental studies

The production of a load-bearing capacity structural bonding under construction site conditions depends on many factors which have not yet been examined in precise detail in this context. Here the environmental conditions affect the exposure time of the adhesive. This refers to the time taken to apply the adhesive, assemble and apply the required pressing pressure. Key environmental factors are room temperature ( $T_{\min}=20^{\circ}\text{C}$ ), relative humidity ( $\text{RH}=65\%$ ) and wood moisture ( $u \leq 15\%$ ).

In addition to the theoretical considerations, the technological methods for producing a load-bearing composite cross-section were tested experimentally.

During the production of the test beams on a scale of 1:2, the environmental conditions were not kept at an optimum by air conditioning. Summer temperatures ( $T=29^{\circ}\text{C}$ ) and high relative humidity ( $\text{RH}=78\%$ ) shortened the open time of the adhesive noticeably, without causing any abnormalities when testing the load-bearing capacity of the test beams.

The structural experimental investigations into the load-bearing behaviour [10] and the vibration behaviour of the composite section were carried out in the Otto-Mohr Laboratory at the TU Dresden (Fig. 11).

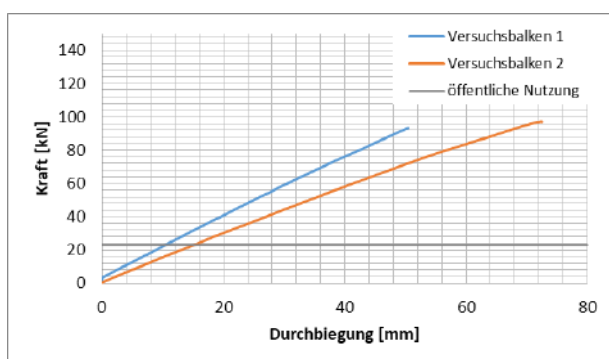


**Fig. 10** Results of the test until failure - 4-point bending test



**Fig. 11** 4-point bending test source: Meyer



The test was carried out on two test beams under a path-controlled load until failure (Fig. 12).

The clear fracture in the tensile zone of the composite section and the achieved average fracture load of 96.2 kN at an average deflection of 65.3 mm prove the load-bearing capacity of the strengthening, as well as the successful application of the technology under investigation for producing a load-bearing, structural bond under construction site conditions (Fig. 10). The full-scale cross section meets the requirements on load-bearing capacity and serviceability for the target load of public use [5].

Vibration was measured on the third test beam using the Syscom MR2002 vibration measurement system for test beams (Fig. 13). Acquisition of the measurement data was carried out by Prof. Dr.-Ing. habil. Peter Ruge from the Institute for Statics and Dynamics, TU Dresden. A natural frequency of the test beam  $f = 24$  Hz was determined as a result of these measurements.



**Fig. 12** 4-point bending test on the test beam  
source: Jäger

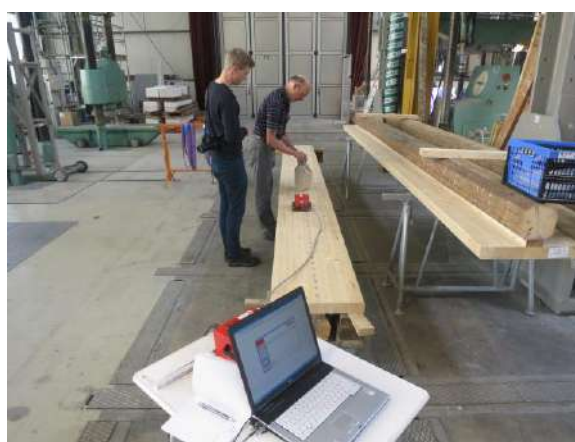


**Fig. 13** Vibration test source: Jäger

#### **4. Conclusion**

The necessary increase in the load-bearing capacity of historical wooden beam ceilings using the example of the ceilings of Palace Sztynort was investigated theoretically and experimentally. The historical rectangular cross section was transformed into a T-beam by using a structural bonding under construction site conditions.

The first application of the production technology and the structural bonding as a contemporary timber connection technique compensate the disadvantageous properties of the conventional, metallic connecting elements.

The new technology for protection and strengthening historical wooden beam ceilings using structural bonding (Fig. 14) has been proven in theory and tested under construction site conditions. With a high degree of prefabrication, the technological effort required on the construction site can be kept to a minimum.

In terms of technology and statics, modern use of wide-span wooden beam ceilings with imposed load up to 5 kN/m<sup>2</sup> is possible.

The effort for the implementation of this innovative renovation method is justified by the high cultural, historical and architectural significance of Palace Sztynort.

As a result of these investigations, the cultural monument of Palace Sztynort is getting its significant and extensively polychrome painted wooden ceilings back, which are then also suitable for bearing greater loads from future usage requirements.

The protection and strengthening of the wooden beam ceilings of Palace Sztynort is an innovative combination of new engineering science, structural approaches and high social requirements for modern monument conservation to the benefit of the continuing of cultural heritage.

These emergency safeguarding measures on the Palace Sztynort cultural monument are made possible to a large extent through assistance from the Federal Government Commissioner for Culture and Media (BKM) [12].

The technical method described here to increase the load-bearing capacity of historical wooden beam ceilings is an effective technique used in timber construction engineering. It is mainly applied in monument preservation and all other areas of timber construction engineering.

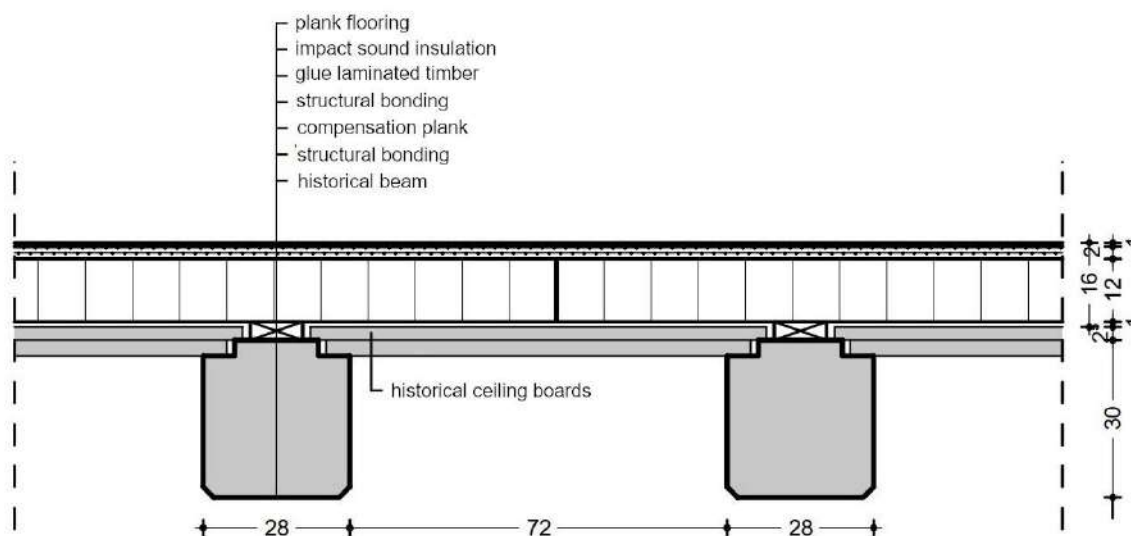


Fig. 14 Strengthening wooden beam ceiling – structural solution, source: Meyer

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## МОСТОВЕТЕ СЛЕД ДЪРВОТО И КАМЪКА И ПРЕДИ СТОМАНАТА И БЕТОНА

Красимир Дойчев<sup>1</sup>  
TERNA SA

### **Abstract:**

*През индустриалната революция мостовете от желязо (чугун и ковано желязо) заменили част от мостовете изпълнявани до този момент с дърво и камък (тухли). Железните мостове създали възможност строителството да се извършва по-евтино, по-бързо и по-дълготрайно, като в същото време премоствали по-големи отвори.*

*Новаторите при железните мостове овладели тънкостите на приложение на желязото и придобили опит, който ги превърнал в търсени специалисти. Били създадени нови и различни мостови конструкции, изградени с по-малко материали и труд.*

*Измислени били и нови технологии за изпълнение на мостовете, някои от които се прилагат и в наши дни.*

### **1. Икономически предпоставки за развитие на мостовото строителство с използването на чугун и ковано желязо**

През 18 век в Европа настъпили социално-икономически промени, довели до трансформация на аграрната и занаятчийска икономика в икономика, доминирана от промишлеността и машинното производство. Първоначално тези процеси възникнали във Великобритания, след което се разпространили в държавите от континентална Европа, а по-късно и в други части на света. Преди този период се наблюдавал един продължителен период на постепенно ускоряване на производствената и търговската дейност, продължил около хилядолетие.

Процесът на бързо ускоряване на промените в икономиката е известен като Индустриална революция. Този икономически феномен съдействал за развитието на науката и технологиите и за социалните и културните промени по целия свят.

Усъвършенстването на методите за добиване и преработка на природните ресурси и използването на нови енергийни източници довели до увеличаването на обемите на производство на чугун и ковано желязо. Потребностите от независими от капризите на природните сили задвижване на производствените машини в предприятията довели до изобретяване на парната машина на Джеймс Уат (*James Watt, 1736-1819*).

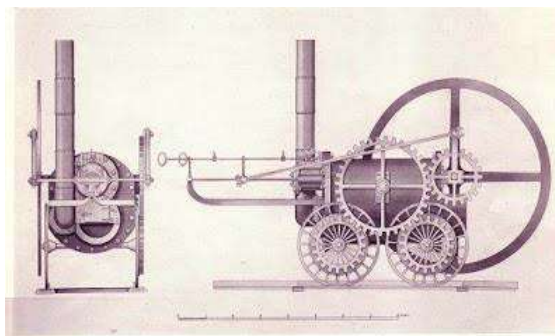
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Парният двигател на Уат бързо намерил приложение в мините, мел-ничното, текстилното и много други производства. През следващите години Уат усъвършенствал двигателя и повишил мощността и възможностите за приложението му. Той създал парната машина с двойно действие, регулатора на парата и планетарната предавка, които направили двигателя удобен за приложение в промишлените предприятия.



**Фиг. 1:** Последният оцелял двигател Boulton & Watt, 1817 г, Нетъртън, железарския завод М W Grazebrook



**Фиг. 2:** Чертеж на демонстрираната в Колбрукдейл парна машина - 1802 г.

За транспорта обаче бил необходим по-мощен и по-лек двигател, който използвал пара под високо налягане. Първите опити за създаване на пътни превозни средства показали, че пътищата лесно се разрушавали от тежките парни машини. Дори чугунените релси се чупели от голямото натоварване. По-голямото тегло на парните машини увеличило изискванията и към товарната носимоспособност на мостовете.

С изграждането на множество нови железопътни линии се увеличила и потребността от строителство на мостове. Ако преди индустриалната революция били изградени един – два или няколко моста в рамките на година, в условията на ускорено изграждане на пътища и нови жп линии възникнала необходимост от строителство на стотици мостови съоръжения за година. Възникнали затруднения с финансирането на строителството. Зиданите конструкции били изпълнявани бавно и с голямо количество работна сила, което повишавало тяхната цена. Не бил за пренебрегване и дефицитът на строителни работници, породен от повишеното търсене на строители поради големия обем на строителството.

Решаването на всички тези проблеми било в приложението на желязото при строителството на мостове, тъй като „железните“ конструкции осигурявали нови възможности.

До IX век в Европа се запазили металургичните техники наследени от античността. През X – XI век континентална Европа повишила добива на железни руди и развила металургията. Възникнали железолейрни на днешната територия на Нидерландия, Белгия, Люксембург, Германия и Франция.

След XII – XIII век започнало използване на водната сила за задвижване на духалата и чуковете в ковашките и леярските цехове. Век по-късно започнала по-широка употреба на каменните въглища като гориво при топенето на руди, което било следствие от унищожението на горите и развитието на каменния въгледобив. Било поставено и началото на механизирания натрошаване и промиване на руди.



През XV век са появили първите доменни пещи<sup>2</sup>. Много бързо те се разпространили във Франция и Англия<sup>3</sup>. Доменните пещи позволявали достигане на температури достатъчно високи за получаване на чугун.

През 1709 година на Ейбрахам Дарби (*Abraham Darby, 1678-1717*) [1] дошла идея, да замени частично въглищата в доменната си пещ с кокс. Вследствие на това той получил чугун с много по-високо качество, тъй като въглищата силно замърсявали с вредни примеси добития метал. Но въпреки подобрието с използването на кокс, добития чугун все още не притежавал желаното качество. Причината за това била сярата в желязната сплав. Сярата се съдържа в кокса и в много по-голяма степен във въглищата. Първата крачка към решаването на този проблем била направена през 1750 година, когато в доменните пещи започнали да използват духала задвижвани от парни двигатели. Температурата в пещите се повишила и това позволило да се добави към рудата и кокса достатъчно варовик, който да елиминира отрицателното влияние на сярата.

Високата якост и възможността за отливане в различни форми правели чугуна все по-предпочитан материал, прилаган при производството на изделия за бита, военното дело, индустрията и строителството. Той обаче имал своите слаби страни. Когато се преминели границите на якост на материала, чугунът изведнъж се разрушавал без предварителни симптоми. В чугунените елементи внезапно се образували пукнатини, които водели до резки счупвания. По тази причина чугунът бил използван в мостовото строителство до появата на начини за по-евтино и масово производство на ковано желязо. До създаването на технология за добив на ковано желязо, то се получавало чрез изковаване на отливки от чугун.

През 1783-1784 година Хенри Корт (*Henry Cort, 1740-1800*) патентовал пудлинг (*puddling*) процеса за рафиниране на чугун. Същността на процеса била, че в реверберационна пещ се стопявали чугунени отливки, произведени в доменни пещи. Устройството на пещта позволявало горенето на въглищата да бъде отделно от стопения метал, поради което той не се замърсявал допълнително с отпадъци от горенето. Пещта се наричала *пудлингова пещ*.

В пудлинговите пещи с помощта на кислород се изгарял излиш-ния въглерод в чугуна. Процесът се ускорявал, чрез добавяне на желязни пръти, които засилвали реакцията на въглерода с кислорода в стопената чугунена маса и с разбъркване на стопения метал. При бъркането на пудльора с лопатката се получавало допълнително окисляване на въглерода.

Разтопеното желязо било изгребвано през специален отвор в пещта на топки (блумове) пластичен метал, който се подавал към чукове за изковаване на съответните изделия или към валцовъчни станове. Процесът бил трудоемък и отнемал много време за производство, а произведеното количество ковано желязо за ден достигало до 1500 килограма на пудльор. Това правело метала дефицитен и скъп.

За да се получат необходимите на пазара пръти, листи, профили, релси и други желязни продукти, желязото се валцувало. Обработката на пластичния метал се

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<sup>2</sup> Първите доменни пещи в Европа се появили в района на Лиеж, Белгия

<sup>3</sup> Първите доменни пещи в Англия са построени през 1491 година.

състояла в това, железните блумове да преминат между специално оформени ролки на валцовъчни машини, които ги формовали и нарязвали.



**Фиг. 3:** Производство на желязо в пудлингова пещ

Усъвършенстването на валцовачните техники може да бъде преписано на пионерските усилия на Хенри Корт, на когото през 1783 година е издаден патент за използването на набраздени ролки за валцуване на железни пръти, като елемент от усъвършенстваната от него пудлингова пещ.

С усъвършенстваното решение валцовъчните машини произвеждали 15 пъти повече продукция на ден отколкото с чук. Хенри Корт бил първия, който комбинирал използването на много от най-добрите характеристики на различни процеси на обработка и оформяне, вече известни по онова време, с което постигнал по-голямо, по-качествено и по-евтино производство. Съвременниците му го наричали "баща на модерното валцуване".

Чугунът и кованото желязо са материалите, дали тласък на строителството на мостове през индустриалната епоха.

## **2. Мостове с използване на традиционни дърводелски и зидарски техники**



**Фиг. 4:** Железният мост - картина на Уилям Уилямс, 1780 (фрагмент)

През 1773 година архитект Томас Причард (*Thomas Farnolls Pritchard, 1723-1777*) дал смела идея. Комбинирайки инженерни познания и нови техники за леење на желязо, той предложил първия в света железен сводов мост. Производството на връхната конструкция било възложено на Ейбрахам Дарби III (*Abraham Darby III, 1750 – 1791*) - собственик на чугунолеярната в Колбрукдейл, след като той предложил най-ниската оферта за производството на чугунената конструкция. През

1777 година започнало производството на елементите за моста. Тогава внезапно Томас Причард починал и Ейбрахам Дарби III се ангажирал да ръководи цялата работа по строителството. Технологиите на изграждане била напълно непозната. За производството на много части от мостовата конструкция било необходимо да се разработят фундаментално нови инженер-ни решения. Мостът бил завършен през юли 1779 година и отворен за движение на 1 януари 1781 година (виж. фиг.4). Това бил първият мост в света построен с чугунени отливки, свързани с болтове от ковано желязо. При строител-ството на моста били използвани типични дърводелски техники, като снаждания зъб и дълб и тип „лястовича опашка“.



**Фиг. 5:** Съвременен изглед на моста Понт и Кафна, 1793



**Фиг.6:** Акведукт над р. Търн, 1796

За изпълнението на следващите мостове трябвало малко време. През 1792-1793 година дърводелецът Уоткин Джордж проектирал и постро-ил моста „Понт и Кафна“ - чугунен мост и акведукт (виж. фиг.5). В горната му част, по релсов път преминавали теглени от коне каруци, доставящи варовик. Отдолу преминавала правоъгълна тръ-ба на акведукт, доставящ вода до предприятията. Мостът бил с отвор 14.2 метра (47 фута). Конструкцията силно наподобявала дървена покривна конструкция, но се отличавала с оригиналност, простота и изящество. Той бил първия железопътен мост служещ за преминаване на леки релсови превозни средства в света и първия комбиниран мост и акведукт.

Акведуктът Холмс, проектиран от Бенджамин Аутрам (*Benjamin Outram, 1764-1805*) и акведуктът над река Търн (*Longdon-on-Tern*) (виж фиг.6), проектиран от Томас Телфорд (*Thomas Telford, 1757 – 1834*), били построени през 1796 година. Това са първите плавателни акведукти. Изпълнени са от чугунени фланцови плочи, свързани с болтове от ковано желязо. Примерите на тези първи мостови конструкции скоро били последвани в Обединеното кралство, където пионерски опити в проектирането и изграждането направили множество строители на мостове.

С овладяване на металур-гичните и металообработващи технологии на континента се появили първите „железни“ мостове, какъвто бил проек-тирания от Джон Баилдън (*John Baildon, 1772 - 1846*) мост Лаасан в Прусия (днес Лазани в Полша), построен през 1796 година (виж фиг.7). Това бил първия европейски пътен чугунен мост.

В началото на 19 век Франция разполагала с триста доменни пещи, които произвеждали доста-тъчно чугун, за да се съизмерва с водещите производители в Европа. Това дало основания на прив-лечения от новите технологии Наполеон Бонапарт (*Napoléon Bonaparte, 1769-1821*) да изиска построяването на метални



мостове. Първият френски чугунен мост бил построен на река Сена, в Париж, между 1801 и 1803 година от инженерите Луи-Александър дьо Сезар (*Louis-Alexandre de Cessart, 1719-1806*) и Жак Винсент дьо Лакроа Дилон (*Jacques Vincent de Lacroix Dillon, 1760-1807*) (виж фиг.8). Мостът бил открит за експлоатация през 1804 година. Той бил с много интересен дизайн, състоящ се от опори от каменна зидария и връхна конструкция наподобяваща короните на дървета в овощна градина.



**Фиг.7:** Изглед 1 от моста Лаасан  
(днес Лазани, Полша), 1796



**Фиг.8:** Изглед към Понт дез  
Арт, 1804

Между 1801 и 1806 година на река Сена в Париж бил построен първият френски пътен мост, наречен Аустерлиц в чест на едноименната победа на Наполеон над коалиционните армии при Аустерлиц през същата година. Подобно на построения по-рано пешеходен мост Понт дез Арт, бил изпълнен с чугунени арки върху опори от каменна зидария. Плановете на моста били на Джоузеф Мари Станислас Беки-Бопре (*Joseph Marie Stanislas Becquey-Beaupre, 1751-1834*).

Въпреки двата чугунени моста на река Сена, Франция имала да решава технологични проблеми, за да достигне напредъка на Обединеното кралство. Технологиите на металолееене с използване на дърва правели продукцията скъпа и некачествена, което довело до временно изоставяне на чугунените мостови конструкции.

До края на XVIII век Русия застанала начело по производството на чугун в света, като произвеждала 9 908 хиляди пуда чугун годишно, докато в Англия били произвеждани 9 516 хиляди пуда<sup>4</sup>, следвали Франция, Швеция и САЩ.

Строителството на първите руски пътни мостове започнало през 1806 година. В Санкт Петербург били построени два моста, първият от които бил с ажурна оребрена чугунена конструкция. Премоствал Салнобуянский канал, близо до брега на река Нева. Бил известен като Бердовия мост. Той бил завършен през 1807 година и имал отвор от 19.2 метра. Вторият руски железен мост е известен като Зеления мост. Премоствал река Мойка. Той се използва и в наши дни. Зеленият мост бил завършен през 1808 година. Той бил едноотворен, с плосък свод, изпълнен от свързани една с друга кухи чугунени кутии (тюбинги), с клиновидна форма и с размери 2.8/ 1.53/ 0.38 метра и дебелина на стената 2.5 см, свързани с болтове.

Фугите между кутиите запълвали със замазка от сяра, нишадър и чугунени стружки в пропорция 1:1:16. Устоите на моста били фундаментирани върху дървени пилоти и ростверки. Мостът бил с обща дължина 30.7 метра и имал светъл отвор от

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<sup>4</sup> Стара руска мярка за тегло. 1 пуд = 16,38 кг

21.5 метра. Автор на проекта и ръководител на строителните работи бил Василий Иванович Гесте (*англ. William Hastie, 1753 или 1763-1832*), руски инженер с шотландски произход.

Гесте изпълвал в своя проект авторската идея на американския инженер Робърт Фултон (*Robert Fulton, 1765-1815*), публикувана в негова книга, издадена в Лондон през 1796 година. Това бил първия в историята на мостовото строителство типов проект за метални мостове. В съответствие с него започнали да изготвят чугунени блокове за цяла серия мостове, но началото на Френско – Руската война от 1812 година задържало строителството и то било извършено през 1814 - 1818 година, когато били построени още три моста от този тип.

### **3. Заслугата на един политически памфлетист за големите чугунени, сводови конструкции. Мостът на река Тийс в Съндърланд от 1796 година.**

Между 1776 и 1783 Томас Пейн (*Thomas Paine, 1737-1809*), емигрант в американските колонии (САЩ) от Англия, участвал във войната за независимост на страната на Пенсилвания. Той написал 16 памфлета, които подписал с псевдонима „Здрав разум“ (*Common Sense*), които са останали в американската история. Те били толкова популярни и въздействащи, че Джон Адамс (*John Adams, 1735 – 1826*) написал за автора им: „Без перото на автора на „Здрав разум“ мечът на Вашингтон би бил вдигнат напразно“ [1].

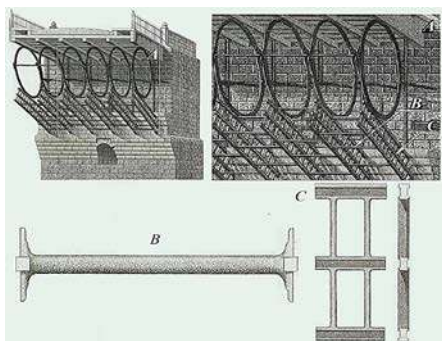
След войната вниманието на Томас Пейн било насочено към различни изобретения, едно от които било за сводов, чугунен мост, арките на който щели да бъдат изпълнени от къси чугунени сегменти, свързани един към друг с шини и болтове от ковано желязо. Той определял елементите на моста като „достатъчно компактни, че да позволяват транспортирането в трюмовете на кораби, от фабриката за производство, до която и да е част на света“ [2].

Неразбирането и неприемането на идеите му за възможността да бъдат изпълнявани типови мостове, произвеждани във фабрики отвела Т. Пейн обратно в Европа. В родната му Англия той финансирал изпълнението на експериментално ребро, а по-късно изпълнил мост в селцето Падингтън, близо до Лондон през 1790 година. Томас Пейн не успял да привлече индустриалците към идеята си за строителство на фабрика за производството на елементи на сглобяеми мостове, но мостът останал и бил използван по-късно като идея за изграждане на големия мост на река Уер в Съндърланд.

А Томас Пейн се завърнал на полето на политическата сатира с книгата, превърнала се в настолно четиво на европейските революционери, в която в пълна степен формулирал принципите на изграждане на демократичните общества – „Правата на човека“.

През 1792 година била подета инициатива за изграждане на мост на река Уер в Съндърланд, за което бил приет акт на парламента от 1792 година. С ръководството на проектирането, финансирането и изграждането на моста се заел Роуланд Бърдън (*Rowland Burdon, 1757-1838*), местен депутат в парламента. За изпълнението на връхната конструкция той се обърнал към леярната работилница Уокърс в Родъръм. Проваленият търговски проект на Томас Пейн от 1791 година накарал Джошуа Уокър - собственик на леярната, в която била изпълнена конструкцията на Пейн,

заедно с тримата му братя, да се обърнат към идеите на задължнелия си клиент. Заедно с Бърдън планирали изграждането на железен мост, имитиращ моста в Паддингтън.



**Фиг. 9:** Детайли от връхната конструкция на моста Уермът



**Фиг. 10:** Изглед към моста Уермът, преди 1857 година, фотография

Организацията на строителството била на Бърдън и Томас Уилсън - инженера на обекта. Няма сведения за проектанта на моста. Проектът бил изготвен така, че да е по-близо до сегментните криви на каменните сводови мостове строени в онези години. Ребрата били чугунени, очертани по сегмент от окръжност с хорда 70.9 метра и максимална стрелка 10.4 метра. Били сглобени от 105 отливки, всяка с дължина около 0.75 метра. Подобно на каменните мостове били изпълнени с вусоари (виж фиг. 9 - елемент „С“), свързани с шини от ковано желязо и болтове. Свързването (раздалечаването) на ребрата било решено с влагането на напречно разположени тръби (виж фиг. 9 – елемент „В“) между всеки две ребра. Броят на ребрата бил шест, което осигурявало ширина на моста от 9.75 метра (32 фута).

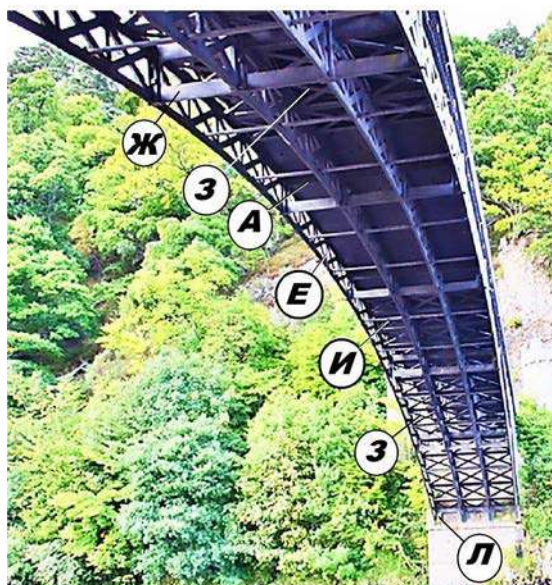
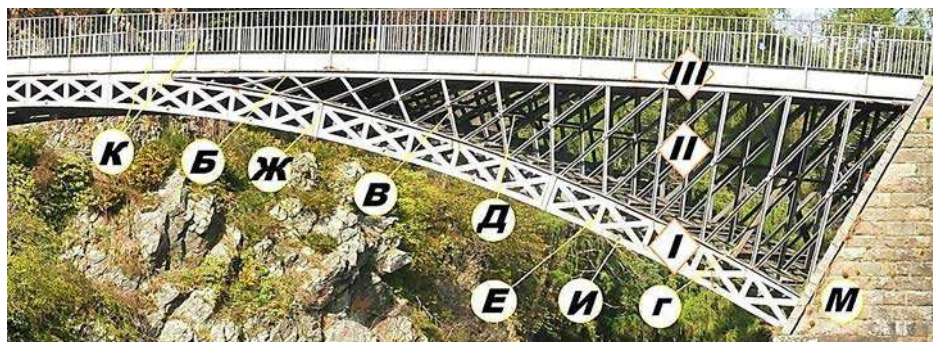
Надстройката била изпълнена от чугунени носещи елементи с кръгла форма. Максималният диаметър бил 3.96 метра. Мостът имал просвет от около 29 метра при отлив и около 25 метра при високи води. Пътното платно било павирано и било с ширина от 6.7 метра (22 фута). От двете му страни имало тротоар с ширина по 1.5 метра, покрит с плочи, в краищата на които били монтирани железни парапети.

Конструкцията тежала 250 тона, от които 210 тона били от чугун, а останалите 40 тона били от ковано желязо, използвано за свързващите елементи. Металната конструкция била обработена повърхностно с въглищен лак. Мостът бил официално открит на 9 август 1796 година.

#### **4. Сводовите конструкции на Томас Телфорд, които могат да бъдат използвани и в наши дни.**

Потребностите от бързо и евтино изграждане на мостове, довели до проектирането на конструкции като тази на моста Бонар. Той бил проектиран от Томас Телфорд (*Thomas Telford, 1757 – 1834*) през 1810 година и построен от септември 1811 до ноември 1812 година. Металната конструкция на моста Бонар послужила като прототип при строителството на множество подобни мостове (виж фиг. 11).





Легенда:

- А. Палубни чугунени плочи
- Б. Пътни носачи
- В. Коси подпори
- Г. Напречни укрепителни връзки
- Д. Хоризонтални болтове
- Е. Дъгообразни ребра
- Ж. Напречни ребра
- З. Решетъчни рамки
- И. Лежащи пръти
- К. Железни бордове и парапети
- Л. Плочи върху устоя
- М. Каменна зидария

**Фиг. 11:** Изгледи отстрани и отдолу на чугунените мостове, построени от Томас Телфорд в периода от 1812 до 1829 година, с означение на съставните елементи на конструкцията.

В тези мостове на Телфорд, освен дъгообразните ребра (Е), съставени от няколко (седем или осем) сегмента и прътова надстройка от коси подпори (В), образуващи т.нар. „диамантена структура“, може да се види надеждното напречно укрепване на моста, изпълнено от напречни ребра (Ж), решетъчни кръстообразни рамки (З) разположени по горната повърхност на дъгообразните ребра, както и успоредни на устоите напречни укрепващи връзки (Г), които заедно с лежащите пръти (И) и хоризонталните болтове (Д) създават надеждна и сигурна напречна стабилност на конструкцията.

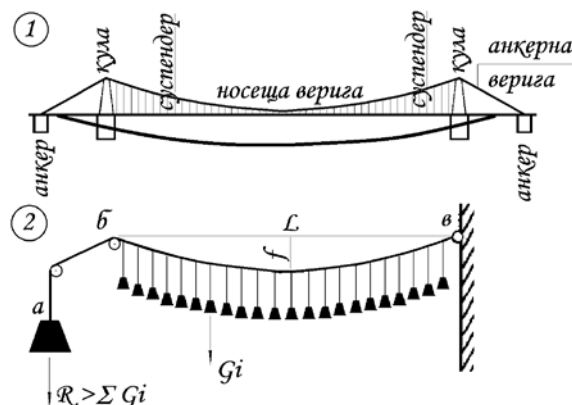
Един от тези мостове е моста Олдфорд на река Дий, със светъл отвор 50 метра (виж фиг.12), построен през 1824 година, чиято богата декоративна украса го превръща в изящно произведение на изкуството (виж фиг.12).

Друг от тези мостове е моста Галтън, светъл отвор 46 метра, построен през 1829 година. Височината на моста (25 метра над канала Смитуйк, Западен Мидландс) го прави най-високия мост в света при построяването му.

Най-голям отвор от построените от Томас Телфорд чугунени мостове има мостът Майт на река Севърн от 1826 година, който е с отвор 52 метра.



**Фиг. 12:** Чугунен мост Олдфорд (Aldford), ОК, Т.Телфорд, 1824



**Фиг.13:**

1. Принципно схема на верижния окачен мост
2. Метод на Финли за определяне на силите и деформациите на системата

### 5. Значението на верижните мостове на Джеймс Финли за съвременните окачени мостове.

През 1801 година Джеймс Финли, мирови съдия, политик и изобретател построил на река Джейкъбс Крик окачен верижен мост с отвор от 21 метра (70 фута). Ширина на пътната палуба била 3.8 метра (12 фута и 6 инча). Това бил първия мост с вериги от ковано желязо, с нивелирана палуба в света. Веригите били прехвърлени върху каменни пирамидални стълбове с височина 4.3 метра (14 фута), построени от двете страни на рекичката. Те били направени от еднодюдмови<sup>5</sup> пръти от ковано желязо, образуващи халки с дължина 1.5 и 3 метра (5 и 10 фута). От носещите вериги се спускали вертикални окачвачи (суспендери), поддържащи напречни дървени греди, разположени под мостовата палуба. Тъй като суспендерите били с променлива дължина, пътната палуба била почти права.

Верижните мостове на Финли били в състояние да издържат много по-големи товари в сравнение със собственото им тегло. В популярното по онова време списание „The Port Folio“, през 1810 година Финли дал подробно описание, как конструкция с дължина 90 метра (300 фута) и ширина от 9 метра (30 фута) ще се справи с натоварването. Самата конструкция изисквала 140 тона метал, но в крайна сметка можела да поддържа 540 тона тегло, което надвишавало два пъти собственото тегло на конструкцията (включително и теглото на дървена пътна част) [3], [4].

Финли построил, проектирал и консултирал общо 21 окачени моста, построени на територията на САЩ.

<sup>5</sup> Прието е английският инч, немският цол и холанският дюйм като измервателни единици да имат еднаква дължина - 25,4 mm.

Мостовете на Финли имали няколко предимства пред окачващите мостове с палуба върху носещите вериги, използвани преди това. Първото от тях било, че имали солидна хоризонтална палуба (виж фиг.15 и фиг.16).

Друга важна особеност на конструкцията на Финли била, че тя имала по две опорни точки за кабелите в двата края на моста. Едната била върху опорните стълбове, а другата – в мястото на анкерирание на веригите в земята. Това нововъведение позволявало по-равномерно разпределение на натоварването.

Следващо нововъведение на Финли било използването на вкоравяващи пътната платформа греди, които били разположени надлъжно под палубата и подпомагали поемането на тежестта. Освен това те предотвратявали вълните в пътната част на моста. Когато се прилага натиск върху палубата, се изпращат малки вълни от точката на контакт, което засяга стабилността и формата на мостовата палуба. Гредите разпределят силата и гарантират, че палубата остава на постоянно ниво. Също така гредите спомагат и за поемане на натоварването от вятъра, като се съпротивяват на люлеенето.



**Фиг. 15:** Мост на Финли над река Потомак над Джорджтаун, 1807 година (Публикувано в The Family Magazine, 1839 година)



**Фиг. 16:** Мост на Финли на река Меримак между Нюбрийпорт и Дий, Масачузетс

Много важна особеност на изобретението на Финли било характерното преминаване на носещите вериги под палубата в средната част на моста, описвайки крива, върху която били разположени директно две напречни греди, което създавало допълнителна стабилност на конструкцията.

Част от принципите в конструкцията на Финли били пренебрегнати от строителите изпълнили мостове по проектите му, което по-късно довело до инциденти. Големият проектант на висящи мостови конструкции Отмар Аман коментирал, как са се отразили тези промени върху първите окачени мостове с думите: „Ако се спазват принципите на Финли за строителството, много от ранните инциденти биха били избегнати“ [5].

Идеите заложили в конструкцията на Финли достигнали Европа, където били използвани в проектите на Самюел Браун, Марк Брунел, Томас Телфорд, братята Сегуин, Гийом-Анри Дюфур, Изамбард Киндъм Брунел и много други проектант и строители на мостове.

В наши дни Джеймс Финли е наричан от мнозина изследователи „баща на модерните окачени мостове“.



## 6. Първите големи окачени верижни мостове: мост на Самюел Браун на река Туид (1818), мост на Томас Телфорд над пролива Менай (1826)

Първите верижни мостове в Европа, построени в началото на 19 век претърпели същия неуспех, както и конструкциите на Джеймс Финли и неговите американски колеги.

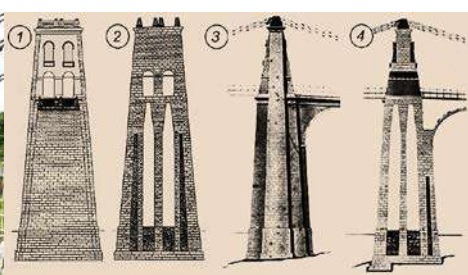
През 1813 година капитан Самюел Браун – военен морски офицер изградил прототипен верижен окачен мост с дължина 32 м (105 фута), в който били използвани прътови вериги, изпълнени по патентования от него през 1816 година метод за производство на връзки за верижни кабели. Браун използвал само 1880 кг желязо за носещите вериги. Той запознал с разработките си някои от водещите британски инженери в строителството на мостове по онова време, между които Джон Рени (*John Rennie, 1761–1821*) и Томас Телфорд (*Thomas Telford, 1757 – 1834*), които били впечатлени от способността на веригите да пренесат толкова голямо натоварване при такъв малък разход на желязо.

През 1818 година бил обявен конкурс за проектиране на пътен мост над река Туид (*Tweed*) - гранична река между Англия и Шотландия. Самюел Браун представил свой проект на моста, според който реката щяла да бъде премостена с единичен отвор от 137 метра. Той спечелил конкурса и до 1820 година построил мост, който заел първенството сред мосто-вете с най-голям отвор в света.

Прътовите вериги били прилагани в две различни напречни сечения на прътите – кръгло, като при моста на река Туид и правоъгълно, каквото избрал за своите окачени верижни мостове Томас Телфорд. Телфорд построил два окачени верижни моста. Единият над река Конуей, а вторият над пролива Менай, който с отвор от 176 метра оглавил мостовите с най-голям отвор.



Фиг.17: Union Bridge - изглед и детайли на свързването на веригите



1. Изглед на стълба напречно;
2. Разрез през стълба напречно;
3. Изглед на стълба надлъжно;
4. Разрез на стълба надлъжно.

Фиг. 18: Стълб на моста на пролива Менай

Прътовите вериги предлагали редица предимства пред пръстените, използвани от Джеймс Финли. Те имали по-голяма носимоспособност, били по-компактни, позволявали по-лесен контрол на състоянието и по-лесна поддръжка. Високата носимоспособност на веригите позволявала прилагането на по-тежки и по-устойчиви на несиметрично натоварване и на въздействието на вятъра пътни конструкции.

Прътовите вериги с правоъгълно напречно сечение пък имали някои предимства пред тези с кръгло напречно сечение, а именно корозирали по-трудно и се контролирали по-лесно. По тази причина те били избрани от Томас Телфорд.

И при двата окачени верижни моста на Томас Телфорд (Менай и Конуей) пътната конструкция била окачена с прътови суспендери на 4 броя разположени една над друга вериги. На моста над пролива Менай, където пътните платна били две, веригите били 16 броя, а на моста на река Конуей, който бил с едно пътно платно, веригите били 8 броя.



**Фиг. 19:** Изглед и план на моста на пролива Менай на Томас Телфорд, 1826

В краищата на моста веригите преминавали в тунели в дълбочина на скалата, в които с нитовани чугунени плочи били изпълнени рамки, към които били закачени веригите, а тунелите - запълнени с каменна зидария.

Носещите кули били от каменна зидария, подсилена с метални скоби. Телфорд изисквал всички фуги да бъдат внимателно запълнени с разтвор, както във видимата част на зидарията, така и от вътрешната страна на кулите.

Подпирането на веригите върху носещите кули ставало на специални подвижни седла, осигуряващи движението им над опорните стълбове. До такова движение се достигало при промяна на товара и при промени в температурата, водещи до удължаване или скъсяване на носещите вериги. За да се избегне ръждясването на веригите, желязото било наkisвано в ленено масло и по-късно било боядисвано.

Мостът над пролива Менай бил открит на 30 януари 1826 година.

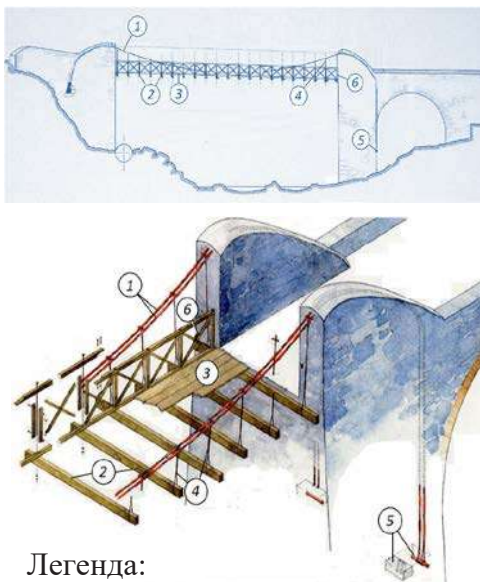
### **7. Окачени мостови конструкции с телови кабели.**

През 1822 година братята Сегуин построили над река Канс (*Cance*), в родното си село Верноск лез Аноне (*Vernosc-les-Annonay*) окачен пешеходен мост с кабели изпълнени от желязна тел (виж фиг.20). Този и още един експериментален мост, изпълнен в Сен Валиер (*Saint Vallier*) през 1823 година послужили като ценен опит преди следващата стъпка - построяването на окачен мост на река Рона между Турнон и Тайн през 1825 година. Според договора, те щели „с помощта на концесията, да



наложат пътна такса, която да бъде установена на този пътен мост“ за 99 години. Това била първата концесия предоставена от френското правителство.

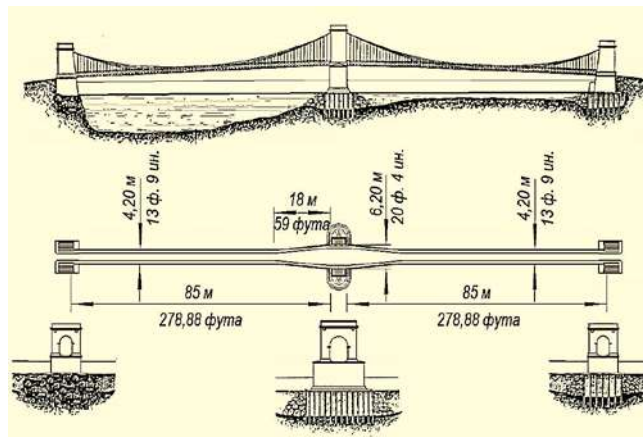
Първият проблем бил с изпълнението на опората в реката, където силното течение и липсата на естествен остров или плитчина затруднявало изпълнението на опората. Сегуин решили този проблем, като изпълнили дървена шпунтова ограда, около която направили насип от камъни, а във вътрешността изпълнили фундамент от римски бетон, армиран с метални пръти (виж фиг. 22).



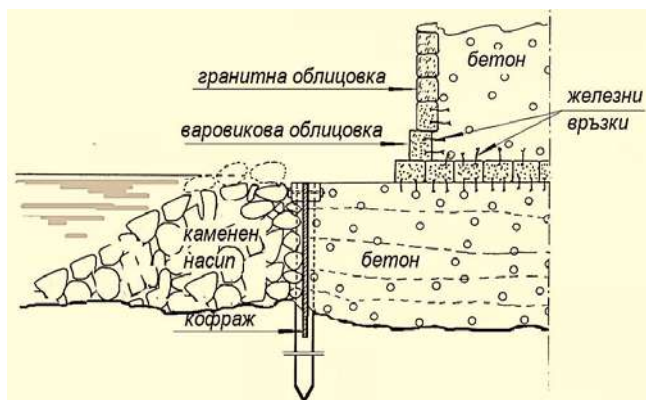
Легенда:

1. Телени въжета
2. Напречни (носеци) греди
3. Дъсчена палуба
4. Суспендери
5. Железен цилиндър и закотвящ камък
6. Връзка от ковано желязо

Фиг. 20: Скица и детайл на моста Vernosc-les-Annonay на река Cance, 1822



Фиг. 21: Скица на моста Турнон – Тайн на река Рона, 1825



Фиг.22: Детайл на фундамента на стълба на моста на р.Рона между Турнон и Тайн



Фиг. 23: Детайли от моста Анданс



Опората над фундамента била зидана, от варовикови камъни, с монтирани в тях железни пръти, които да направят връзка с изпълнения във вътрешността на опората бетон.

Окачващите кабели в мостовете на Сегуин са независими за всяко поле. Те преминават над опорите и се спускат към основата, където се свързват към анкерирани в основите железни пръти (виж фиг. 23).

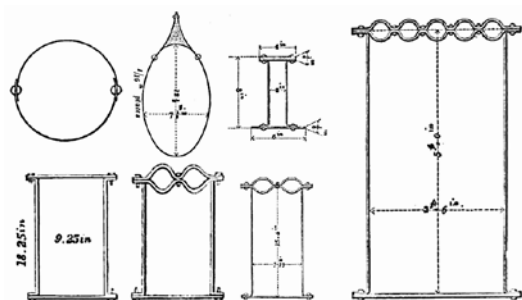
Успехът на моста на река Рона, Турнон – Тайн, бил забележителен, което личи и по финансовия успех. Официалната прогноза за себестойността на моста била за близо 200 хиляди франка. Строителството възлязло на малко повече от 150 хиляди франка, което било много ниска цена, ако се сравни с каменни мостовете. Дори окачени мостове с вериги от ковано желязо, построени на това място, биха възлезли на стойност в пъти по-висока от кабелната конструкция, построена от Сегуин. Годишните доходи от таксата за преминаване по моста през първата година добили 50 хиляди франка, което означава, че перспективата за възвръщане на инвестиция капитал била за след три години.

Мостовете с телени кабели от ковано желязо били много ефективни, поради което били построени стотици мостове по системата Сегуин, но имали и сериозни недостатъци. На първо място била корозията на теловите. Друг недостатък бил че леките конструкции лесно влизала в резонанс с преминаващи по моста групи от хора, нещо което било установено още в началото на строителството на мостовете на Сегуин, и те забранили ходенето в крак по мостовете. От голямо значение за по-нататъшното приложение на тези мостове било и невъзможността им да поемат натоварването на железните пътища. Самите братя не използвали окачени мостове при изгражданите от тях жп линии. Това било така, докато на Американския континент не намерили решение на проблема.

#### **8. Тръбните железопътни мостове на Робърт Стивънсън. Тръбен мост „Британия“ на пролива Менай (1850), тръбен мост „Виктория“ на река Сейнт Лорънс (1859).**

При изграждане на железопътната връзка между Лондон и пристанищния град Холихед, т.нар. линия на „ирландската поща“ възникнала необходимост от изграждането на два големи моста – над пролива Менай и над река Конуей. Тъй като трасето на жп линията минавало в близост до пътната връзка, изпълнена четвърт век по-рано от Томас Телфорд, първоначално се смятало пътните мостове да бъдат използвани за преминаване на железопътните състави по тях, като бъдат разкачвани на единия бряг, прекарвани по мостовете с животинска тяга и отново прикачвани към локомотив на другия бряг. Тази идея била изоставена и било възложено на Робърт Стивънсън (*Robert Stephenson, 1803-1859*), утвърден железопътен инженер, син на пионера в железниците Джордж Стивънсън, да проучи и докладва възможността за пресичане на двете големи водни препятствия. Първоначалната идея за прилагане на окачена верижна конструкция била отхвърлена от Стивънсън. След обсъждане и отхвърляне на редица варианти, той предложил тръбна конструкция, която при необходимост да бъде подкрепена с вериги, окачени към носещи кули. Влакът цял да се движи в носещата тръба. Последвала дълга поредица

от експерименти с пробни образци, за да се достигне до необходимата форма, размери и технология на изпълнение.



Фиг. 24: Скица на моделите, изпитвани от Феърбърн



Фиг. 25: Фрагмент от кутията на моста Британия

За детайлния проект на тръбите Стивънсън си осигурил помощта на инженер Уилям Феърбърн (*William Fairbairn, 1789-1874*), известен с големите си практически познания в изследването на експериментални модели и Ийтън Ходжкинсън (*Eaton Hodgkinson, 1789-1861*) – математик, на когото се разчитало да изведе формула, отразяваща корелативна зависимост на параметрите на конструкцията с резултатите от изпитването. След редица изпитвания на модели с различни форми, било достигнато до решението за приложение на правоъгълни тръби с „гофриране“ на натисковата зона на сечението. За целта била изпълнявана клетъчна структура от плочи и ъглови профили, усилващи натисковата зона, повишавайки коравината и устойчивостта на тази част от тръбата.

Окончателното изпитване на първата тръба показало, че тръбите ще имат достатъчна носимоспособност и коравина за да поемат натоварванията от тежките железопътни състави преминаващи през тях и да ги пренесат с висока скорост. Отпаднала необходимостта от укрепващи вериги. Все пак кулите, върху които биха могли да бъдат монтирани такива вериги били построени.



Фиг. 26: Монтиране на тръбната греда на моста Британия



Фиг. 27: Изглед от моста Британия

Гениален бил и подходът към монтирането на връхната конструкция. Тръбите от средните отвори били монтирани на брега и впоследствие докарани до отворите и издигнати с помощта на хидравлични преси. Това се налагало основно поради изискването на Адмиралтейството проливът да остане плавателен през цялото време на изпълнение на моста. В крайните отвори монтажът на тръбите се изпълнявал върху скеле, на място. В най-натоварения период от време на строителната площадка работили около 1500 мъже.

Мостът над пролива Менай бил с два средни (светли) отвора по 140,2 метра и два крайни (светли) по 70,1 метра. Общата дължина на моста била 460,55 метра. Всяка тръба образувала непрекъсната греда от ковано желязо с тегло от 4680 тона.

При монтирането на тръбите на моста Британия били занитени над 2 милиона нита, с общо тегло около 900 тона. На 5 март 1850 година Робърт Стивънсън монтирал собственоръчно последния нит на конструкцията, отбелязвайки официалното завършване на моста.

При строителството на моста Виктория на река Сейнт Лорънс в Монреал (виж фиг.28), Робърт Стивънсън участвал като проектант на връхната конструкция. Той проектирал конструкция от двуотворни непрекъснати греди. Ползвайки опита от изпълнените до този момент тръбни мостове, проектирал тръбата с плочи от ковано желязо с различни дебелини. Плочита трябвало да бъдат оребрани с нитовани двойно Т-греди. За разлика от клетъчното укрепване на натисковата зона при мостовете над пролива Менай и река Конуей, приложено от Феърбърн, Стивънсън не предвиждал създаване на клетъчна структура в натисковата зона, а усиляване на зоната със занитени към плочите Т-профили. Тръбата била от ковано желязо, с дължина 157.3 метра (516 фута) и стъпвала на 3 опори. В средата била закрепена към стълба, подпряна с неподвижен лагер, с размери 4.9/ 5.8 метра (16/19 фута), а в краищата била подпряна на подвижни лагери с размери 2.1 / 5.8 метра (7/19 фута). За осигуряване свободно разширение и свиване краищата трябвало да бъдат опрени на ролки с диаметър 15.2 сантиметра (6 инча) и дължина 91 сантиметра (3 фута), разположени между горна и долна чугунени плочи. Общата дължина на моста била 2009 метра.

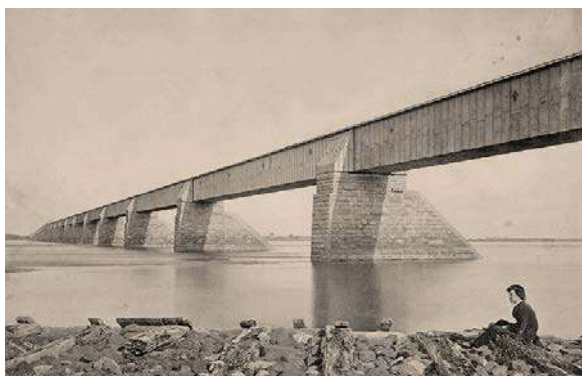
Монтирането на тръбите извършвали върху скеле. Железарските заготовки за моста били изпълнени в Англия. Били разработени детайлни планове за издигане на всяко парче и то било внимателно изпълнено и маркирано с местоположението си и номера на издигането.

При пристигане на мястото на моста, парчетата били систематично подреджани, така че нужните парчета да могат да се намерят лесно. Във всеки отвор имало около 5000 плочи и около половин милион нита. Заготовките от Англия били толкова точни, че снаждането било извършвано само с няколко незначителни настройки.

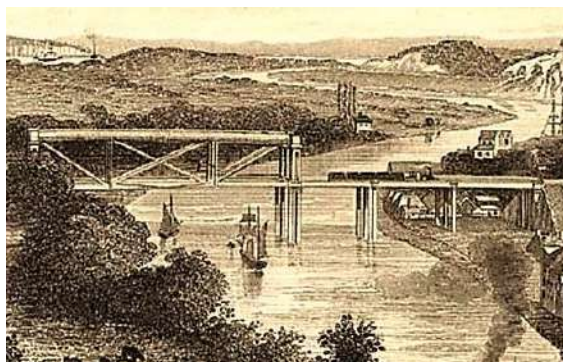
Мостът бил завършен и пуснат в експлоатация в края на 1859 година (виж фиг.28). Официалното му откриване станало девет месеца по-късно, на 25 август 1860 година.

За строителството на моста Виктория била изразходена колосалната за онова време сума от 6,6 милиона долара. Заради всичко, което представлявал мостът, бил наречен Осмото чудо на света.





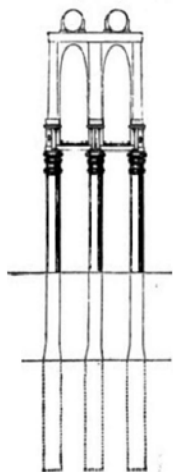
**Фиг. 28:** Изглед към моста Виктория от западния бряг (откъм Монреал)



**Фиг. 29:** Изглед на моста Чепстоу

**9. Железопътните мостове на Изамбард Киндъм Брунел. Тръбен мост на река Уай в Чепстоу (1852) тръбен мост „Крал Албер“ на река Тамар (1859).**

Брунел проектира и изгражда два тръбни моста, през 1852 на река Уай в Чепстоу и 1859 година на река Тамар в Салташ, при които тръбният елемент вместо основен носещ, бил спомагателен и съдействал за работата на носещите вериги на моста. Тръбата в мостовите на Брунел била с по-малко напречно сечение от тръбата в мостовите на Робърт Стивънсън и била разположена над железния път. Тя изпълнявала ролята на разпънка, поемаща хоризонталната компонента от опорните реакции на носещите вериги и съдействала за устойчивостта на конструкцията. Натоварването от главните греди на моста се поемало изцяло от носещите вериги.



**Фиг. 30:** Напр. р-з при опората



**Фиг. 31:** Изглед и напречен разрез на моста на Брунел в Чепстоу (Chepstow), 1852

В моста в Чепстоу тръбите били с кръгло напречно сечение. Носещите вериги поемали част от теглото на тръбата и целия товар от главните греди. Всяка от тръбите стъпвала посредством две междинни рамки върху носещите вериги на моста. Рамките съдействали и за създаване на по-добрата устойчивост на конструкцията, подпомогнати от верижните диагонали, разположени в средната част на отвора.

Долното строене на моста било с конструкция от чугунени тръби (виж фиг.30), изпълняващи роля както на колони на опората, така и на техен фундамент. В основния отвор от 91 метра, главните греди били окачени през 30,5 (100 фута) метра на носещите вериги, а в плитката част на реката били опрени на тръбни опори на разстояние също от 30,5м.

Върхната конструкция била изпълнена върху опорно скеле.

Мостът бил отворен за обществено ползване на 19 юли 1852 година. 110 години по-късно, през 1962 година, върхната конструкция била заменена със стоманена фермова конструкция.

Макар и изглеждаща на пръв поглед много различна, конструкцията на моста на река Тамар в Салташ принципно много прилича на тази в Чепстоу. При нея обаче тръбата е дъгообразна и е симетрична на очертанието на веригите. Напречното сечение на тръбата е елипсовидно. Тръбата стъпва върху веригите в 11 точки. Окачването на главните греди към веригите е в тези 11 точки и в средата между тях. Основната разлика между двете тръбни конструкции на Брунел е в технологията на изпълнение. Ако в конструкцията в Чепстоу е изпълнена върху скеле, на място, то тази в Салташ образува лентикулярната ферма от тръбата и веригите, която била изпълнена на платформа до брега, след което била превозена и издигната на необходимата височина. Всяка една от двете ферми, в двата основни отвора, била изпълнена в различно време, за да се осигури корабоплаването по реката.

В този мост на Брунел бил използван голяма част от арсенала с иновации, които приложил в предишните си мостове, а също и две нови уникални строителни решения. Първото от които било проучване и картографиране на речното дъно за установяване най-подходящото място за фундиране на средната опора, за което била използвана тръба от ковано желязо с подходящи дължина и диаметър, пренасяна от баржи, с чиято помощ били направени необходимите сондажи на дъното.

Второто решение е изработването на тръбен кофердам, с чиято помощ били извършени строителните работи на фундамента на средната опора. При изпълнение на изкопните работи бил използван и пневматичен апарат, т.е. бил приложен способът на изпълнение на строителни работи под налягане, по-широко известен като „кесонен способ“.

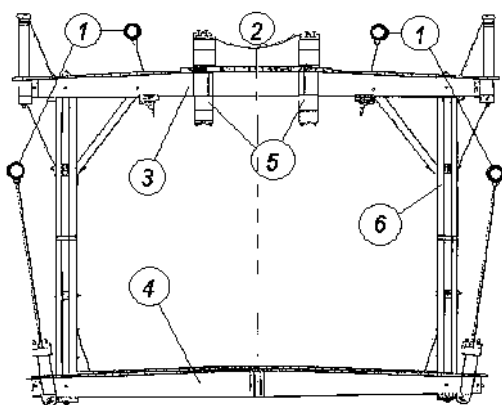
Монтирането на лентикулярните ферми Брунел изпълнил заимствайки приложения от Робърт Стивънсън подход при монтиране на тръбните му мостове – издигайки конструкциите с помощта на хидравлични преси.

Мостът бил открит тържествено на 3 май 1859 година. По здравословни причини Брунел отсъствал от тържеството.

*„След завръщането на г-н Брунел в Англия той набързо посети железопътната линия в Корнуол и за първи и последен път видя в завършено състояние великото дело, за което той бе изразходвал толкова много мисли и грижи“* - пише синът му [6].

Днес мостът на крал Алберт е в редовна експлоатация. Върху двата му портала е изписано „инженер И. К. Брунел, 1859“. Едва ли има по-голям паметник за един инженер от това, което той е построил през живота си.

## 10. Революцията на Ръоблинг с моста на река Ниагара (1855).



Легенда

- |                   |                   |
|-------------------|-------------------|
| 1. Носещи кабели; | 4. Долна палуба;  |
| 2. Железен път;   | 5. Масивни греди; |
| 3. Горна палуба;  | 6. Ферма.         |

**Фиг.32:** Напречен разрез на моста на Ръоблинг

Ръоблинг бил първия, който осъзнал, че за да могат окачените мостове да пренасят жп състави е необходимо те да имат достатъчна коравина. Пътната конструкция трябвало да бъде достатъчно корава и за да поеме опасните колебания, предизвикани от ветровото натоварване.

Той постигнал тези две цели в моста си над река Ниагара с добавяне към окачването на наклонени опорни кабели и чрез образуването на решетъчна кутия, съставена от двете палуби и две дървени ферми, с което без много излишно тегло постигнал забележителна коравина на конструкцията. В допълнение използвал и масивни дървени греди, разположени под железния път, който бил за три различни междурелсия, за което служили четири релсови нишки, монтирани на различни разстояния върху две масивни греди (виж фиг. 32).

В последния си доклад за изпълнението на моста, Ръоблинг гордо заявил за собствения си мост: *„Едно-единствено наблюдение на преминаването на влак по моста на Ниагара ще убеди и най-скептичните в практичността на окачените железопътни мостове, в които толкова много се съмняваха до сега, че е била успешно демо-нстрирана...“*

*Мостовете от половин миля, за обичайно пътуване или за железопътно пътуване, може да бъдат изградени, използвайки желязо за кабелите, с пълна безопасност. Но замествайки ги с най-доброкачествената стоманена тел, можем почти да удвоим разстоянието и да си позволим една и съща степен на сигурност... [7]“*

## 11. Епилог.

Създаването и развитието на мостовете от чугун и ковано желязо през Първата индустриална революция очертава взаимосвързаността на три фактора: **потребност, нови материали, нови инженерни решения.**



**Фиг. 33:** Изглед на моста на Джон Ръоблинг над река Ниагара, 1855 г.



Развитието на производството и свързаното с това увеличение на транспорта, породило потребности от нови пътища и множество нови мостове.

Развитието на технологиите довело до създаване на нови материали – чугун и ковано желязо.

Чугунът и кованото желязо били основата, върху която се развили преобладаващата част от съвременните строителни мостови технологии и мостова наука.

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## PROBABILISTIC RELIABILITY ASSESSMENT OF AN EXISTING STEEL STRUCTURE WITH REGARD TO THE OCCURRENCE OF FATIGUE DAMAGE

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**Abstract:** *Fatigue phenomenon is one of the main factors influencing the life of steel structures and bridges subjected to cyclic loading. The assessment of fatigue life and, in particular, the prediction of residual service life in the existing buildings are a significant and current engineering problem. A number of studies have been conducted for the stochastic estimation of reliability and the subsequent prediction of the life of various carrying capacity elements and constructions. Numerous numerical methods, mostly based on the finite element method - FEM, have been developed to aid in the understanding of the behavior of the fatigue phenomena. The essential tools for these calculations are provided by fracture mechanics and the reliability theory. Some of approaches used for the fatigue crack prediction are based on stochastic methods.*

*The paper focuses on the probabilistic analysis of fatigue damage of the supporting structure of the crane runway serving the steel warehouse operation of the Vitkovice Machinery Group, Czech Republic. For the prediction of fatigue damage over time, calibration functions for short edge cracks were derived based on the results of the experiment, and the acceptable size of the fatigue crack in damaged structural component under analysis was determined. The stochastic methodology for the prediction of fatigue crack propagation is based on linear fracture mechanics utilizing the Paris-Erdogan's law and Direct Optimized Probabilistic Calculation (DOProC).*

**Key words:** *Steel structure, existing structure, fatigue, fatigue crack propagation, linear fracture mechanics, probability of failure, Paris-Erdogan's law, Direct Optimized Probabilistic Calculation, DOProC*

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## 1. Introduction

In the Czech Republic, industrial production has continually intensified since the end of the 18<sup>th</sup> century. Some of the original steel-riveted structures erected at that time have been continuously operated until the present, and these historical structures still fulfil their original function. The assessment of the remaining fatigue life of the members of these structures is difficult to estimate because such older materials and their characteristics are no longer commonly known, and often, the load history has not been recorded. Furthermore, changes in such structures due to repair or strengthening measures within the last hundred years may not be indicated in the drawings. The supporting truss of the crane runway serving the steel warehouse of Vitkovice Machinery Group, Czech Republic (see Fig. 1 and Fig. 2), is an example of such a structure.



**Fig. 1.** Steel truss structure supporting an overhead crane

In many ways, steel bridges exhibit very similar problems to those of such truss structures in industrial facilities [1]. The fatigue assessment guideline for the estimation of the remaining fatigue life of steel bridges [2] is a useful tool commonly applied for condition assessment that complements bridge management systems. Riveting was the dominant joining technique for steel structures from the last decades of the 19<sup>th</sup> century until the late 1930's, and determining the expected lifespan of riveted joints, with a focus on fatigue, is quite complicated due to the general present-day lack of understanding of riveted structure fatigue behaviour [3]. Additionally, more knowledge and experience in the field of fatigue crack propagation in steel structures and the resulting reduction in strength due to cyclic, time-varying loading is necessary [4].

This study accordingly focuses on the analysis of a load-bearing overhead crane support structure subjected to repeated loading. The modern design of these types of steel structures must be in accordance with applicable standards [5–7], which enable efficient and sustainable design often using the appropriate application of available computing software. However, due to the ageing of existing infrastructure worldwide and to the lack of codes for the continued safety management of older structures over their lifetimes, it remains necessary to develop mathematical tools to evaluate structural reliability over time

taking into consideration any reduction in structural capacity [8]. There have been a number of studies on the stochastic estimation of reliability and subsequent prediction of the remaining life of various load-bearing elements and structures [9, 10]. However, there has been to date no generally valid calculation methodology based on probabilistic modelling that is directly applicable to the analysis of existing load-bearing overhead crane support structures assembled by rivets.



**Fig. 2.** Overall view of analyzed steel truss structure

## **2. Fatigue Assessment of Existing Steel Structures**

The proposed computational process described first in this paper includes an analysis of the fatigue load on an existing load-bearing overhead crane support truss. The subject structure is a 3D steel truss built in the 1920s and constructed of mostly riveted joints. In order to conduct an accurate fatigue assessment, it is necessary that the loading history, the details of the individual elements and joints, and the material properties be accurately reflected in the analysis. It is then possible to determine the load duration on a selected element in the structure. Then, the estimation of the residual lifetime of the structure can be performed using methods based on probabilistic or statistical calculations [11].

The analysis of the overhead crane support structure can be divided into three stages. In the first stage, a static FEM model of the entire support structure was prepared in the SCIA Engineering software that could reflect the effects of a moving load through multiple load cases as an influence line. In the second stage, special MATLAB code was written and used to generate an estimated load history using stochastic calculations in combination with the derived influence line. Data from the load history were divided into a spectrum of stress peaks using the rainflow counting method and subsequently used as load cases. The final stage combined the results of the FEM analysis with a 3D model of a selected diagonal member and connection using the ANSYS Workbench Fatigue Module, and a mathematical evaluation of the results provided predictions of the fatigue crack timing and location, and thus the remaining fatigue life. In this study, a diagonal member was selected for examination and modelling that could be removed from the structure for laboratory

examination and that was determined to carry the greatest axial force among diagonals of similar cross sections.

### 2.1 3D FEM numerical model of overhead crane

The SCIA software package was used to prepare a basic 3D model composed of beam elements for the static analysis of the selected continuous frame overhead crane supporting truss, shown in Fig. 3.

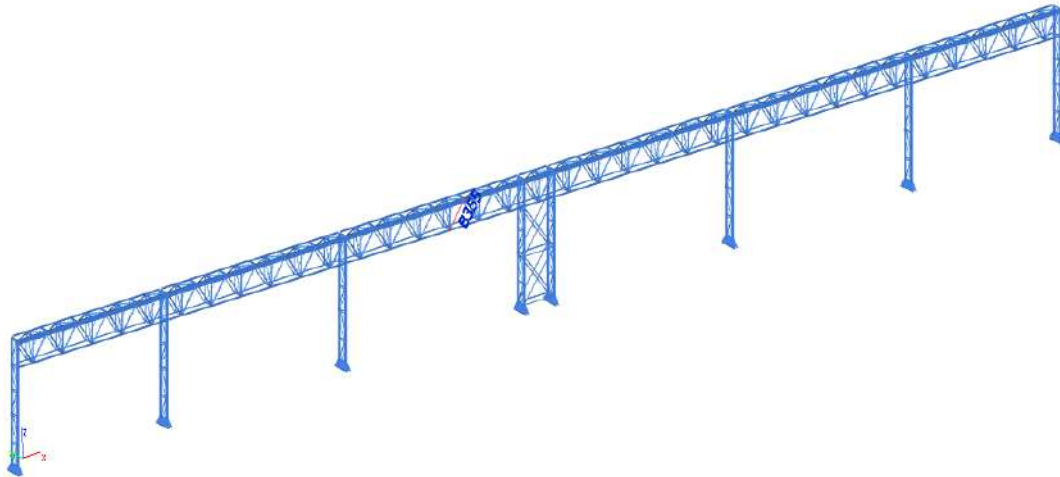


**Fig. 3.** FEM model of the steel truss supporting the crane track in SCIA

The frame structure was 150 m long and composed of the six spans supported by seven truss columns (one of which was the main double column in the middle of the structure). The spans were 23.87 m and 26.04 m long. The main columns were 4.34 x 1.16 x 11.45 m. The main horizontal elements were constructed of two L140x30x14 mm with a steel plate P15x430 mm, fixed by rivets onto the top chord. The bottom chord of the horizontal elements was composed of the same angles, but without the plate. The diagonals consisted of two L90x10 elements and two L120x10 elements. At all connections, the connection plates were P15s.

In the model, the frame corners were considered to be fixed due to their massive riveted connections, as were the supports of all columns. Initial material testing of the steel indicated that it was similar to grade S235 [12]. This material classification is only useful for demonstrating the functionality of the procedures in this paper; it remains necessary to determine the complete mechanical properties and chemical composition of the structural materials using samples taken from the structure to capture the overall fatigue life. Notably, the chosen material classification does not affect the distribution of internal forces in the structure, which is important for further calculations using this procedure.

An initial analysis was performed to determine the most loaded element of the structure. This calculation included a simulation of the moving overhead crane wheels on the crane track using the time-dependent load step method. The simulation was conducted using two loads of 100 kN at a constant spacing of 4.875 m according to the wheelbase of the overhead crane. A readily accessible diagonal, B355 in the third span as shown in Fig. 3, was selected for evaluation and removed for material testing purposes. However, it is important to note that the proposed procedure is applicable to any part of the structure and makes it possible to determine the residual life of any element. The validity of the element selection can, however, be affected by the calculated maximum stress, or other assumptions.



In order to conduct the numerical analysis of the riveted connection model, the influence line of the moving load on the selected diagonal was determined. This was obtained using the numerical model of the structure, geometrical and technological parameters of the crane under operation, and expert input from the operating technician, who estimated the working conditions of the crane, including its characteristics (e.g. distribution of the lifted mass), number of crane movements over the selected working cycle (e.g. one working shift), and number of working shifts per year, etc. A transition analysis was then used to apply a constant moving load from the overhead crane wheels. The resulting influence line was then used to evaluate the load history in the following calculations.

## **2.2 Load modelling and structure behaviour**

To evaluate fatigue damage, it is necessary to obtain a history of the loading (or load response) of the overhead crane structure for a sufficiently representative period of its operation. Long-term measurements are usually very expensive and often very complicated, so simulation methods can be used to adequately determine the load history of critical elements or details. Accordingly, a computational procedure was employed that allowed for the determination of load history or load response in the selected details using the Monte Carlo simulation method [13].

The load can be characterized by a time course over the investigated life span. For structures such as crane tracks, the size of the wheel pressure areas, the number of movement repetitions, and the resulting internal forces in the target element should be determined. To determine the equivalent fatigue load, dynamic coefficients corresponding to the applied load must be implemented. From the point of view the high-cycle fatigue of the elements of the subject structure, it was possible to use the so-called spectrum stress range in place of the standard equivalent fatigue load [5, 14]. This spectrum stress range can be obtained by stochastic analysis using the Monte Carlo method by entering random input parameters. The result of the analysis is the variable amplitude load history of the selected element. This record can then be applied to a detailed model of that particular element or connection.

## **2.3 Sorting by rainflow counting**

To analyse the connection details, it is important to determine the specific effects of the load [14], for which a spectrum stress range, derived from the amplitude of the load history using the rainflow counting method [15], can be used. This method was developed in the 1960s and has been widely used for the fatigue analysis of machine parts and other engineering components. The rainflow method reduces the number of peaks and valleys in the amplitude of the load history and can be used to determine so-called half-cycles, which are then sequenced and sorted. Several available software packages can be used for the method such as the Fatigue Module in ANSYS Workbench. However, this procedure is not easy to apply in complex structures and some input parameters cannot be changed.

Therefore, a separate algorithm that uses the rainflow counting method was developed in this study. This algorithm is able to determine the exact values of each load class and create a 3D rainflow matrix (see Fig. 4), making it possible to precisely determine the half-cycle values for each load class.

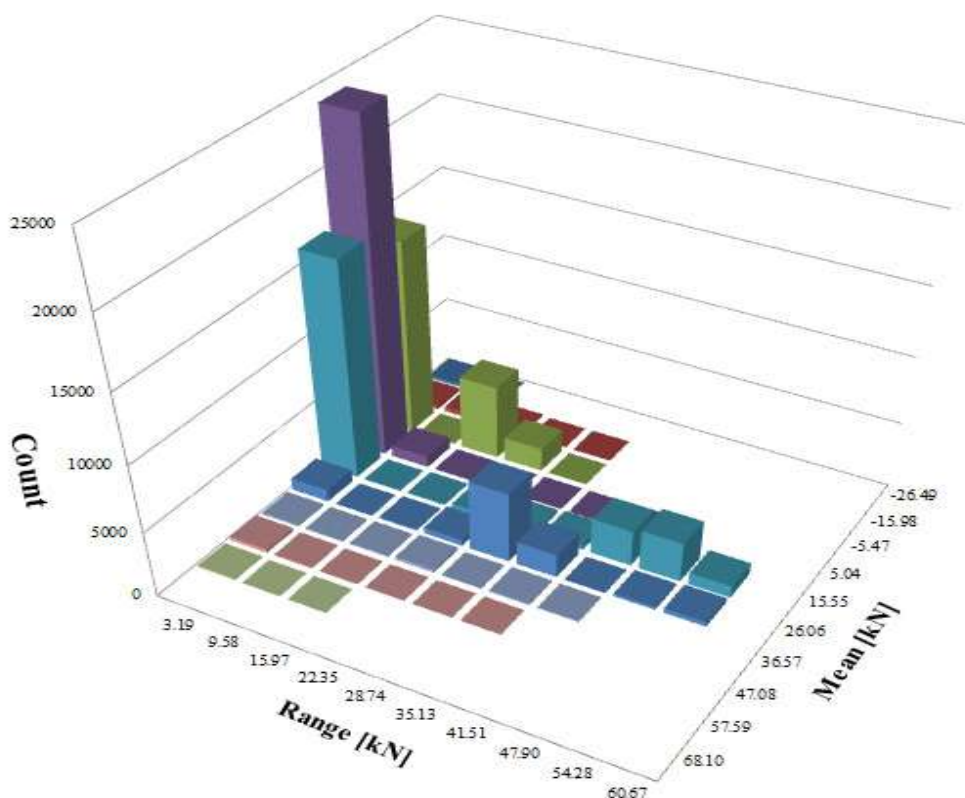


Fig. 4. 3D rainflow matrix load history data for one year

### 3. Fatigue life prediction

#### 3.1 FEM model of connection

For the analysis of the residual life of the riveted diagonal connection, a model of the connection was prepared and evaluated in the ANSYS Workbench software. The model included the gusset plate, the diagonal, and three 24-mm diameter rivets and was constructed of volumetric elements connected by non-linear friction contacts (see Fig. 5).

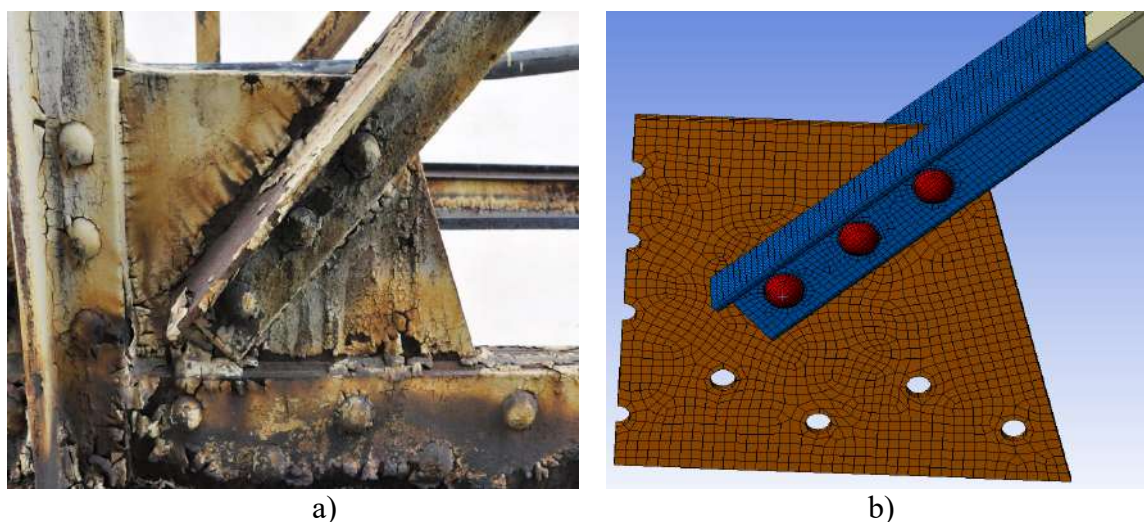


Fig. 5. Selected diagonal connection (a) detail and (b) numerical model

### 3.2 Cumulative Palmgren–Miner Rule

When the individual values of the load combinations are known, it is possible to conduct a numerical fatigue analysis of the selected construction detail. Thus, a prediction of fatigue damage was calculated for each individual combination. These values are the inverse of the predicted residual lifetime. Subsequently, all particular damages were summed by the cumulative Palmgren–Miner rule [16] to determine the predicted residual lifetime of each finite element in the model. The cumulative Palmgren–Miner rule can be expressed by:

$$D = \frac{n_1}{N_1} + \frac{n_2}{N_2} + \dots + \frac{n_k}{N_k} \quad (1)$$

where  $D$  is the resulting cumulative damage,  $n_i$  is the number of cycles for each stress range (load combination), and  $N_i$  is the number of cycles of residual lifetime for each stress range.

The resulting cumulative damage was transferred back to the numerical model for graphical display, allowing for the evaluation of the dangerous parts of the detail considering fatigue damage from a given load. These dangerous parts were generally found to be located at stress concentrations, especially at the locations of the holes where the rivets deform. The maximum cumulative fatigue damages to the gusset plate, diagonals, and rivets were calculated and were then used to determine their residual lifetimes.

To determine the residual lifetimes of each component of the FEM model, the selected S–N curve was used with the cumulative damage to obtain the results given in Table 1. The most vulnerable part of the target detail considering fatigue damage can be observed to be the diagonal, which has an estimated life span of 249 years, and so a remaining lifetime of 159 years. In contrast, the rivets and the connection plate have considerably longer remaining lifetimes.

**Table 1** Connection part lifetimes based on cycles required for fatigue crack initiation

| Connection part | No. of cycles | Predicted life span | Remaining lifetime* |
|-----------------|---------------|---------------------|---------------------|
| Diagonal        | 2.0656E+07    | 249 years           | 159 years           |
| Rivets          | 1.8557E+08    | 2237 years          | 2147 years          |
| Gusset plate    | 2.6628E+07    | 321 years           | 231 years           |

Note: \*Remaining life = Predicted life – 90 year structure age

### 4. Probabilistic modelling of fatigue crack propagation

When investigating the propagation, the fatigue crack that deteriorates a certain area of the structure component is described with one dimension only - fatigue crack length  $a$ . In order to describe the propagation of the crack, the linear elastic fracture mechanics is typically used [4]. This method uses Paris-Erdogan's law [19] and defines relation between propagation rate of the crack size  $a$ , and range of the stress rate coefficient,  $\Delta K$ , in the tip of the crack:

$$\frac{da}{dN} = C \cdot (\Delta K)^m, \quad (2)$$



where  $C$ ,  $m$  are material constants, that are determined experimentally,  $N$  is the number of loading cycles and  $\Delta K$  is range of the stress intensity factor in front of the crack tip. Range of the stress intensity factor is defined as follow:

$$\Delta K = \Delta \sigma \cdot \sqrt{\pi \cdot a} \cdot F(a), \quad (3)$$

where  $\Delta \sigma$  is constant stress range and  $F(a)$  is the calibration function which represents propagation of the crack (for instance, from the edge or surface) and various boundary conditions.

Three sizes are important for the description of the characteristics of the propagation of fatigue cracks. The fatigue crack will propagate in a stable way only if the initial crack  $a_0$  exists in the place where the stress is concentrated. Existence of the initiation cracks during the propagation should be revealed, along the detectable length of the crack  $a_d$ , e.g., during inspections. The crack propagates in a stable way until it reaches the third important size - acceptable length of the crack  $a_{ac}$ , which is a limit for the required reliability. When using Eq. (2), the condition for the acceptable crack length,  $a_{ac}$ , is:

$$N = \frac{1}{C} \cdot \int_{a_0}^{a_{ac}} \frac{da}{(\Delta K)^m} > N_{tot}, \quad (4)$$

where  $N$  is the number of cycles needed to increase the crack from the initiation size  $a_0$  to the acceptable crack size  $a_{ac}$ , and  $N_{tot}$  is the number of cycles throughout the service life.

The main assumption is that the primary design should take into account the effects of the extreme loading and the fatigue resistance should be assessed. The probabilistic methods should be used for the investigation of the propagation rate of the fatigue crack until the acceptable size is reached because the input variables include uncertainties and reliability should be taken into account. The equation for the propagation of the crack size (2) needs to be modified for this purpose. After the change of the number of cycles from  $N_1$  to  $N_2$ , the crack will propagate from the length  $a_1$  to  $a_2$ . Having modified Eq. (2) and using Eq. (3), the following formula will be achieved:

$$\int_{a_1}^{a_2} \frac{da}{(\sqrt{\pi \cdot a} \cdot F(a))^m} = \int_{N_1}^{N_2} C \cdot (\Delta \sigma)^m dN, \quad (5)$$

If the length of the crack  $a_1$  equals to the initial length  $a_0$  (this is the assumed size of the initiation crack in the probabilistic approach) and if  $a_2$  equals to the final acceptable crack length  $a_{ac}$ , then the left-hand side of the Eq. (5) can be regarded as the resistance of the structure  $R_{(a_{ac})}$ :

$$R_{(a_{ac})} = \int_{a_0}^{a_{ac}} \frac{da}{(\sqrt{\pi \cdot a} \cdot F(a))^m}, \quad (6)$$

Similarly, it is possible to define the cumulated effect of loads (random variable effects of the extreme load) that is equal to the right side of Eq. (5):

$$E_{(N)} = \int_{N_0}^N C \cdot (\Delta \sigma)^m dN = C \cdot (\Delta \sigma)^m \cdot (N - N_0), \quad (7)$$

where  $N$  is the total number of oscillations  $\Delta \sigma$  for the change of the length from  $a_0$  to  $a_{ac}$ , and  $N_0$  is the number of oscillations in the time of initialization of the fatigue crack (typically, the number of oscillations is zero).

#### 4.1 Stochastic reliability assessment

As with other probabilistic methods, a safety margin  $Z$  can be defined for reliability assessment:

$$Z_{(X)} = R_{(a_{ac})} - E_{(N)}, \quad (8)$$

where  $X$  is a vector of random physical properties such as mechanical properties, geometry of the structure, load effects and dimensions of the fatigue crack. The analysis of the safety margin  $Z$  gives a failure probability  $P_f$ , which is equal to:

$$P_f = P(R_{(a_{ac})} < E_{(N)}) = P(R_{(a_{ac})} - E_{(N)} < 0) = P(Z_{(X)} < 0), \quad (9)$$

#### 4.2 Application

Described probabilistic analysis of fatigue damage was applied to a nearly 100-year old exterior riveted crane support truss (see Fig. 1 and Fig. 2) focusing on the most susceptible element in terms of fatigue damage - a diagonal formed by a pair of profiles L90x10 in the middle of the truss structure. The study follows the numerical model where the fatigue resistance and the residual life estimation of this structure was analyzed using numerical modelling in ANSYS software based on rainflow counting and the Palmgren-Miner rule, described in chapter 3.

The input quantities were determined deterministically (geometric parameters,  $m = 3$ ,  $C = 2.2 \cdot 10^{-13} \text{ MPa}^m \text{ m}^{m/2+1}$ ) or stochastically using parametric probability distributions (e.g. yield stress of old steel obtained by laboratory experiment, published in [12]; axial force as time-dependent load effect based on the operation of the overhead crane generated by Monte Carlo method [13]; initial size of the crack  $a_0$  according to [18]). Calibration functions  $F(a)$  describing the way of fatigue crack propagation for different types of loads in Eq. (6) were determined experimentally [19]. The required reliability was described by the reliability index  $\beta = 2$  which corresponded to the target probability  $P_d = 0.02277$ .

The described methodology for stochastic modelling of fatigue phenomena has been implemented into the FSCProbCalc (Fatigue Short Crack Probability Calculation) code, which allows probabilistic calculation to be performed efficiently in a user-friendly environment. Using the program the probability evaluation of construction reliability focused on fatigue damage can be carried out upon exact definition of fatigue crack admissible size and definition of occurrence probability of three basic facts related with spreading of fatigue cracks which lead to calculation of probability defect for each operational year of the solved construction. At setting of the reliability required rate also time of the first inspection of the construction can be set which will be focussed on fatigue damage and using the conditioned probability also times of the following inspection checks. Figs. 6 to 10 show the program's desktop with calculated results.

The Fig. 6 presents a histogram stress range  $\Delta\sigma$  in Eq. (7), Fig. 7 shows the resulting histogram of structural resistance  $R_{(a_{ac})}$  calculated according Eq. (6). Fig. 8 contains the accumulated load effect  $E_{(N)}$  for 277 years of structural operation determined by Eq. (7). Both variables are needed to define the safety margin  $Z_{(X)}$  according Eq. (8), see Fig. 9, and probability of failure  $P_f$  estimation in Eq. (9) for the relevant year of operation of the structure under assessment, see Fig. 10.

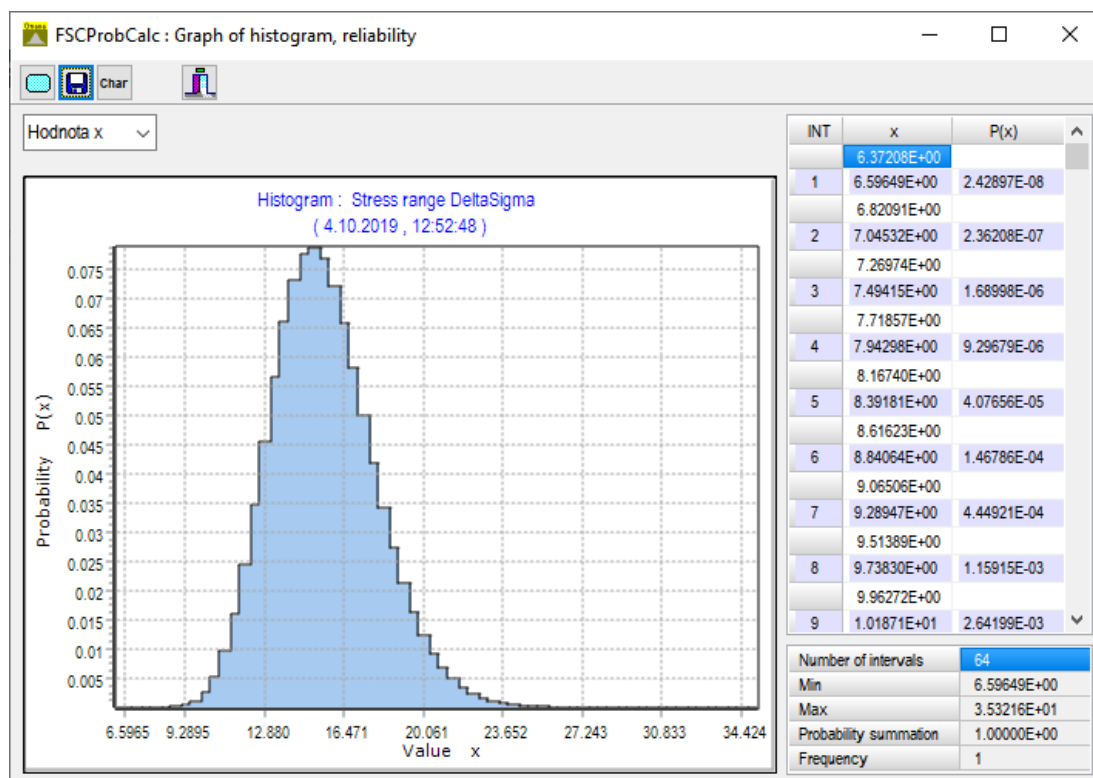


Fig. 6. FSCProbCalc desktop: Calculated histogram of stress range  $\Delta\sigma$

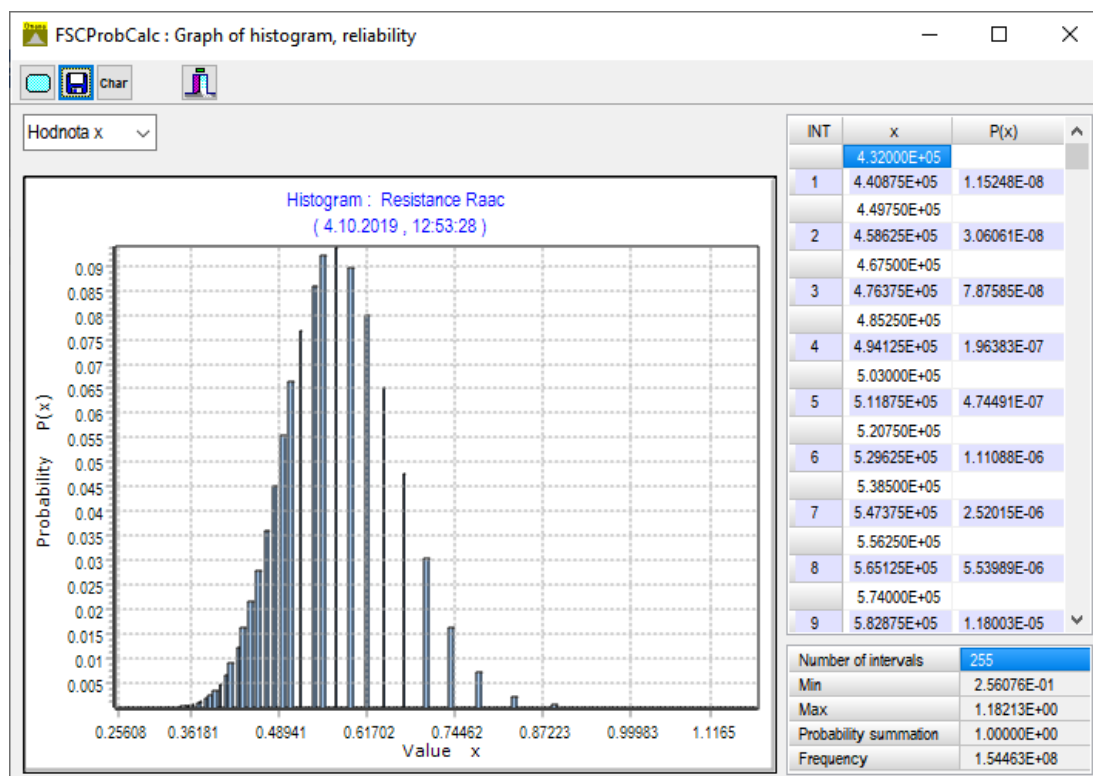


Fig. 7. FSCProbCalc desktop: Calculated histogram of resistance  $R_{(a_{sc})}$



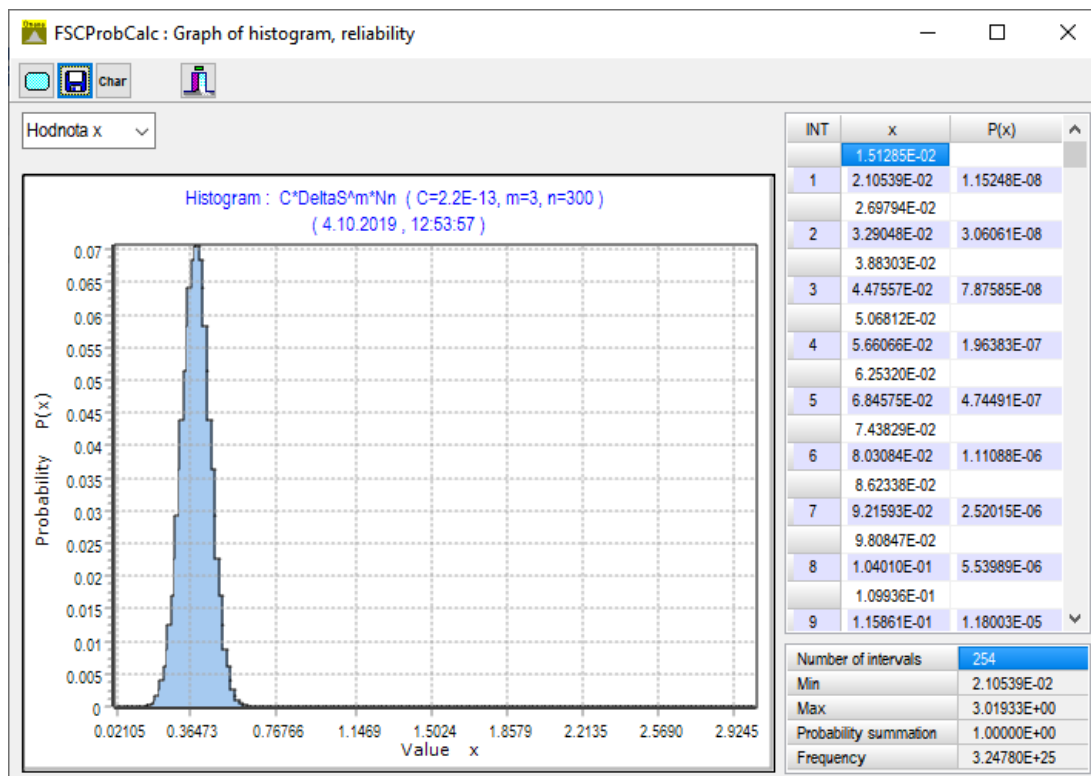


Fig. 8. FSCProbCalc desktop: Calculated histogram of accumulated load effect  $E(N)$  for 277 years of structural operation

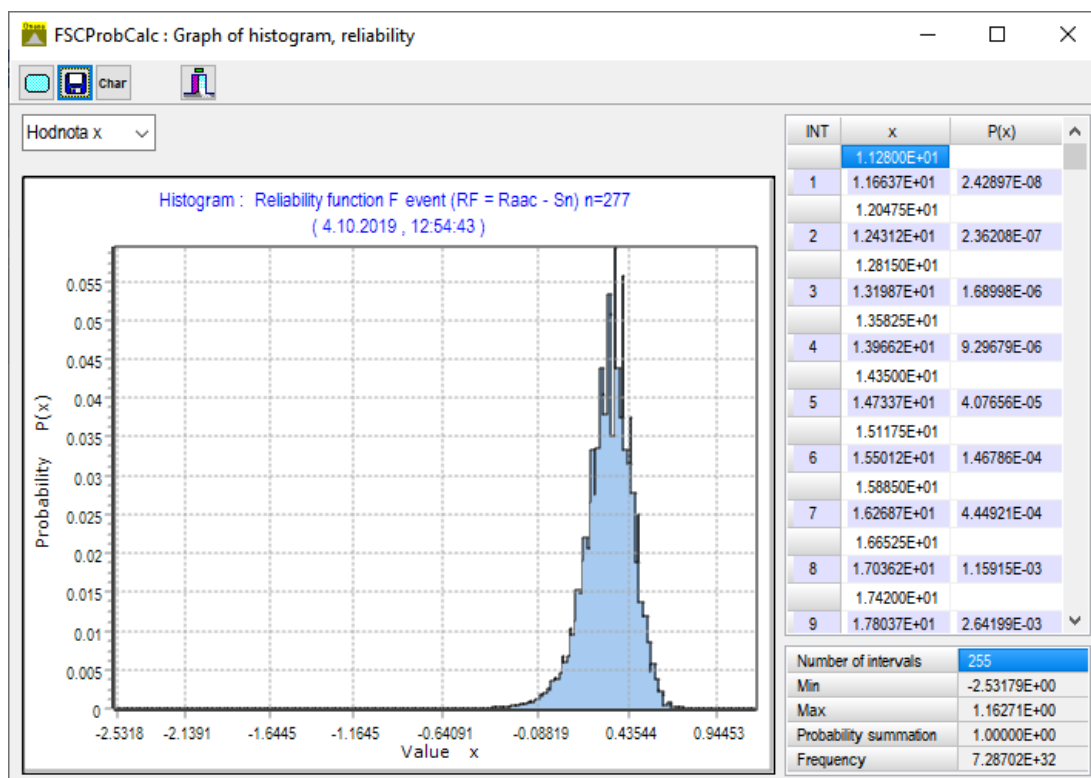


Fig. 9. FSCProbCalc desktop: Calculated histogram of safety margin  $Z$  for 277 years of structural operation

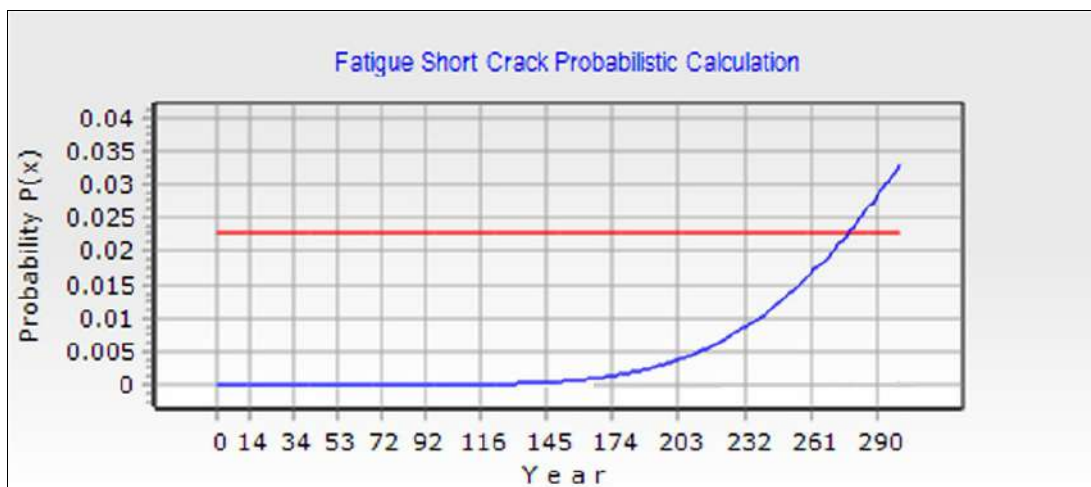


**Fig. 10.** FSCProbCalc desktop: Probability of failure  $P_f$  calculation for each year of structural operation (red curve) with defined target probability (red line)

The target probability  $P_d$  is exceeded in the 277<sup>th</sup> year of the structural operation, which corresponds to a real lifetime of the analysed steel structure. This value is comparable to the result obtained by numerical model, described in chapter 3.

## 5. Conclusion

The objective of this study was to propose and evaluate a probabilistic approach for the fatigue damage analysis of an existing steel truss structure supporting an overhead crane track. To demonstrate the proposed approach, a fatigue numerical and stochastic analysis of the diagonal of the steel truss structure was presented in this paper. Based on the results of numerical fatigue modelling, it was determined that there is substantial remaining fatigue life in the structure. As could be expected, the rivets showed the highest resistance of the considered components.

The methodology for the probabilistic modeling of fatigue crack propagation in the load-bearing member of a cyclically stressed steel structure was applied in FSCProbCalc code using Direct Optimized Probabilistic Calculation (DOProC). The resulting probability of failure was obtained for stochastic described phenomena which are related to propagation of the fatigue cracks and calculated for each year of operation of the construction. When determining the required degree of reliability, it is possible to specify the time of inspection focused on the structural fatigue damage.

The fatigue resistance of the crane can therefore be assumed to be sufficient. The fatigue damage accumulated in old structure during time of the operation is negligible in comparison with, for example, corrosion losses.

The processed methodology together with the stated application thus may substantially improve the estimation of costs spent in maintenance of cyclically loaded constructions and bridges.

## Acknowledgement

This contribution has been developed as a part of the research project GACR 17-01589S "Advanced computational and probabilistic modelling of steel structures taking account fatigue damage" supported by the Czech Grant Agency and also 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 2020 year.

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## USE OF HBIM IN HISTORICAL BUILDING DOCUMENTATION

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*The degradation and deterioration of building should be identified prior to the intervention of historical buildings. For this, it is important to precisely identify the technical features of the time, as well as all the load-bearing and non-load bearing features of the building. Traditional restoration works includes architectural survey and mapping the damages on this survey. These data obtained from the documentation phase is usually kept in different hands in different software formats. A working platform is needed to collect all the data acquired today in a single database, in compatible format.*

*BIM, Building Information modelling system, is a system developed to share files between various disciplines. With this framework, users and contractors can construct a digital project archive, which includes contraventional design details, during the building's life cycle. All data can be controlled by 3-dimensional information sharing. Since the BIM system involves building's physical, functional and technological data and enables multiple disciplines to work on the same project, a research on using BIM in historic buildings will be carried on. In the literature, review of historic buildings with the BIM method is referred to as HBIM (Heritage Building Information Management). While there are numerous studies on the use of the BIM method in modern buildings, there is still insufficient research to do this in historical buildings.*

*It is important to define the history, record its past and archive the data processed to preserve historic buildings, the difficulties that occur in the accessibility, storage and transfer of the documents needed for the historical building intervention process are identified as problems within the scope of study. The purpose of the study is to test the HBIM system in historical buildings as a new method of archiving.*

**Key words:** *Historical Object Library, Object Parameter Management, Historical Information Management*

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## **1. Introduction**

The difficulty and complexity of maintenance methods and lack of awareness of historic buildings are also the cause of inadequate preservation and loss of building integrity [1]. Therefore, prior to the intervention of historical buildings, all information about the building should be compiled and assessed for a reliable intervention decision. This information gathering process is incredibly complicated with traditional restoration techniques. Various methods are being researched to organize the documentation process.

BIM, the framework for Building Information Management, is a system built to exchange files between different disciplines. During the life cycle of the construction, users and contractors can create a digital project repository of design information with this platform. The 3-dimensional exchange of information with Building Information Management (BIM) presents a shared system in which multidisciplinary teams exchange geometrical and documentary building information in a structured manner. In the literature, review of historic buildings with the BIM method is referred to as HBIM (Heritage Building Information Management) [2]. The capacity of BIM to add to the compilation of the dynamic relationships between visible and intangible heritage is increasingly understood [3]. In other words, via object-oriented semantic modelling enriched by heterogeneous information, building information modelling enables all facets of the construction process to be merged into the single platform [4].

BIM improves the performance of the production process in new building through library of BIM environment programs. This library contains geometric and semantic information of building elements. Through this information, it is possible to predict the amount, expense, construction time of the building and to make different analyses as thermal analysis, energy analysis, acoustic analysis etc. However, this library cannot be used in historic buildings because historical building elements are unique. For this reason, a special library should be created for historical building elements.

The purpose of this study is to establish a HBIM library. Considering that the BIM processes of historical buildings have just begun, this study will form the basis for future HBIM studies.

## **2. Creating a historical building library**

Building Information Management (BIM) for historical buildings is a new research area. It is still a matter of discussion whether the preservation of historical buildings can be achieved using HBIM. The unique and irregularity of historical architecture and the large number of details needed to be understood in architectural history, require proper reflection. However, HBIM is an opportunity for both heritage documentation and historical environmental management in adapting the BIM approach to built heritage [4]. HBIM study can be done in order to manage the information system and to create historical building documentation in a single file. It enables virtual building models to be generated that can be connected to numerical data, texts, photographs, and other information forms [5].

It is possible to create documents of historical buildings and integrate the collected data into a single data structure with BIM tools [6]. Software of BIM can be used for HBIM in consisting of objects as structural elements and non-structural elements. Structural elements are made up of walls, abutments, columns, beams etc. Non-Structural elements are made up of windows, doors etc. A parametric object is acquired as geometric and semantic information applied to these modules (Fig. 1).

The Industry Foundation Classes (IFC) are an open standard for the exchange of building data models used in building design and construction across different software. When the information gathered through BIM and HBIM is converted to IFC format, it can



be used in other BIM-based programs. This means that parametric objects can be used for analysis of energy analysis, structural analysis, acoustic analysis etc. in different BIM-based software.

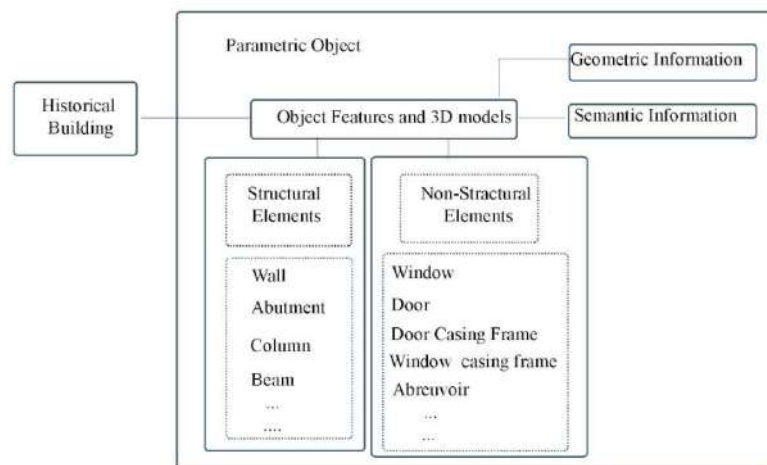


Fig. 1. Parametric object diagram of historical building elements

### 3. Case Study

In order to increase the understandability of the documentation of historical buildings with the HBIM method, a study was carried out on a historical building. There are various methods, documentation of historical building with HBIM. One of these methods is to convert TLS (Terrestrial Laser Scanner) point cloud data into mesh objects using plug-in of BIM programs. However, these transformed objects are not parametric objects. In order for these objects to be parametric objects, they must contain various information as geometric information, semantic information etc.

In this study of Tahtani Mosque in Gaziantep, it was preferred to create parametric objects without TLS data. In the study, firstly, the history of the building was researched and then door, window and wall models were drawn with the Revit program. Finally, semantic information has been added to the Revit objects.

#### 3.1. Historical of the Building

Tahtani Mosque is situated near the castle of Gaziantep in the historical market (Fig. 2). The Mosque's construction date is unknown, but is estimated to have been built before 1543. It was first used as a prayer room.

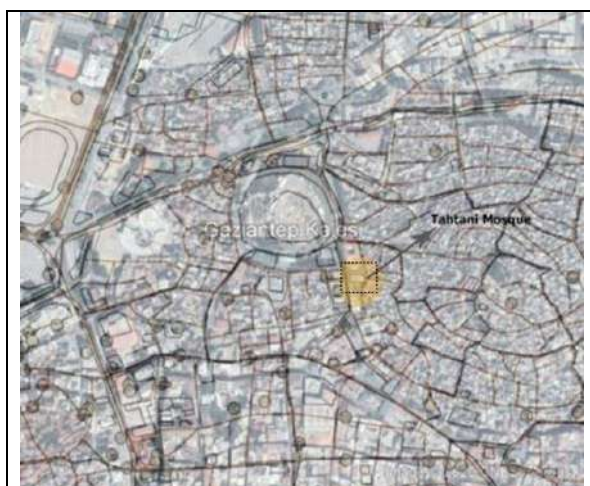


Fig. 2. Location of Tahtani mosque

Tahtani Mosque coordinates were determined from Google earth then uploaded to the Revit program. The climate data is automatically inserted into the device according to the coordinates.

According to researches;

- The building was repaired in 1604.

- It was repaired on 1804.

(In this repair, the minaret's main door and the side door were repaired)

- In 1959, it was repaired under the leadership of Abdüsselam Kazaz.

- Repair was done again in 1983.

(In this repair, the old stones of the minaret and the umbrella-shaped cone of its balcony were repaired, the crown gate of the mosque was renewed and the courtyard floors were changed.)

- It was restored by the General Directorate of Foundations in 2011.

All information collected was manually processed in the Autodesk Revit program (Fig. 3).

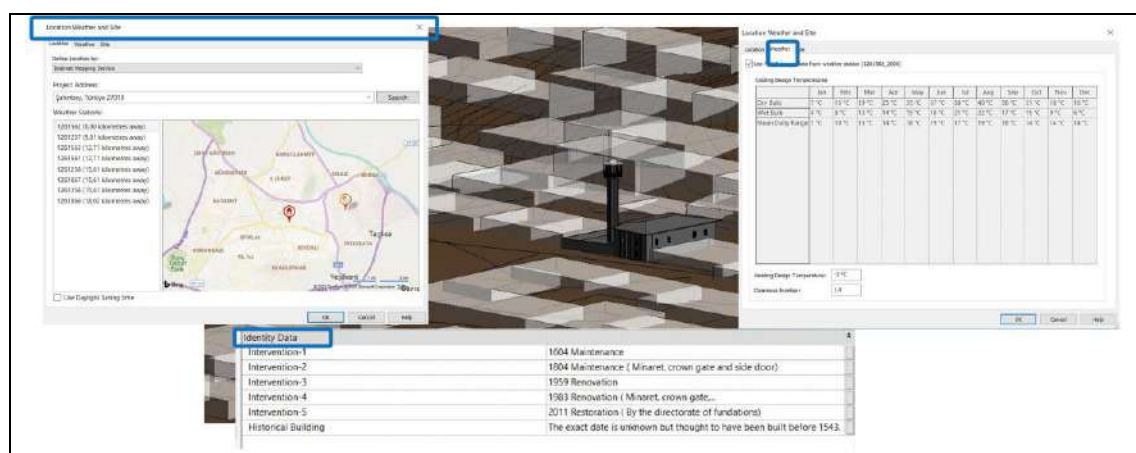


Fig. 3. Location and building Information

### 3.2. Building object

During the field study, door and window measurements were taken manually. It is also accepted that these measurements in HBIM studies are translated from TLS point cloud data. Building elements are drawn with the Revit Family-Window parameter or Revit Family-Door parameter. In this way, it was attempted to create a historical building Autodesk Revit library (Fig. 4). Plans, sections and views are automatically created during the drawing of building elements. In the case of geometric logic errors caused by the person during the drawing, the software gave a notice. In this way, the drawing accuracy can be proved. All the building elements drawn in the project have been integrated into the system and the Tahtani Mosque object library has been obtained. During the drawing, the geometric information was created automatically in the system, but the semantic information was entered into the system manually.

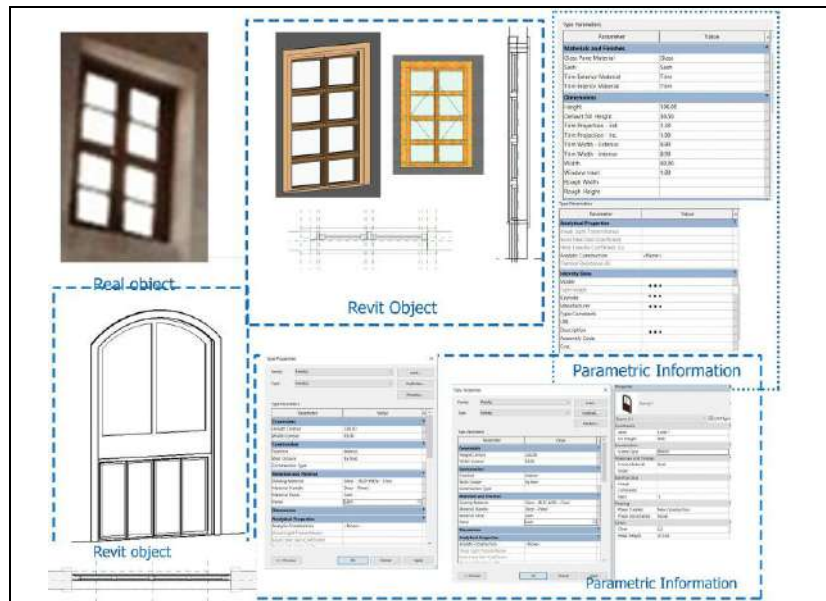


Fig. 4. Parametric Information of window and door

The wall element that existed in the Revit program was not suitable for historical building drawing. Because the Autodesk Revit program is a software designed for new production. In order to draw a wall in the historical building, using the wall parameter in the Architecture Component tab, all dimensions, recesses and protrusions were drawn manually. Following the drawing of the walls, the building elements previously incorporated into the system were positioned on the walls according to their location.

Damages detected in the building are included in the system as semantic information. Semantic information was created under the project parameters tab in the Revit Manage section. All walls are coded according to their axes and their damages are entered manually. The detected damages are supported by photographs under the “other parameter” section (Fig. 5).

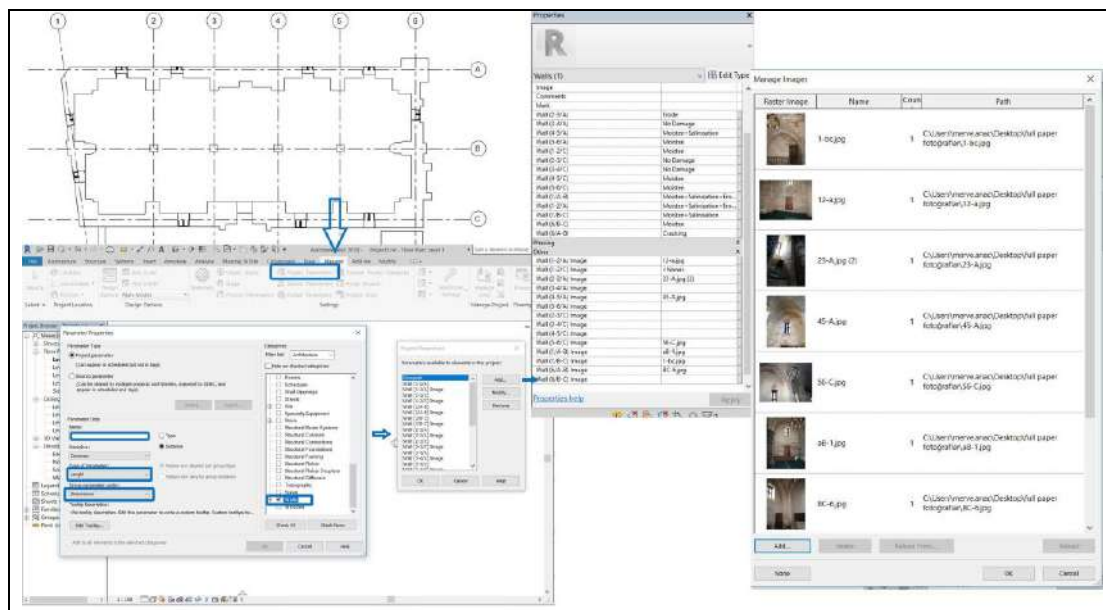


Fig. 5. Parametric Information of wall



#### 4. Conclusion

Whether BIM applications may be used in preservation processes is not clear yet but it is clear that documentation processes have potential. HBIM documentation methods can be used as a new tool in the records management reporting processes in a single file. The documentation and archiving of historical building elements help to review all the information gathered and not to miss any information about the building. In addition, the HBIM objects that have many parametric information allows to make various analyses as energy analysis, structural analysis, acoustic analysis etc. In this way, it is possible to evaluate interventions. Although various problems are encountered in the use of HBIM in historical buildings, since it is an information-based system, it can be used to pile information in archives.

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## REVIEW OF DIFFERENT MODELLING APPROACHES FOR MASONRY ARCH BRIDGES

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**Abstract:** *The oldest arch bridges are ones made of masonry. In the 19th Century, plastic limit analysis has been developed, today rigid block and finite element methods are used. However, in the case of fast assessment of numerous masonry arch bridges, the scientific community and practitioners may turn to a simple procedure to get adequate results in a short time. A review of different modeling approaches for masonry arch bridges is presented, indicating the benefits and lack of each of the presented methods. It was seen that the FEM models are rather time-consuming however more detailed in respect to the rigid block models.*

**Keywords:** *Masonry, Bridges, Arch, Lower Bound Theory, Rigid Block, Finite Element Methods*

### 1. Introduction

Masonry is one of the oldest materials used for the construction of bridges. It has been used all around the world and for many centuries. Bridges are structures that are exposed to various actions, from natural hazards to manmade effects. Most of these structures are considered heritage structures, due to the time of their construction. The time is the witness of their construction and their deterioration over time. It may be argued that the need for a better understanding of these structures has been identified however detailed research and novel modeling procedures began in the last 25 years. The reasoning for this lies in the complexity of masonry structures, their varieties, lack of information regarding the mechanical characteristics of its components and their interaction. Masonry is composed of units with or without joining material in the joints, which can be made of various materials. This is a very heterogeneous and brittle material, and its variations are numerous. This all affected the postponement of research activities in this field.

Thought out history modeling of masonry arch bridges started from lower bound theories (lower and upper bound theories). Later on, this modeling procedure was replaced by a rigid block and different finite element methods. In the paper, brief data will be presented regarding different modeling procedures as well as their benefits and drawbacks.

### 2. Plastic Limit analysis

Application of the plastic theory is possible for masonry bridges provided that some assumptions have been fulfilled, as stated by Heyman [1]. The assumptions' that needed to be fulfilled are: no tensile strength in the arch; the masonry in the arch is incompressible; and sliding between masonry units cannot occur. One of the benefits of this procedure is

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that one needs not to know the initial stress state in the bridge, only a few material characteristics are needed, and only the ultimate state of the bridge is elaborated.

In order to obtain the correct load factor in plastic analysis, several conditions have to be fulfilled. First of all, equilibrium criterium has to be satisfied, meaning that internal bending moments must be equated by the external load. Secondly, the yield conditions have to be satisfied, meaning that no point in the arch should have a moment that is larger than the plastic moment ( $M \leq |M_p|$ ). Considering other internal variables this basically means that the stresses in the material must be everywhere less than or equal to the material strength (e.g. shear, crushing and tensile strength limits must all be respected) [2]. At the final stage (collapse), the arch will transform into a mechanism. And finally, the energy which is being dissipated has to be positive, which means that the work done by the forces at the boundary at the plastic hinge has to be negative.

The external work is defined as:

$$W_e = \underbrace{D}_{\text{extrenal work}} + \underbrace{U}_{\text{dissipation energy} \geq 0, \text{ elastic energy} = 0} \quad (1)$$

$$W_e = W_b \quad (2)$$

representing the work done by the boundary external forces. Meaning that,

$$W_b = -D \leq 0 \quad (3)$$

Depending on the value of the load factor  $\lambda$  by which the applied forces are multiplied three theorems can be identified:

The lower bound theorem (static theorem), where  $\lambda = \lambda_s \leq \lambda_f$ , where  $\lambda_f$  represents the failure load factor. As this is a static theorem this means that the equilibrium and the yield conditions have to be satisfied in all points. In this way, the solution is obtained from the lower side so this represents a method where the obtained results will be on the safer side. Innumerable possible solutions are probable.

The upper bound theorem (kinematic theorem), where  $\lambda = \lambda_k \geq \lambda_f$ , where  $\lambda_f$  represents the failure load factor. This is a kinematic method where equilibrium and mechanism conditions need to be satisfied. The mechanism has to be admissible meaning that the rotations and deflections under loads and adequate boundary conditions need to satisfy the virtual work equations. This means that the internal work by the plastic moment at plastic hinges is equal to the external work of the loads acting on the structure. This is also known as an unsafe method, as one reaches the exact value from the upper side. Innumerable possible solutions are probable.

Finally, the true failure load is obtained if the load determined by the above-mentioned methods is the same. This means that all the requirements of the plastic analysis are fulfilled. This means that there is only one unique solution for a particular structure. In this situation  $\lambda_s = \lambda_k = \lambda_{lim}$ .

Graphical representation of the theories are shown in Fig. 1



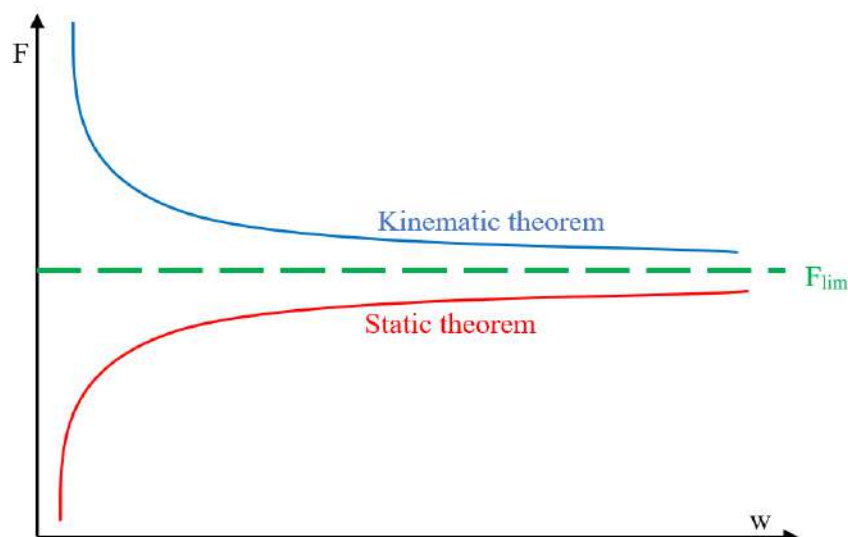


**Fig. 1.** The relationship between the three theorems

Regarding masonry arch structures it is necessary to determine the thrust line at each cross-section. If the thrust line is within the masonry arch this indicates that it is tension-free. On the other hand, the possible location of the plastic hinges has to be determined. At first, only hinging plastic conditions were looked at [3], while later on sliding conditions were taken into account. The masonry structure is constructed of rigid voussoirs laid dry without any mortar. This assumption is quite acceptable, even though stone as the material may possess some tensile strength, the joints will not, so in this respect tensile free forces can be conveyed from one voussoir to another. Usually, no crushing of the stone is expected as the stresses are low, and friction between voussoirs is usually rather high. This justifies the assumptions regarding the infinite compressive strength of stone and that no failure will occur due to sliding.

Finding a thrust line that is tangent to the hinge position represents the uniqueness theorem.

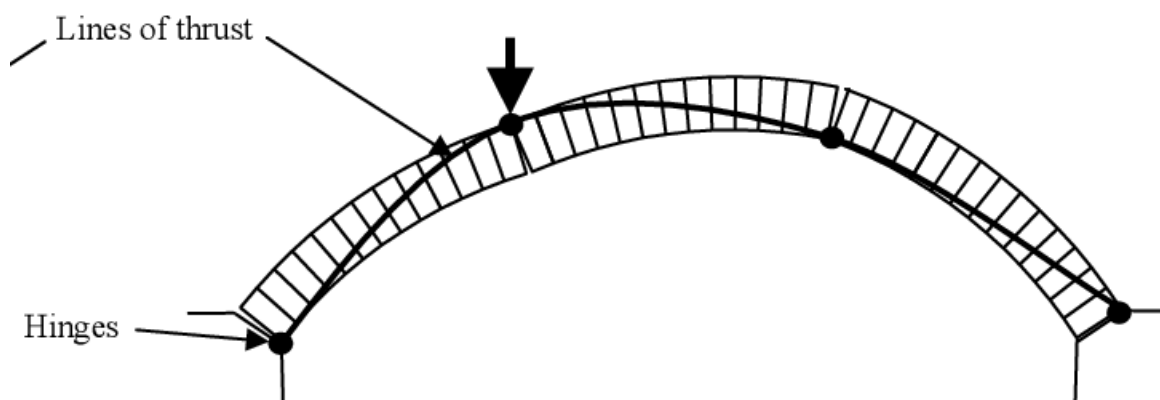
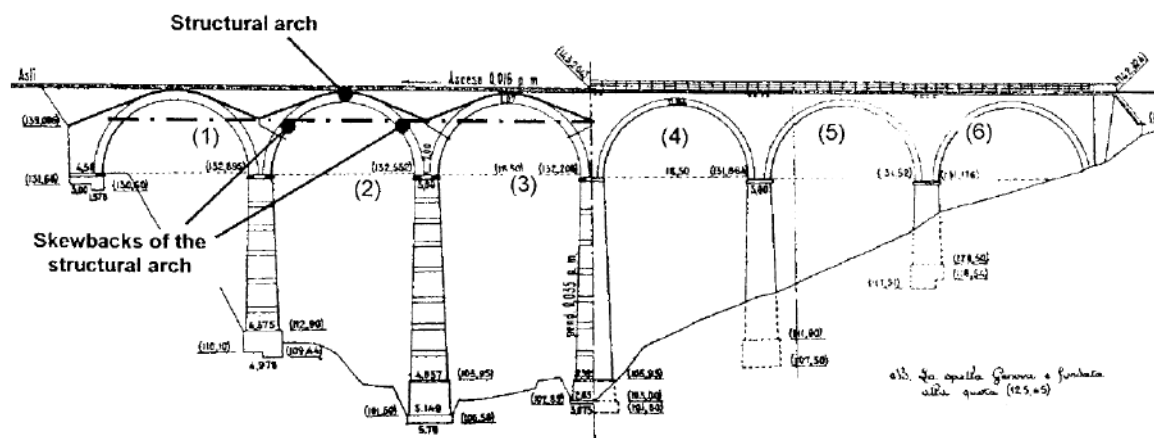


**Fig. 1.** Thrust line at the collapse in masonry arch [2]

Several authors developed different numerical solutions based on this method [4, 5]. Numerous researchers used this method to study stone bridges (collapse mechanism and limit behavior) exposed to dead and live loads, and at first, only vertical loads were considered. Clemente et al. 1995 [6] concluded that if only dead loads are acting on the bridge the safety check is a function of the arch thickness. The work was extended to the

dynamic behavior of stone arch bridges and seismic actions [7, 8]. Here the comparison is done between the expected Peak Ground Acceleration (PGA) with the acceleration value necessary to turn the structure into a mechanism. As various assumptions are considered here (impact between the stone blocks is neglected, infinite strength of the material is considered) this represents only a first approximation of dynamic response. Alexakis and Makris [9] concluded that directions of the rapture in the arch once it translated into a mechanism have a direct impact on the location of the imminent hinges and the minimum thickness of the arch. The main advantages of this method are its simplicity and easy application by engineers in practice, the solution is straight forward, meaning that it can be easily checked, and it is very fast, meaning that it is economically feasible. The drawback is connected to several assumptions. First of all, the compressive strength is always a finite and never an infinite value. To overcome this issue the thrust line was replaced by the trust zone [4], which is of sufficient depth to carry the load at each cross-section. This has led to its application in practice and being adopted by the Department of Transport in the USA as the primary means for the assessment of arch structures. The trust zone refers to a specific cross-section having adequate depth to withstand the applied loads. Further developments were done by Taylor and Mallinder [10] with the introduction of the parabolic stress-strain constitutive relationship for local crushing of masonry at locations of hinges. Brencich and Di Francesco [11] introduced an elastoplastic iterative analysis, with no tensile restraints. Obtaining the ultimate strain represents the collapse conditions. This procedure lies in the conservative part due to the assumptions that have been made and the fact that it is a 2D model. The involvement of the spandrel wall and the arch barrel-fill interaction were not taken into account, and it is well known that they have a beneficial effect on the behavior of masonry arch structures. The proposed method was elaborated on several examples including a railway bridge with six spans know as Cantalupo viaduct (Fig. 2) and good agreement was noted [12]. This method was able to take into account the interaction between the spans of a multi-span masonry arch bridge.



analysis was done for each bridge class and giving a series of acceleration curves as the final results. da Porto et al. [19] based on this analysis concluded that due to an overdesign of masonry structures they, in general, have good resistance to earthquake ground motion. Indicating their good behavior in the case of medium size earthquake actions, without major damage to their main loadbearing structure components.



**Fig. 3.** Analysed bridges (a) single span (b) multi-span [19].

Generally, this procedure is very user friendly, very fast, and does not consume much computational time as is the case with the finite element method (FEM). However, it has its drawbacks, meaning that no information is available readings stresses, strains, and displacement of the structure's elements. The position of the hinges has to be known beforehand and the actual load-carrying capacity due to the assumption of the unlimited ductility may be overestimated.

### 3. Finite element method

The method which has found its application in the masonry arch bridges among other structures is the Finite Element Method (FEM). In this procedure, a continuous macro-modeling is used (Fig. 4), where several assumptions are made.

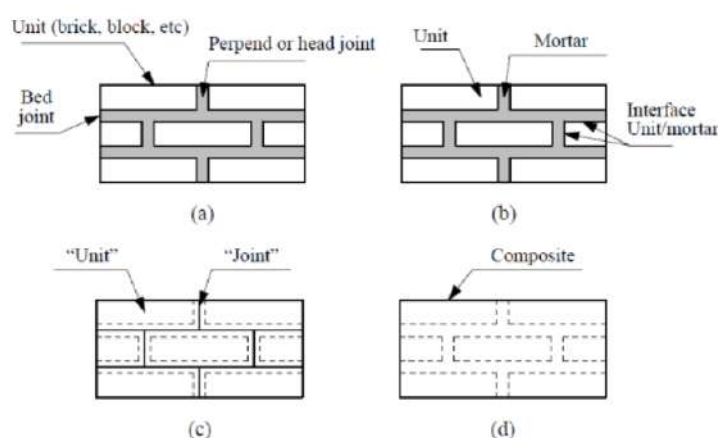


**Fig. 4.** Modelling strategies for masonry structures: (a) masonry sample; (b) detailed micro-modeling; (c) simplified micro-modeling; (d) macro-modeling [20]

First of all, the material is assumed to be a homogeneous isotropic or anisotropic, meaning that the complex heterogeneous nature of masonry is not considered. The composition and mechanical characteristics of individual masonry components (unit, mortar, the interface between mortar and unit) and their interaction is not taken into account individually, but



the masonry characteristics are smeared out in the continuum over the entire masonry structure. In this way, one is dealing with average stress-strain relations. This kind of modeling procedure is usually used for various arch structures, from single to multi-span structures for a preliminary assessment, as well as for the assessment of the structural response to earthquakes and the estimate of the seismic capacity. This method provides a good understanding of the global behavior of the structure. It is quite evident based on the assumptions that this modeling procedure will not be able to grasp specific details in the structure, like cracks along the interphase, some failure modes like failure shear slinging, de-bonding of bricks, etc. Modeling of the masonry arch structures can be done with the application of various elements, from one-dimensional (1D) to three-dimensional elements (3D).

The first application of the one-dimensional finite element method was done by Towler and Sawko [21] in a single-span masonry arch bridge. With the sophistication of the computers modeling of masonry arch structures went one step forward. In the sense that now 2D and 3D finite element analysis of masonry arch bridges are employed. This has enabled us to take into account the interaction of the infill material with the arch barrel [16] and investigated the influence of the spandrel walls [22]. The comparison was done in relation to the real structures or conducted experimental tests on scaled models. Of course, for 2D or 3D models, the required number of input parameters increases, and the determination of the boundary conditions is not an easy task. The computational time increases so here the issue of feasibility of such analysis should be discussed. It can be argued that such an analysis is required in the case of high-level assessment.

**De Felice** (2009) [23] for the first time employed the non-linear beam elements with a fiber-cross section in the assessment of the load-carrying capacity of multi-span masonry arch bridges (Fig. 5). The arch is discretized into straight-lined beams and each beam is subdivided into longitudinal fibres, and their stress-strain behavior is defined in relation to the material properties obtained by experiments. The non-linear constitutive relation of the beam section is derived by integration of the stress-strain relation of the fibres [23], as well as taking into account the effective ductility capacity of masonry under compression.

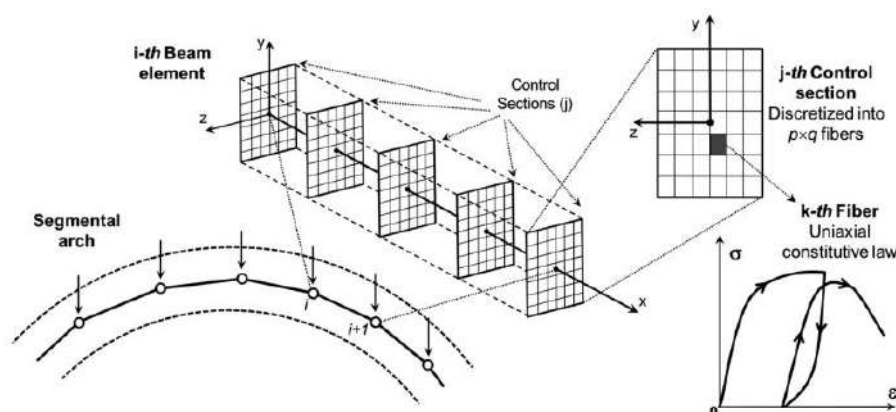


**Fig. 5.** Fibre beam model [24]

This method was used for an assessment of two and three-span arch bridges and compared with the results obtained by the limit analysis. Excellent agreement was obtained for the obtained level of the load-bearing capacity and the failure mechanism. Taking into account the limited ductility capacity a reduction in the maximum load was observed, once again indicating that the limit analysis is too conservative. It has been shown that the computational time is low, while the interaction among the spans is enabled. **De Santis and de Felice** (2013) [25] assessed 50 large-span railway brick masonry bridges in Italy. This method proved to be an excellent tool for a preliminary assessment of railway structures

and showed its benefits regarding computational time and simplicity. This method was further applied to study its dynamic behavior [24]. The method was applied to an existing seven-span railway masonry viaduct for determination of its seismic capacity (Fig. 6).

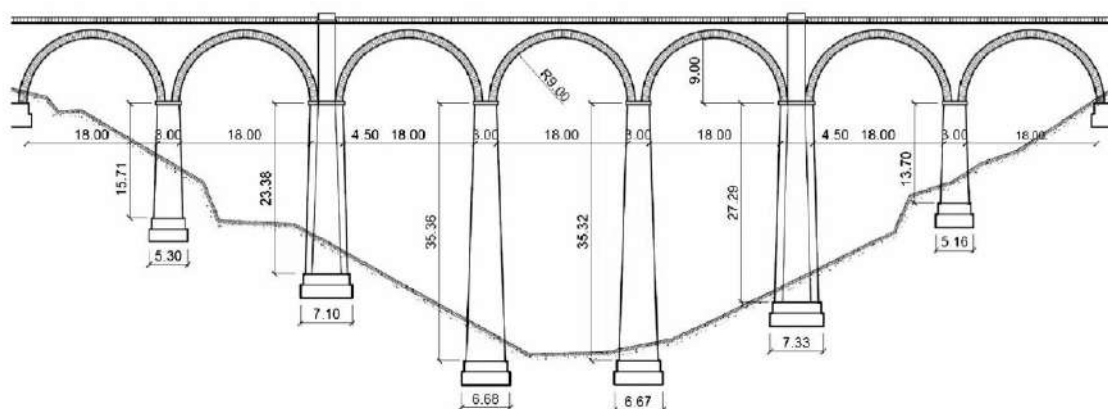


**Fig. 6.** Ronciglione viaduct [24]

It was observed that the application of this methodology was beneficial for computational time and can be applied for preliminary assessment of the seismic capacity of multi-span masonry arch bridges.

The application of 2D and 3D elements for modeling of masonry arch bridges enables a better description of the geometry, infill-material characteristics as well as spandrel walls. In this respect, the number of the input parameters increases, difficulty in the correct definition of the boundary conditions arises and the computational time highly increases. As well, calibration is highly dependent on numerous uncertainties. The development of 2D models was mainly connected to the adequate representation of the in-fill material and the contribution of the spandrel walls in the load-carrying capacity and the structural response.

#### 4. Conclusion

Assessment of the old numerous masonry arch bridges has become a continuous elaboration issue for various public network authorities as these structures are still used as transportation lines. This is additionally emphasized due to the seismic vulnerability of masonry arch bridges. Different methods can be applied for the assessment from the simplest, moderate simplicity to the highly advanced numerical modeling. The paper revisits the pros and cons of different modeling approaches for masonry arch bridges. Depending on the needs and available time for execution of the modeling procedures a method is selected. An interest in the application of simple procedures for seismic assessment in the last decades has emerged, and in this respect, the limit analysis demonstrated its simplicity and robustness. In the case more details are needed modeling can be done with the application of the 2D and 3D elements and the usage of the FEM. However, in this situation, the increase in calculation time is more than evident. A good balance between the accuracy of the required results, availability of input data, and computational time has to be obtained.

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## DAMAGE ASSESSMENT OF CAPPADOCIA HISTORICAL STRUCTURES AND INTERVENTION SUGGESTIONS BY USING WASTE SOILS

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**Abstract:** Cappadocia is an area that is located on the Central Anatolia Region of Turkey. It is in a spectacular volcanic landscape, which is sculpted by erosion and it contains historical rock-cut and masonry structures that were created by different civilizations. Natural and cultural heritage of Cappadocia was inscribed on the World Heritage List in 1985. The natural environment, rupestral and masonry historical buildings, which have been created within the social, cultural and physical dynamics of many civilizations over the centuries. These are about to lose their authenticity and they are vulnerable under the combined detrimental effects of human and nature centered reasons. The growing amount of waste generated through the excavation process of rock-cut buildings, numerous damages, various deterioration, unsustainable and irreversible conservation practices threaten this world heritage site.

In this study, many damages and deteriorations that threaten historical structures and the vestiges of the civilizations of Cappadocia were researched and revealed. In addition, the characteristics of the waste soil of the rock-cut warehouses, which increase rapidly as a result of developing mechanical carving techniques, were investigated using the previous studies. The aim of this study is to scrutinize the damages and deteriorations of the outstanding ancient structures in Cappadocia and to reveal what type of renovation and restoration techniques can be realized by investigating material properties of the waste that is excavated from rock-cut buildings of the same area.

**Keywords:** Damage, Restoration Techniques. Cappadocia, Natural Pozzolan, Waste Utilization

### 1. Introduction

Cappadocia is a World Heritage Site and is home to many historical settlements and structures that civilizations left for future generations. Its borders were determined in 1973



and were listed as a natural and cultural site in the World Heritage List by UNESCO in 1985 (Fig. 1a). Three types of buildings according to construction types of historical buildings' structural systems have existed in this region. These are masonry, rock-cut&masonry, and only rock-cut. Many deteriorations and damages can be observed in these buildings due to multifarious reasons. Beside naturally occurred damages such as cracks, rock-falls, scaling and crust, some type of intervention materials and techniques that are used for restoring these buildings causes several permanent damages on them and destroy their authenticity (Fig. 1b).



**Fig. 1.** a) Cultural and natural site, Cappadocia b) Inappropriate material usage

There are three types of rock-cut structures according to their usage in the region. These are religious, military (defense), and civil architecture buildings [1]. The most common types of rock-cut civil architecture buildings are rock-hewn warehouses in the region. These warehouses are generally used for storing thousand tons of citrus fruits and potatoes. While productions are remained fresh in rock-cut warehouses, electricity is not used to preserve them and their weights increase as a result of the natural material characteristics of rock formation. Therefore, manual and traditional carving methods are not appropriate to satisfy the growing demand for rock-hewn warehouses. Accordingly, mechanical carving was started to be used to carve rocks 30 years before and waste materials as a result of this type of carving reached million tons currently. Although excavated carving wastes do not contain harmful chemicals or content, they threaten the “authenticity” and the “outstanding universal value” of the region. Also, more than 90% of the wastes that cannot be utilized in any way destroy the natural geological heritage of the region (Fig. 2a) and cause visual pollution (Fig. 2b).



**Fig. 2.** a) Excavated waste materials on the buffer zone of heritage site b) Waste material hillocks(~30m) and visual pollution

The utilization of Cappadocia tuff as a natural pozzolan in cement production was worked by Atan [2]. Also, Bek et al. [3] studied excavation waste of one area of

Cappadocia and evaluated that pozzolanicity of waste material is obvious at least for the excavation waste of the investigated area. There is a lack of studies on Cappadocia excavation waste utilization and the problems of the structures in the region that are explained above. It is aimed in this study, to investigate the characteristic features of Cappadocia rocks, which have the same intrinsic properties when they turned into excavated waste. The intention of this study is to reveal up-to-date restoration techniques and compatible materials by evaluating material properties of the region's rock, which will be an excavation waste eventually as a result of increasing rock-cut warehouse requirements.

## 2. Properties and Features of Rock-Cut Structures

Characteristic features and properties of rock-cut structures in Cappadocia are as follows.

### 2.1. Damages and Deteriorations

Rock-cut buildings have unique features that make them a world heritage, besides that different damages are also seen on them. Deteriorations as exfoliation, crust (Figure 3.a), efflorescence (Fig. 3b), crumbling, scaling, biological colonization (Fig. 3d), weathering (Fig. 3d) and damages like spalling (Fig.3c), cracks (Fig. 3d), inappropriate material usage and repair damages (Fig. 3e) are ordinarily observed in rock-cut buildings. Intervention materials especially cement-based ones, which are the most common building materials in the world, cause various damages when used in historical buildings [4]. And they are incompatible with the original rock material. These unsuitable materials were used in the preservation of some historical buildings and initiated several irreparable damages to the structural elements of these buildings [5]. A novel, innovative, reversible and appropriate techniques and materials are required for conserving and consolidating these inimitable structures.

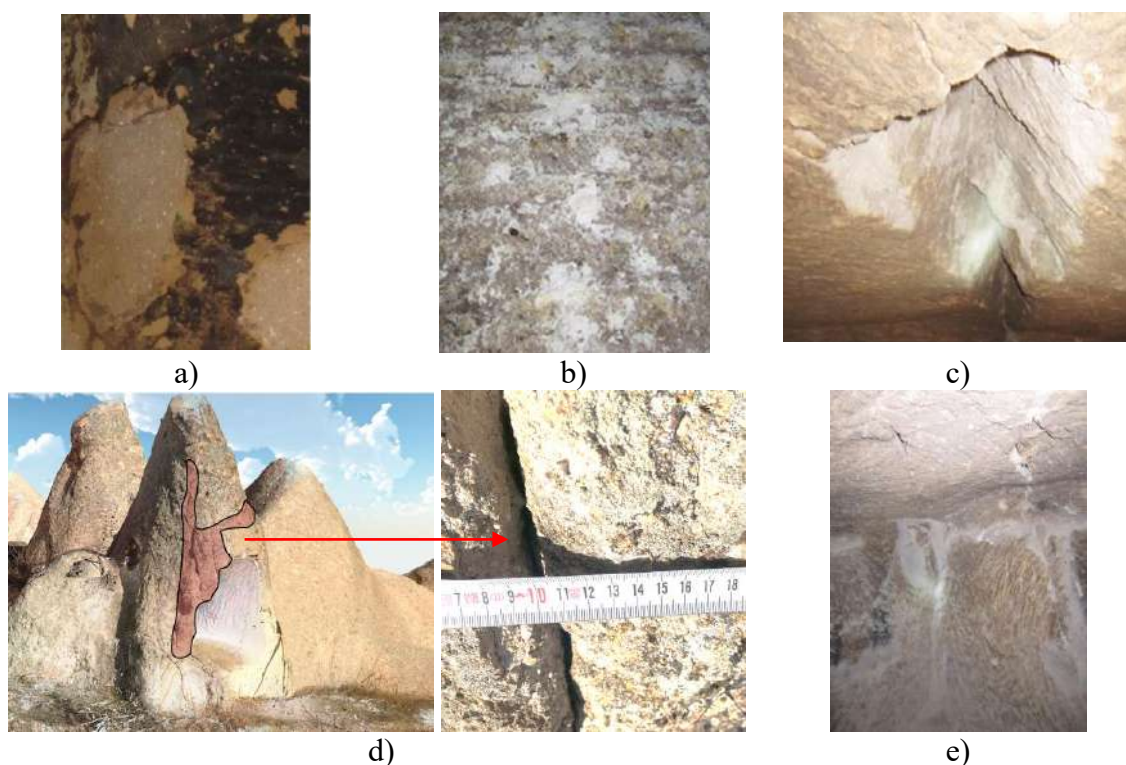


**Fig. 3.** Damages a. Crust b. Efflorescence c. Spalling d. Crack e. Repair damages and inappropriate material usage [1, 5, 6, 7]

## 2.2. Materials and Methods

In the Cappadocia Region, the rocks used to constitute a rock-cut or masonry buildings are generally divided into three groups according to local building masters' definition. These are stone rock, yellow rock, and clayey rock [1].

Rock samples named S1 and S2 are collected from Ortahisar and S3 is taken from Bahceli Village in Cappadocia. Numerous former test results [1,6] were investigated for explaining the material characterization of these rocks.

### 2.2.1. Compression Strength Test

In the Cappadocia Region, the rocks used to constitute rock-cut or masonry buildings are generally divided into three groups according to the definition of local building masters. These are stone rock, yellow rock and clayey rock. S1 is a clayey rock. S2 and S3 are in the yellow rock group. These two types of rocks are selected for investigation because they are the most common in use to carve a warehouse.

Uniaxial compression strengths were determined with the methods defined in TS EN 1926[8]. The group of rocks that is constituted by considering rocks' compressive strength is commonly divided into two according to the description of the International Society for Rock Mechanics [9]. They were shared in Table 1.

**Table 1** Uniaxial compressive strength (MPa) and description of rock samples

| Notation | Compressive Strength, MPa | Description Code* | Description Term* | Strength Range, MPa * |
|----------|---------------------------|-------------------|-------------------|-----------------------|
| S1       | 3,13                      | R1 (1-5)          | Very weak         | 1-5                   |
| S2       | 9,55                      | R2 (1-5)          | Weak              | 5-25                  |
| S3       | 14,23                     |                   |                   |                       |

\*Strength range, quantitative description code and term of stone according to the International Society for Rock Mechanics [9]

### 2.2.2. Petrographic Characterization

Petrographic characteristics of S1, S2 and S3 samples were investigated through microscopic and macroscopic analysis. The results are as the following.

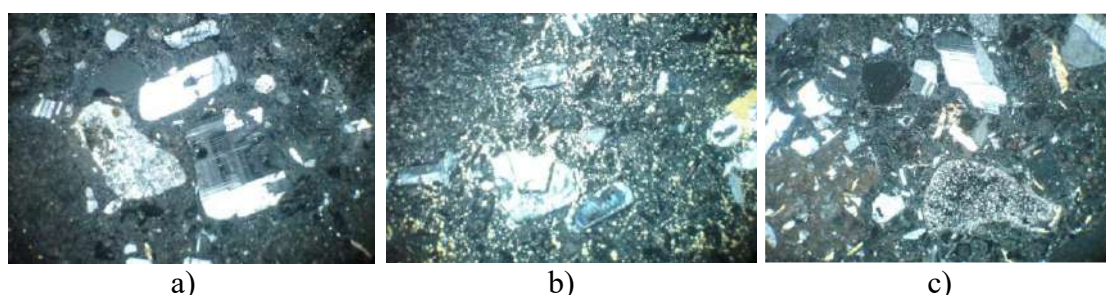


**Fig. 4.** Photomicrograph of (a) S1 (b) S2 (c) S3 (double nicol, width 4mm)

S1 macroscopic- microscopic analysis: It has fine-medium grain sizes and a speckled appearance due to black and beige minerals. It is very breakable. Weathering on the rock is seen with naked eyes. There was no reaction when an aqueous solution with HCl dripped onto the sample as a sign of no or too little amount of calcite and dolomite (Fig. 4a).

Sample in Fig. 4a is a volcanic type rock. It has a matrix (binder) that consists of volcanic glass and mineral-crystals and it has a volcanic tuff texture. It contains a slight amount (% 30-35) of mineral. They are quartz, sanidine, plagioclase, and biotite. Plagioclases are coarse-grained.



S2 macroscopic- microscopic analysis: The sample is very breakable and its hardness is variable. There is no sign of carbonate group minerals.

This rock sample has a volcanic porphyritic-glassy tuff texture, contains a slight amount ( 30-35 %) of minerals and aggregates, more of it is the matrix. Plagioclase, amphibole, quartz low, sanidine (a type of K- feldspar), and biotite forms a big part of aggregates. Most of the matrix possesses a similar characteristic with volcanic glass. Volcanic glass has secondary conversions (Silica-mica-chlorite conversion).

S3 macroscopic- microscopic analysis: The sample has a medium grain size and is very breakable. It does not contain calcite and dolomite.

The sample has a volcanic porphyritic texture. The proportion of matrix is around 75-80 percent and includes microlites and volcanic glass. Aggregates consist of quartz, quartz low, plagioclase, biotite, amphibole, and sanidine.

S2 and S3 samples have porphyritic texture and this may induce or accelerate the process of deteriorations like crumbling, weathering, spalling and exfoliation.

### **2.2.3. Chemical and Mineralogical Characterization**

X-Ray Fluorescence (XRF) and X-Ray Diffraction(XRD) methods were used to identify chemical and mineralogical features of three different rocks. X-ray diffraction patterns were collected in the range of 5°–74° (2 $\theta$ ) with a step size of 0.019° (2 $\theta$ ) and a measuring time of 1 s=step. The major oxide amounts of samples were supplied with XRF analyses as seen in Table 2. Mineral ratios of the rock samples were obtained with XRD analyses as shown in Table 3.

According to data in Table 2, the SiO<sub>2</sub> amount is the highest of major oxides in the three igneous rock samples. Igneous rocks are classified on the basis of silica content. Felsic rocks have over 65 percent silica. Therefore, all samples are acidic due to their SiO<sub>2</sub> ratio.

The silicate minerals depend on factors such as silica concentration in the melt, the presence and concentration of other elements such as aluminum, iron, magnesium, calcium, and potassium within the melt, the temperature and pressure at which crystallization takes place[10]. It is foreseen that mainly albite (a kind of Na-feldspar), opal and quartz minerals, silicates, contain silica constitute the SiO<sub>2</sub> amount in the samples. Al<sub>2</sub>O<sub>3</sub> has the second-highest ratio in the major oxides of the samples. It is supposed that albite, illite, and amorphous minerals constitute the Al<sub>2</sub>O<sub>3</sub> ratio. Relative low amount of Al<sub>2</sub>O<sub>3</sub> in the S3 sample may be caused by the absence of illite in this rock (Table 3).

**Table 2** Major oxide ratios of S1, S2 and S3

| Sample                         | S1 (wt.%) | S2 (wt.%) | S3 (wt.%) |
|--------------------------------|-----------|-----------|-----------|
| SiO <sub>2</sub>               | 69.08     | 72.20     | 66.10     |
| Al <sub>2</sub> O <sub>3</sub> | 14.18     | 11.91     | 9.76      |
| Fe <sub>2</sub> O <sub>3</sub> | 2.80      | 1.55      | 5.02      |
| MgO                            | 1.01      | 0.34      | 0.23      |
| CaO                            | 2.64      | 1.58      | 1.05      |
| Na <sub>2</sub> O              | 1.98      | 1.19      | 1.27      |
| K <sub>2</sub> O               | 3.80      | 4.69      | 4.40      |
| TiO <sub>2</sub>               | 0.27      | 0.19      | 0.16      |
| P <sub>2</sub> O <sub>5</sub>  | 0.09      | 0.03      | 0.02      |
| MnO                            | 0.09      | 0.06      | 0.05      |
| Loss of ignition               | 3.83      | 5.67      | 8.27      |

**Table 3** Mineral ratios of S1, S2 and S3 rock samples

| Sample | Opal | Quartz | Albite | Illite | Amorphous Mineral | Smectite |
|--------|------|--------|--------|--------|-------------------|----------|
| S1     | 10   | 15     | 45     | 10     | 10                | 10       |
| S2     | 5    | 5      | 75     | 5      | 10                | -        |
| S3     | 75   | 5      | 10     | -      | 10                | -        |

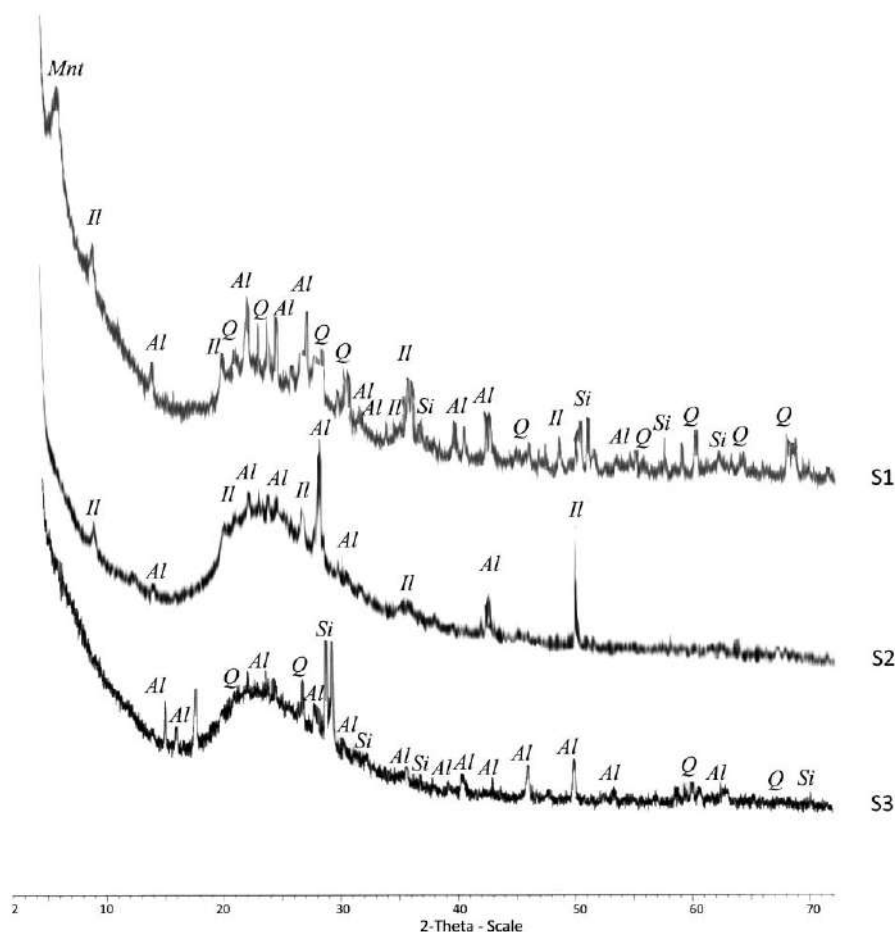


**Fig. 5.** The Mineralogical composition of S1, S2 and S3 (Al: Albite, Il:Illite, Mnt: Montmorillonite, Si: Silicon Oxide, Q: Quartz)

Opal is a tectosilicate and varies from essentially amorphous to partially crystalline. Quartz is another tectosilicate and occurs in nearly all silica-rich sedimentary, igneous, and metamorphic rocks. Quartz in the samples has cryptocrystalline (formed of microscopic crystals) (Fig. 4-5). Albite, which is observed in all samples, is feldspar and a rock-forming mineral [10]. This mineral is a member of both the plagioclase and the sodium- and potassium-rich feldspars. Na and K amounts of samples support XRD results ( Fig. 5).

S1 sample contains smectite, which is known as swelling clay (Tablo 3). According to XRD analyses (Fig. 5 ), only the S1 has montmorillonite, which has the ability of swelling. Samples do not contain calcite or dolomite according to macroscopic analysis. Illite appears to be derived from the weathering of feldspar. Since all silicates contain silicon and oxygen, to observe silicon oxide in S3 and S1sample (Fig.5) is expected. Illite has the feature of swelling due to water or cation exchange but the illite ratio of the samples is

around 5-10 percentage. Due to little amount of illite, illite originated swelling possibility is low for the samples, but should not be ignored.

All chemical and mineralogical results support the petrographic characterization in the samples.

#### 4. Discussion and Conclusion

The two main problems faced in the Cappadocia Region. The first one is damages especially cracks and spalling and incompatible intervention materials used on the historical structures. The second one is arising waste material that could not be evaluated efficiently and cause visual pollution.

The compression strength of the rocks that are used to construct rock-cut or masonry buildings is generally between 0-5 MPa and 5-25 MPa. Only very rare used rocks have compressive strength bigger than 25 MPa and its compressive strength is 31.77 MPa [7]. These rocks are weak and/or very weak rocks. Therefore, producing novel materials to consolidate damaged structures is not hard in the context of compressive strength.

On account of cracks and spalling type damages, deteriorations like efflorescence, exfoliation, biological colonization, and even weathering are triggered as a secondary effect of the former damage. It is obvious that to prevent these secondary deteriorations, the main problems must be solved. On-site studies and 3d models of structures shows detailed damages like Cracks, spalling, and missing parts on the structural elements [5]. Studies and investigations shows that damaged structures must be rehabilitated and consolidated urgently.

The rock, which is the main material of almost all structures in Cappadocia until the beginning of the 20<sup>th</sup> century and the main source of excavation waste nowadays contains the solution within itself for up-to-date and long-lasting conservation solutions. Limited previous studies [2,3,17] about Cappadocia tuff as a natural pozzolan and the pozzolanicity of waste material caused to investigate waste properties by examining the rock, which is the source of it.

A pozzolan is generally defined in ASTM C618 [11] as a siliceous or siliceous and aluminous material which, in itself, possesses little or no cementitious value but which will react chemically with calcium hydroxide (lime) at ordinary temperature to form compounds possessing cementitious properties. A quantification of this capability is comprised in the term pozzolanic activity [12].  $\text{SiO}_2 + \text{Al}_2\text{O}_3 + \text{Fe}_2\text{O}_3$  amount that is bigger than 70% in a material refers to the material's pozzolanic activity according to TSE-25 [13] and ASTM-C618 [11]. The total percentage of these three oxides in all samples is at least 80 percent that indicates materials have pozzolanic activity.

Amorphous minerals and phases in the material's make-up are essential for pozzolanic activity [14]. The volcanic porphyritic-glassy tuff texture of the samples and the high amount of volcanic glass in the matrix refers to pozzolanic activity. Although the percentage of amorphous minerals is 10%, minerals such as opal are also often in amorphous form. These mineralogical and chemical features are the indicators of the possibility of using the waste of excavated rocks as a natural pozzolan.

Results that are obtained from this study support the former studies that were done for a limited area of Cappadocia. However, detailed pozzolanic activity research for the Cappadocia region is needed.

Slaked lime and the addition of natural pozzolans to produce stronger and more durable lime mortars have been used on structures since before the common era. It is not clear when and where mortar technology evolved to incorporate volcanic type natural pozzolans as a functional supplement. More recently, Portland cement that can react with lime to produce a cementitious product has received renewed attention [12]. The production of Portland clinker is energy-intensive, accounts for 8% of the worldwide



carbon emission due to the energy used in its production [15] and renders the economic advantages of replacing a substantial part of the clinker with cheap naturally available pozzolans. Besides, it causes various irreversible damages when used in historical structures [4].

Besides using natural pozzolans as a supplement of slaked lime in a conservation project, consuming them as the main material of a renewed or rehabilitated binder and/or stone will benefit more. The innovative materials like geopolymers or alkali-activated materials are inorganic aluminosilicate binders that have polymeric silicon-oxygen-aluminum framework structures [14]. According to mineral compositions and oxides of the excavated waste of Cappadocia natural pozzolan (CNP) presented in Table 2-3 and Figure 5, it can be used as a precursor for producing an innovative inorganic polymer.

According to studies of Barone et. al [16], inorganic polymers are versatile materials that are chemically and mineralogically compatible with traditional stone substrates. When the silicate and aluminum contents are appropriately balanced with alkali cations and water, a polymeric gel will form at room temperature (Barone et. al, 2020). Cappadocia natural pozzolans (CNP) are suitable precursors for promising high-tech sustainable and useful materials for conservation-restoration of built cultural heritage. It is possible to design a mortar, binder, or cast stone. Although some inventive studies realized by Özata et. al. [17] about CNP alkali activation, further studies are needed to investigate the microstructural properties of these new materials mixes and to assess their applicability on-site, in close cooperation with local enterprises, conservators, stakeholders.

Consequently, inorganic polymer production by using CNP can reduce the carbon footprint and environmental impact of industrial production of conservation-restoration products, and increase the general sustainability of the intervention.

### Acknowledgement

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## SELECTED SOLUTIONS FOR THE CONSTRUCTION OF NAZI BUNKERS LOCATED IN THE WOLF'S LAIR AREA IN TODAY'S POLAND

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### ***Abstract:***

The technology of erecting war bunkers has been evolving over the centuries alongside the building industry was developed. New construction solutions and the results of scientific research were used in the construction of these facilities. Many papers have been published on this subject in recent times. The ruins of the Wolf's Lair war fortress prompted the authors to write this work. This set of military facilities was perfectly adapted to the natural environment, so that it could fulfill its tasks to the maximum. After building it was rebuilt and strengthened twice, each time becoming stronger and stronger. The debris left over after blowing up the objects perfectly present the construction solutions used and their advantages.

This paper presents the design guidelines used in their construction and describes the construction solutions applied. In addition, where appropriate, the advantages and disadvantages of these construction solutions from an engineering point of view are drawn.

***Key words:*** bunkers, defence construction, reinforced concrete, war fortification

### **1. The use of concrete in the construction of war fortifications**

The technology of building war fortifications has evolved over the years. The development of artillery, and in particular the use of threaded barrels, increased the range of guns, and new explosives increased artillery fire power. Initially, attempts were made to solve this problem by surrounding the fortresses with barrier forts, which made it difficult for enemy troops to get under the embankments of the proper fortress and increased the distance of artillery fire. Over time, the forts were dispersed over a large area, but were located at such a distance from each other to maintain the ability to fire the opponent between them. However, forts have been erected using materials known for centuries all the time – brick structures were surrounded by earthen ramparts and brick embankments.

At the end of the nineteenth century, a few gun-resistant domes appeared on shelter ceilings these were cast from steel. However, steel is an expensive material, and although the casting technique of large cast iron and steel castings was developed a long time ago,

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the transportation and assembly of multi-ton domes has given rise to many problems, especially since the forts were often erected in terrains that were difficult to access.

A major milestone in the development of fortification was the development of Portland cement production technology in 1880. As a result of this, concrete allowed the mould to get any shape and significantly increased the resistance of objects to artillery fire in relation to brick and earth buildings. For comparison, the resistance to an explosion provided by a 300 mm concrete layer on the ceiling of a shelter corresponded to the resistance offered by a 2 m thick earth embankment. As a result of the explosion of a 155 mm demolition grenade, a concrete funnel with a depth of 0.6 m and diameter of 2 m is formed.

Concrete turned out to be a very practical material. It could be formed into any shape depending only on the formwork used. Initially, concrete began to be used to strengthen ceilings, replacing the existing protection in the form of a layer of soil. Attempts were made to use it also in the construction of arched ceilings and in the construction of retaining walls stabilizing the soil fortifications. However, the shortcomings of plain concrete soon became apparent. Concrete actually protected against the explosion of an artillery shell, but due to its low ductility, it cracked, not only as a result of fire, but also as a result of temperature differences, especially during frost. Plain concrete during explosion posed an enormous threat to people inside the shelter, where heavy debris from walls and ceilings was torn off from the inner surface and wounds and buried people. Initially, this was attempted to be resolved by strengthening the vaults from the bottom with corrugated steel sheeting; however, it was more effective to pour the ceilings on a composite structure of steel T-beams arranged at short distances with separators between them in the form of a filler steel plate. The concrete ceiling on the steel beams became such an effective solution that it became an important element of fortification architecture for many years.

However, a breakthrough in the construction of fortifications exposed to artillery fire or aerial bombardment was the use of reinforced concrete, i.e. concrete, the internal structure of which was reinforced with a mesh of steel bars (usually 10-20 mm in diameter). Reinforced concrete had greater strength and was more elastic than plain concrete. As a result of its advantages, RC became the basic material for the construction of permanent military fortifications before World War I. It made a big difference, because this war first revealed the threat of air bombing. Reinforced concrete offered the best prospects for the protection of soldiers, and as it turned out later, also for the civilian population against the effects of using this new weapon.

France led the way in the field of the theory of fortification. Here, field test of the construction and firing tests (Fig. 1) were carried out. The first norms shaping the formula of the concrete mixture were created here.

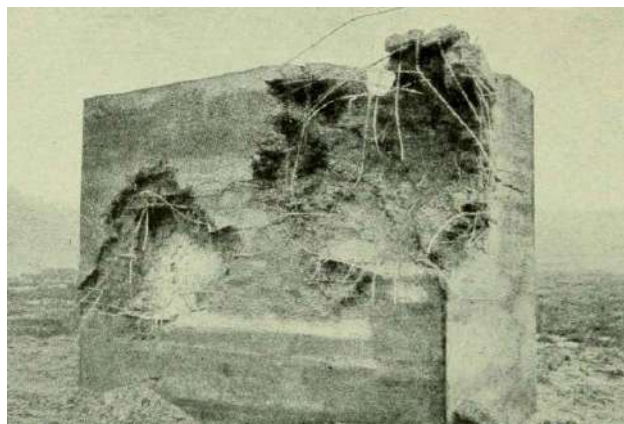


**Fig. 1.** Polygon test on the 1830 mm (6 ft.) thick reinforced concrete wall [3]

The French General Staff claimed that the construction of the mighty and extensive fortifications of the Maginot Line would protect their country from the greatest threat-the German army. This was the reason why the French focused on improving the building technology. Other European countries followed them, including Germany and Belgium. Table 1 shows the composition of the concrete mix used in the aforementioned countries for the construction of war buildings published in the paper [1]. In later years, the composition of concrete mixtures underwent changes due to technical progress and the development of construction techniques. Changes resulting from the geological structure of aggregates obtained locally from sources near to the construction were also introduced.

**Table 1** Composition of the concrete mixture and the amount of steel used to construct fortifications in European countries in the early nineteen-twenties and early nineteen-thirties [1]

| Material                                                 | Germany | Belgium | France |
|----------------------------------------------------------|---------|---------|--------|
| Cement, kg/m <sup>3</sup>                                | 275     | 400     | 400    |
| Sand, m <sup>3</sup> /m <sup>3</sup>                     | 0.40    | 0.40    | 0.30   |
| Aggregate (broken stone), m <sup>3</sup> /m <sup>3</sup> | 0.90    | 0.90    | 0.90   |
| Reinforcement, kg/m <sup>3</sup>                         | 80-120  | 70      | 80     |

In the early nineteen-twenties and early nineteen-thirties of the twentieth century, strength and resistance standards of field fortification facilities were established. These classifications were mainly based on the determination of the resistance of walls and ceilings to the firing of demolition ammunition of a given calibre. Important data derived from the French guidelines for the classification of combat shelters is given in Table 2. This data was used by many countries at that time. The measure necessary for the classification of fortification objects was the reinforced-concrete penetration parameter. The relationship between the necessary thickness of the wall or the reinforced-concrete ceiling and the resistance to artillery fire is described by the following formula [4]:

$$S = \frac{10 \times \text{shell caliber}}{2} \text{ [mm]}$$

This should be interpreted as follows: a 200 mm diameter pierced a 1 m reinforced concrete wall or ceiling. Standards and classifications relating to resistance to fire provided the required construction parameters according to the object element.

**Table 2** French classification of combat shelters [4]

| Shel diameter                        | 380-420 mm     |                     | 305-380 mm     |                     | 210-305 mm     |                     | 150 mm         |                     |
|--------------------------------------|----------------|---------------------|----------------|---------------------|----------------|---------------------|----------------|---------------------|
|                                      | Plain concrete | Reinforced concrete | Plain concrete | Reinforced concrete | Plain concrete | Reinforced concrete | Plain concrete | Reinforced concrete |
| Front walls, m                       | 3.0-3.5        | 2.5-3.0             | 2.5-3.0        | 1.75-2.5            | 2.0            | 1.5                 | 1.5            | 1.2                 |
| Ceiling slab (with span of 3-4 m), m | 2.5            | 2.5-3.0             | 1.75           | 2.1-2.5             | 1.3            | 1.5                 | 1.0            | 1.1                 |



## 2. Wolf's Lair as Adolf Hitler's war headquarter

During World War II, a dozen or so fixed headquarters of the occupying forces were established in Europe, serving as command centres for the war front. They were usually found in the mountains and in areas surrounded by lakes and marshes; they were always near the fronts in areas difficult to penetrate. On the whole, they were powerful, massive reinforced-concrete buildings and bunkers. The most famous and most frequently discussed military quarters is Wolf's Lair, located in the Kętrzyn Forest in Masuria (near the former border with the Soviet Union – Fig. 1). The construction of Wolf's Lair was entrusted to the "Todt" organisation, which under the guise of building the "Askarnia" chemical plants, began construction work in the autumn of 1940. The construction and extension of the fortress can be divided into three stages: 1940-41, 1942-43 and 1944.

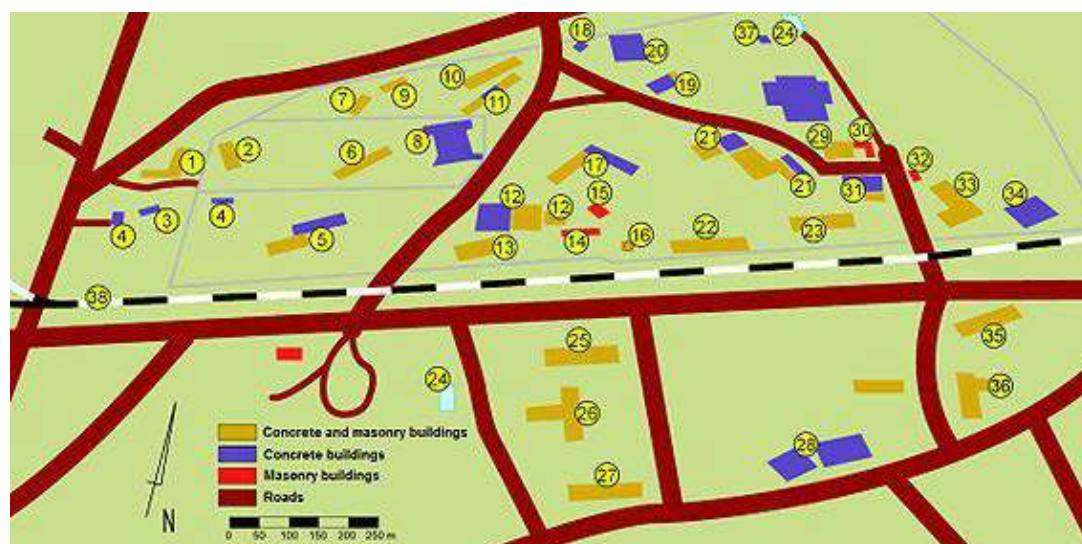
The construction of headquarters started with the modernisation of the railway station in Kętrzyn, the construction of a large platform, railway sidings and utility infrastructure. The forest road to Kwidz (Queden) was paved and the road to Kętrzyn was asphalted. The lifetime of Wolf's Lair was planned for the Blitzkrieg led against the Soviet Union, which was scheduled to start at the Beginning of May 1941. At the time of Hitler's arrival at Wolf's Lair (24.06.1941), the quarters consisted of eight single-storey, terrestrial concrete bunkers and several wooden and masonry buildings. Some of these were equipped with metal doors and shutters (in case of an air attack) and rubber seals (in case of a gas alarm).

In the summer of 1942, as a consequence of the prolonged war with the Soviet Union, the 2<sup>nd</sup> stage of construction commenced. Due to the growing number of persons occupying the quarters, wooden barracks were mostly built – these housed office and residential areas. Due to the risk of air raids, some of these were strengthened with reinforced concrete.

In April 1944, due to the approaching eastern front, the third phase of the extension commenced in Wolf's Lair. Some of the existing concrete buildings were strengthened at that time: Hitler's shelter, the guesthouse (Fig. 2 - No. 8), two teleprinter terminals. Other shelters were constructed from scratch: for Goering (Fig. 2 - No. 34), for Martin Bormann (Fig. 2 - No. 11) and for public use. The above-mentioned objects were created on the principle of double walls and ceilings, between which, there was 700 mm of space filled with basalt grit or gravel. In the event of bombing, these would absorb the shock. The complex of facilities built in the years 1940-44 occupied an area of 250 hectares. In the total, there were about



Fig. 1. Location of Wolf's Lair [6]



- |                                                                                                   |                                                                      |                                                                                            |
|---------------------------------------------------------------------------------------------------|----------------------------------------------------------------------|--------------------------------------------------------------------------------------------|
| 1. Office and barracks of Hitler's bodyguards                                                     | 14. Railway siding for Hitler's Train                                | 27. RSD command post                                                                       |
| 2. FBK/RSD command post                                                                           | 15. Cinema                                                           | 28. Air-raid shelter with Flak and MG units on the roof                                    |
| 3. Emergency generator                                                                            | 16. Generator buildings                                              | 29. Hitler's bunker and air-raid shelter                                                   |
| 4. Bunker                                                                                         | 17. Quarters of Morell, Bodenschatz, Hewel, Voß, Wolff and Fegelein  | 30. New tea room                                                                           |
| 5. Office of Otto Dietrich, Hitler's press secretary                                              | 18. Stores                                                           | 31. Residence of General Field Marshal Wilhelm Keitel                                      |
| 6. Conference room                                                                                | 19. Residence of Martin Bormann                                      | 32. Old tea house                                                                          |
| 7. RSD command post                                                                               | 20. Bormann's personal air-raid shelter for himself and staff        | 33. Residence of Reich marshal Hermann Göring                                              |
| 8. Guest bunkers and air-raid shelter                                                             | 21. Office of Hitler's adjutant and the Wehrmacht's personnel office | 34. Göring's personal air-raid shelter for himself and staff, with Flak and MG on the roof |
| 9. RSD command post                                                                               | 22. Military and staff mess II                                       | 35. Office of the High Command of the Air Force                                            |
| 10. Secretariat under Philipp Bouhler                                                             | 23. Quarters of General Alfred Jodl, Chief of Operations of OKW      | 36. Offices of the High command of the Navy                                                |
| 11. Headquarters of Johann Rattenhuber, SS Chief of Hitler's security department, and Post Office | 24. Friefighting pond                                                | 37. Bunker with Flak                                                                       |
| 12. Radio and telex buildings                                                                     | 25. Office of the Foreign Ministry                                   | 38. Rastenburg railway line                                                                |
| 13. Vehicle garages                                                                               | 26. Quarters of Fritz Todt, then Albert Speer after Todt's death     |                                                                                            |

**Fig. 2.** Layout of Wolf's Lair [7]

200 objects of different sizes and uses, including 7 heavy anti-aircraft shelters, dozens of masonry and concrete buildings and dozens of wooden buildings.

In October 1944, the Red Army troops reached the eastern borders of East Prussia. It could be assumed that its next target would be, above all, Wolf's Lair. For this reason, on 20 November 1944, the headquarters were transferred to Zossen, near Berlin. Two days later, on 22 November 1944, Field Marshal W. Keitel issued an order to blow up Wolf's lair. The bombing of the shelters and barracks under the code name "Inslsprung" was performed on the night of January 24<sup>th</sup>, 1945. These were the strongest explosions that shook the immediate surroundings. The large concrete blocks flew 20-30 meters. According to witnesses, shocks caused ice to crack in the nearby lakes. Polish sappers calculated that about eight tons of tritium had to be used to blow up one heavy shelter. The ruins of the blown-up bunkers, as well as many undamaged, lighter buildings, can now be seen in the state they were left in that January night in 1945. They only carry traces of time, and they perfectly present the construction solutions used.

### 3. Functional layout and bunker structure

War shelters were built according to a developed classification. The classification of the resistance of fortification objects was developed in nineteen-thirties Germany. This classification had four levels: A – resistance to direct artillery fire of 520 mm calibre shell and the strike of an aerial bomb of up to 1000 kg; B – fire with a shell of 220 mm calibre and a single hit with a 500 kg bomb or 300 mm calibre shell; C – fire with shells of up to 105 mm; D – protection against shattering and fire from light artillery guns. The original classification was extended in 1938 (Table 3).

**Table 3** German classification of the resistance of fortification objects to artillery fire from 1938 [4]

| Resistance class | Thickness of external walls, m | Ceiling slab thickness, m | Thickness of internal walls, m |
|------------------|--------------------------------|---------------------------|--------------------------------|
| A                | 3.5                            | $\geq 3.5$                | 1                              |
| A1               | 2.5                            | 2.2                       | 1                              |
| B-neu            | 2                              | 2                         | 0.8                            |
| B                | 1.5                            | 1.5                       | 0.8                            |
| B1               | 1                              | 0.8-1                     | 0.8                            |
| C                | 0.5-0.6                        | 0.5                       | 0.5                            |
| D                | 0.3                            | 0.3                       | 0.3                            |

The first bunkers that were built in the wolf's lair in 1940-1941 were very inconspicuous. The structural assumptions of these objects were based on the pre-war defence standards of fortification construction. These standards were quickly improved by the development of aviation. The wall with the thickness of 1.0-1.5m and two-meter ceiling were supposed to protect against the explosion of 500 kilogram bombs. Several small rooms in their interior provide the minimum of space for work and sleep. The bunkers were equipped with sanitary installations and electricity. In 1943, the effectiveness of bombing by the British and American aviation considerably increased. Their reach grew so much that they began to attack targets in places which had so far been deemed safe, including East Prussia. The threat of an attack on Hitler's main headquarter – Wolfschanze, began to be possible. In mid-1943, Hitler decided to expand the facilities of the headquarters. It was decided to reinforce the existing shelters by enclosing them with reinforced-concrete armour resistant to direct impacts of even the largest 10-ton bombs which the Allies were in possession of.

The technique of building and increasing the resistance of the shelters can be perfectly traced in the remains of the quarters in Mamerki, near Wolf's Lair. Hitler's plans to attack the Soviet Union made it necessary to find a place for a new headquarters, close to the place chosen for the war headquarters (Wolf's Lair). There was little convenient place to build a command center near Gierłoż (Wolf's Lair). There were no larger forest complexes in the area to help mask the construction, and wetlands limited the possibility of connecting roads and railway lines. A forest near the Mamerki forester's lodge, located on the shores of Lake Mamry, was selected as the construction site. It was 18 km from Wolf's Lair and 8 km from Węgorzewo. In addition to the Command Headquarters of the land forces (OKH Hauptquartier "Mauerwald") general quartermaster of land forces was located in Mamerki. In contrast to the Wolf's Lair this complex has not been blown up. In addition to well-preserved bunkers, several buildings at various stages of construction can be seen over there.

Figure 3 shows a 5-room bunker in Mamerki, made according to the guidelines of Hitler (Fig. 4). It was not an innovative structure, but a simplified and slimmed down Regelbau 31 bunker. R31 bunkers had been built earlier on the Eastern Wall. They had



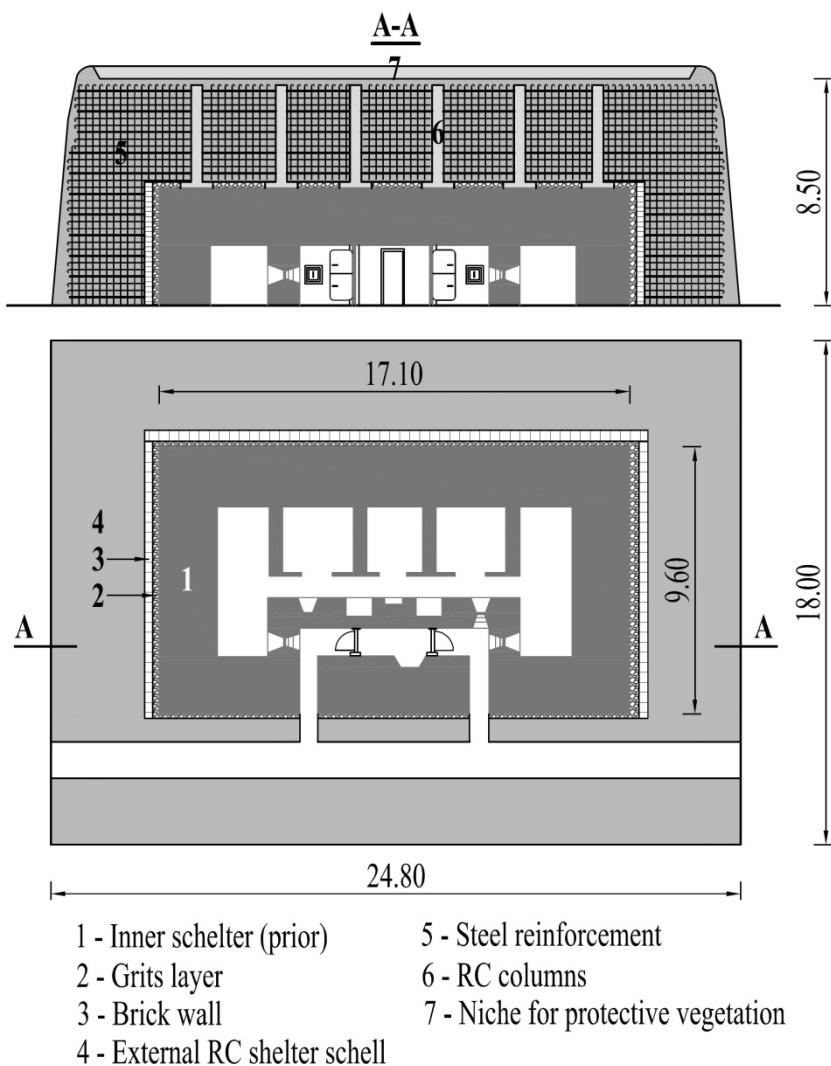


**Fig. 3.** Five-room, enclosed later bunker in Mamerki

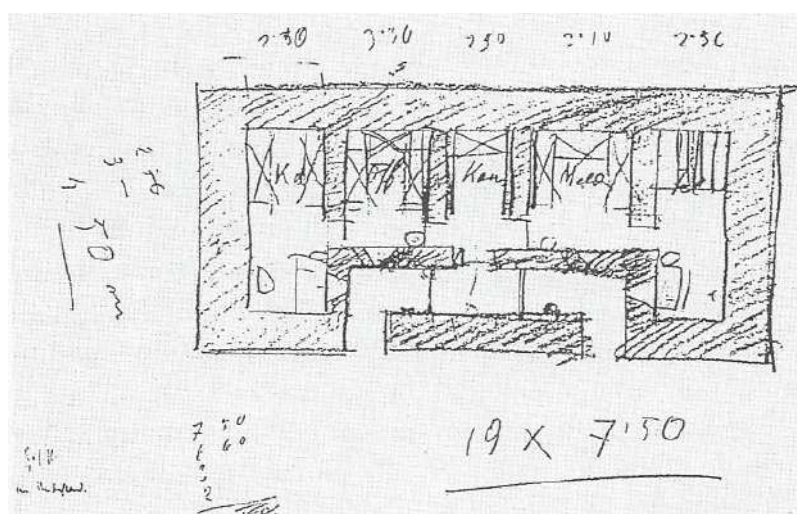


**Fig. 4** A hand-drawn sketch of a five-room shelter made by Hitler [4]

dimensions of  $19.85 \times 7.40$  m instead of  $17.10 \times 9.60$  m, that where built in Mamerki. The layout of the rooms was similar. There were also two entrances leading to the corridor, from which there was a passage to 5 rooms.

In 1944 in Mamerki, similar to the Wolf's Lair, reconstruction began to increase the resistance of the bunkers to bombing. As part of these activities, two bunkers were covered with massive, reinforced concrete linings with 3-4 m thick walls and 5 m thick ceiling (Fig. 3). The original concrete shelters were first enclosed with masonry walls. Between the concrete bunker and the brick wall, a 20-25 cm thick layer of grits was made. A system of reinforced concrete columns was made on the ceiling. It was a support structure for the steel bars reinforcement. The additional concrete ceiling was poured over the grits layer. There is a niche for masking vegetation on the roof of the shelter. The total height of the bunker was about 9 m. Similar works started at the third two-room shelter. A foundation slab around the bunker was made and the reinforced concrete columns prepared for mounting reinforcing bars were made on the top of ceiling. However, the works were not completed (the bunker was not concreted), so today it can be seen in the state in which the works were stopped.

From 1944, after the bombing of Peenemünde, several shelters in the Wolf's Lair were enclosed using a similar technology. Shelters in the Wolf's Lair were strengthened according to a similar technology. The following bunkers were enclosed: Hitler's bunker and air-raid shelter (No. 29 in Fig. 2), Bormann's personal shelter (No. 8 in Fig. 2), guest bunkers (No.8 in Fig. 2) and communication bunker (No. 12 in Fig. 2 and Fig. 5). New shelters were built according to the requirements of class "A" (Goering's personal air-raid shelter – No. 34 in Fig. 2 and Fig. 6). Wooden barracks were enclosed with bricks and

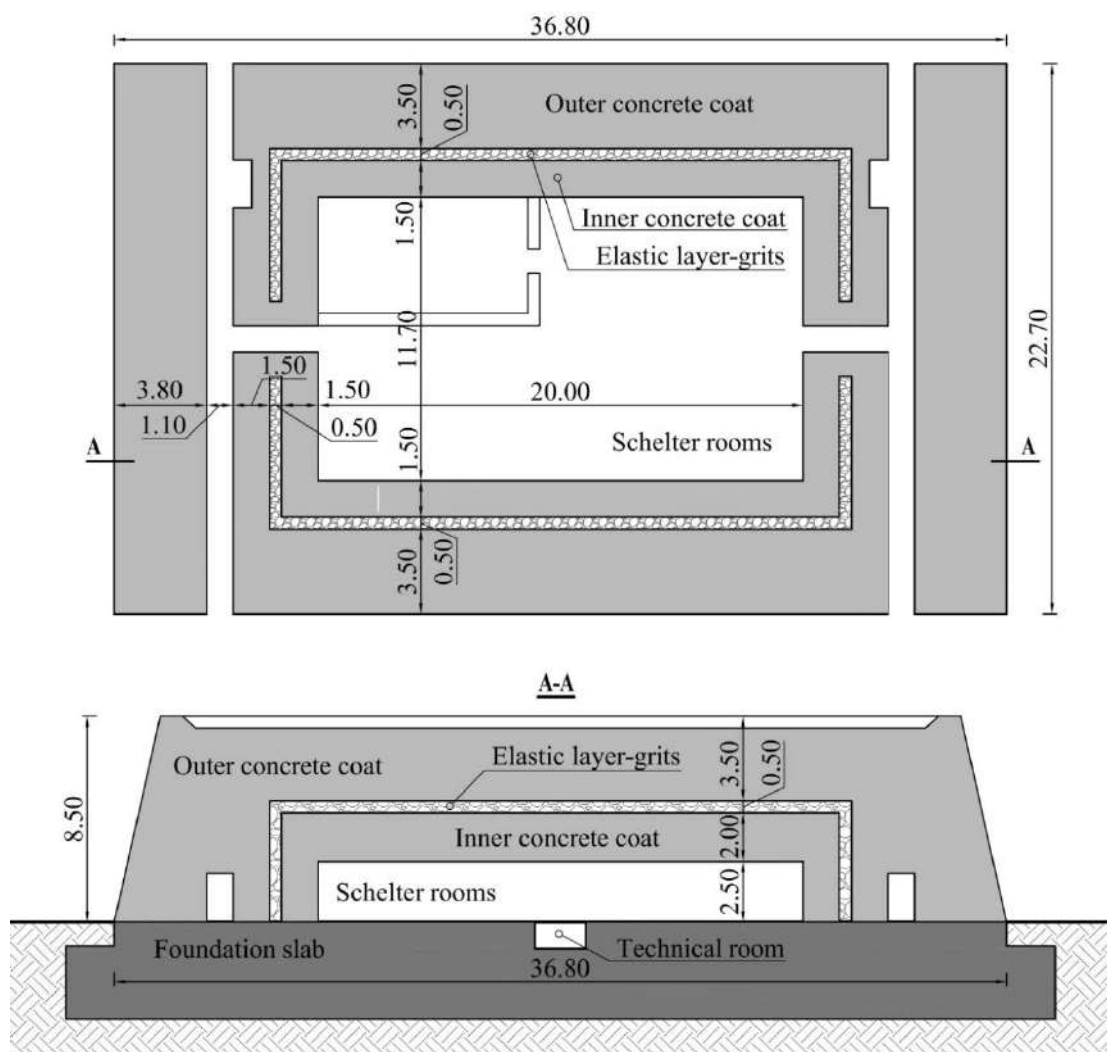


Fig. 5 The Special Communications Bunker with outer concrete coat

prestressed concrete. At the top of some of the bunkers, anti-aircraft defence posts were placed.

Much attention was paid to camouflaging objects. The outer walls were covered by a camouflage mix of sea grass, chips and cement. On the roofs, basins were filled with earth were formed and cultivated.

The strengthened Special Communications Bunker (No. 12 in Fig. 2) is shown in Figure 5. In case of this object, the entrances were at the ends of arterial corridors running through the entire width of the shelter. From there, there were corridors leading to small, internal spaces deprived of windows and with a height of 2.0-2.1 m. Entrances to the internal corridor were equipped with gas-tight armoured doors.

Between the concrete prior shelter and reinforced concrete armour coats, a layer of grit with a thickness of 0.7 m was used. Fat concrete (at least 600 kg of cement in a cubic meter) and significant amount of reinforcing steel were used for the construction of the coat.

An example of a massive bunker meeting the highest requirements of anti-aircraft protection, built in one stage, is the Goering's personal air-raid shelter (Fig. 6). It is a massive reinforced concrete structure with a projection of 27.0×19.5 m and a total height of 11.0 m. It is a one-room shelter, with one longitudinal corridor with two entrances and a third additional transverse entrance. It is a structure made of two reinforced concrete coats: internal and external. The thickness of the inner coat ceiling is 2.00 m and the outer coat is 4.50 m. Similar to the Special Communications Bunker, between the coats, an elastic layer of 0.50 m thick grits was made.

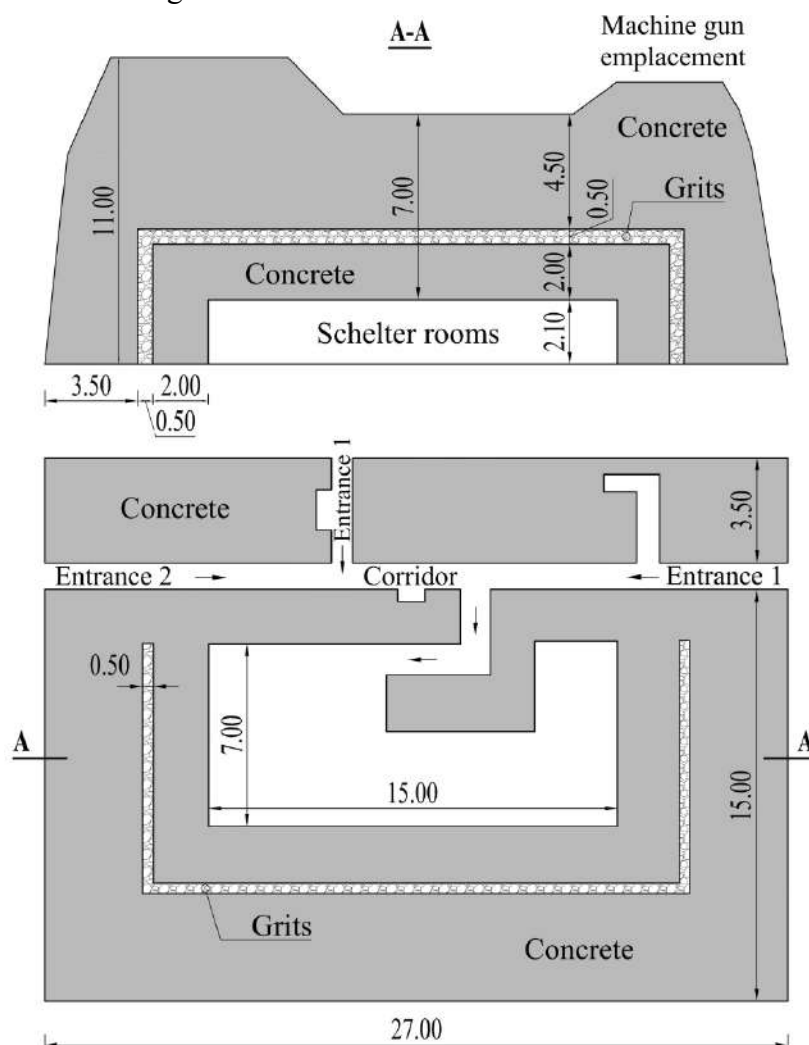


Fig. 6 Goering's personal air-raid shelter



#### 4. Construction of ceilings in bunkers

The construction of ceilings in the fortification buildings evolved along with the resistance need resulting from the development of artillery or warfare techniques. First, these were costly steel domes which were not very resistant to bomb attack and were buried in earth, brick or concrete vaults, and later secured with corrugated steel sheeting. If necessary, the roofs evolved to concrete and later, reinforced concrete cast on steel beams or whole reinforced concrete cast in a traditional wooden formwork. Below are some ceiling solutions in Wolf's Lair.

##### 4.1. Concrete slabs on steel beams

Steel "I" beams were laid parallel at a distance of 0.4-0.7 m. The space between them was filled with a several-millimetre-thick steel sheets based on lower footers (Fig. 7). Steel beams constituted both a self-supporting formwork required for the concreting stage as well as reinforcement of the lower zone of the slab ensuring its bending resistance. Such a solution allowed the pouring of a slab with a thickness of up to 2 m without additional supports (Fig. 8a). A tight steel coating protected against breakage of concrete elements from the bottom during fire or explosion. The maximum span of the ceiling was 7.2 m. Initially, a concrete slab was cast on a steel structure. With the development of reinforced



Fig. 7 Composite slab on steel beams (own photo)



a)



b)

Fig. 8 View of a thick slab on steel beams (a) and dense steel reinforcement (b) (own photos)

concrete, a spatial net was used to provide horizontal reinforcement in both directions and in the vertical direction (Fig. 8b) thus giving the composite slab.

#### **4.2. Reinforced concrete slabs on wooden formwork or on corrugated steel sheeting**

After the dissemination of reinforced concrete and appreciating its advantages, today's reinforced concrete slabs are commonly used in wooden formwork. Such ceilings were constructed in small defensive structures. Their spans didn't exceed 5 m. Wooden formworks – often in fragments – have been left visible in the interiors of buildings until today (Fig. 9). A better type of formwork was corrugated steel sheeting used as permanent formwork. The use of plates from the underside provided additional air tightness and limited the fall of concrete fragments.

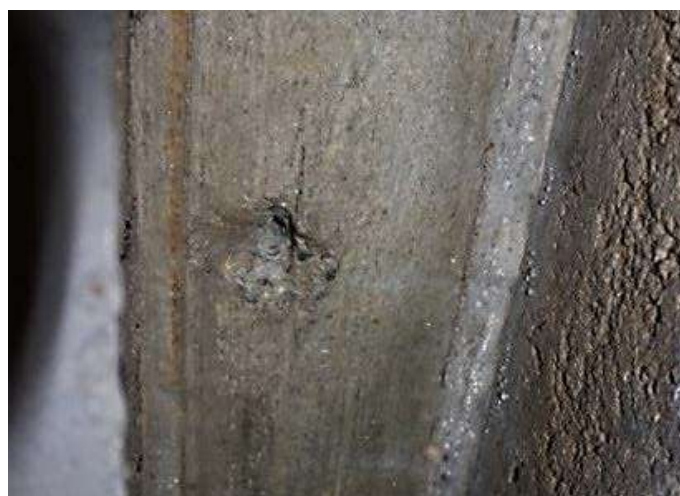


**Fig. 9** View of left fragment of wooden formwork (own photo)

#### **4.3. Ceiling constructed on pre-tensioned concrete beams – the first application of prestressed concrete on Polish land**

Another solution in the construction of ceilings was casting the slab on pre-tensioned concrete beams. This technique of making ceilings was used to strengthen wooden barracks in the Wolf's Lair. The modern form of strengthened concrete was actively used in the construction industry in the 1930s. This was the so-called technology of active reinforcement where preliminary compressive stresses were applied to the concrete, acting against tensile stresses extending from external loads. The prior stress diagram is introduced by tensioning wires of low-relaxation steel, which are able to maintain tension for a long time. Throughout the lifetime of the structure, through the adherence of the wires to the concrete (pre-tensioning), or through additional anchorage at the heads of the element (posttensioning), the tensioned tendons act on the concrete element with the force with which they have been tensed. The force from the tendon, through its eccentric application in relation to the centre of gravity of the element, causes an inverse state of bending stresses generated by the external loads.

T-section pre-tensioned beams were already used in the thirties to make ceilings with a span of several dozen meters – this was necessary for the protection of high volume armour (e.g. U-Boats). Due to the considerable advantage of prestressed concrete structures in terms of bearing capacity relative to other solutions, they have also begun to be used in smaller but strategically significant command centres. This is undoubtedly the first application of prestressed concrete on Polish land in the Wolf's Lair complex.

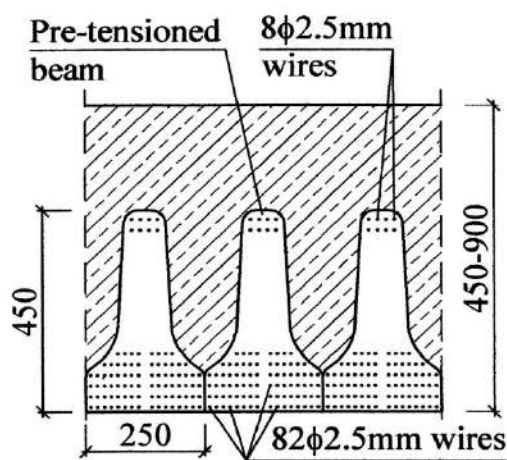
During the last stage of modernisation of the fortress (from 1944), pre-tensioned concrete beams were delivered from Cologne and used to reinforce wooden buildings (Fig. 10). The wooden walls were replaced with brick walls. Beams with an inverted T-shape had a maximum span of 12 m and a height of 0.45 m. Available for research study by the authors, the sections of the beams were prestressed with 82 wires with a diameter of 2.5 mm, placed in the lower footer with a width of 0.25 m and 8 wires in the upper section of the web. Figure 11a shows the cross-section of the beam inverted by 180 degrees relative to the position of mounting into the ceiling, and Figure 11b is the cross-section of the ceiling. The composite slab had different thickness depending on the span and the required load capacity for a given building. Height of the beams was in the range from 450 mm to the total height of about 1.2 m. Depending on the expected load on the slab and its span, different spacing of beams were used. With more prestressing required, the beams were laid for contact (Fig. 10), one next to the other. With smaller spans and slab thicknesses, the beams were parted and space between them was supplemented with shuttering boards placed on the lower footers (Fig. 12).



**Fig. 10** View of the ceiling with pre-tensioned beams and concrete topping (own photo)



a)



b)

**Fig. 11** View of pre-tensioned beam cross-section (a) (own photo), slab cross-section (b)





**Fig. 12** Different spacing of pre-tensioned beams depending on slab load and span  
(own photos)

### **6. The current conservation status of objects**

Most of the heavy concrete shelters in the Wolf's Lair fortress were either completely or partially blown up. While many similar shelters in Mamerki are intact and present their form perfectly (Fig. 13). The steel elements of the ceilings actually show signs of corrosion, but the condition of the concrete is satisfactory. The technology used in the construction of the facilities was adequate and thus, the objects have survived intact for almost 80 years.

On the other hand, the debris in the Wolf's Lair shows the design solutions use. The nature of the ruins of the blown-up shelters shows the strength of the structure and also the difficulty and huge investment cost of their destruction mentioned in Section 2. Many of the walls of the bunkers have survived almost completely or with few cracks, despite being subjected to strong explosions. Solid concrete walls often rest on great concrete blocks (Fig. 14). The amount of explosives needed to separate them and shift them to great distances was impressive. Among others things, a concrete fragment of the Goering's bunker "supported by branches" looks interesting (Fig. 15).

Figure 16 shows the role of reinforcement in structural resistance to explosion. Fragments of walls were actually moved several dozen centimeters but were not separated. In Figure 16b, this is particularly noticeable, the surface of the fraction on the left fragment corresponds exactly to the surface on the right-hand fragment. This demonstrates the fragmentation and elongation of the concrete reinforcing bars. Such ductility of reinforced concrete structures is certainly achieved by the use of smooth rods that lose more adhesion over longer distances than ribbed rods and thus offer greater elongation. This example shows the advantage of smooth reinforcement over ribbed in explosion-prone objects and the need for reinforcement of high ductility.

It is also worth to note the durability and resistance of composite ceilings to explosions, which in many cases have been preserved as a homogenous form or large sections of composite slab. The strength of the bond (especially for reinforced concrete slabs) was achieved by using steel beams and spatial internal reinforcement.



**Fig. 13** The entirely preserved heavy bunkers (a) and building of electric transformers (b) in Mamerki.



**Fig. 14** View of separated but entirely preserved wall of heavy bunkers (own photo)



**Fig. 15** Separated concrete block of the Göring's bunker (own photo)



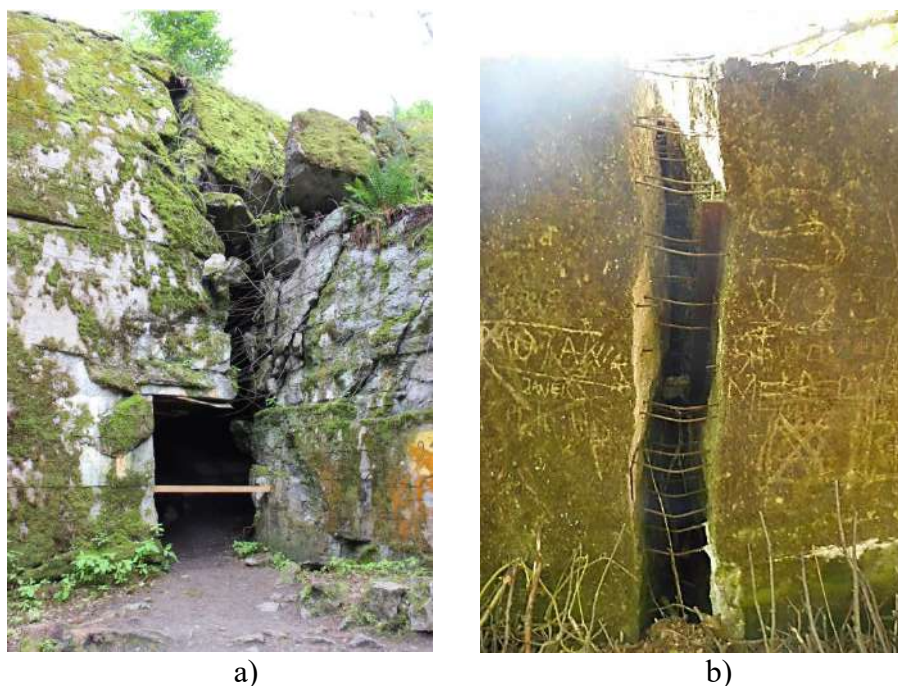


Fig. 16 View of the importance of reinforcement in ensuring ductility and compactness of concrete structures (own photos)

## 7. Conclusions

Many of the technologies we use today, such as mobile telephony, internet, GPS etc. were created for the needs of the army and later popularised for civilian purpose. Also, the first computer was invented and built to speed up the calculation of ballistic missile routes (ballistic curves). Although, cannot be considered that concrete construction has been invented for military purposes, it is safe to say that two world wars have significantly contributed to its development and improvement. Today's reinforced concrete and its existing construction products are not much older than 100 years. However, many buildings have not survived from the early years of use of reinforced concrete, and those that have been preserved are often in poor condition. The condition of the objects not damaged by explosions in Mamerki and fragments of constructions in Wolf's Lair show, however, the advancement of concrete technology and appropriate construction solutions as long as 80 years ago. The materials and solutions used in the construction of military fortifications were certainly helpful, and should continue to be so, in understanding and properly forming concrete structures – especially massive structures.

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## ПРИНОСЪТ НА ПРЕДПАЗНИТЕ МРЕЖИ ЗА СКЕЛЕТА ПРИ МЕРОПРИЯТИЯТА ЗА ОПАЗВАНЕ НА КУЛТУРНОТО НАСЛЕДСТВО

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**Abstract:** Степента на развитие на обществата се определя чрез отношението им към опазването на културно-историческото наследство. Процедурите на архитектурни и технически решения са в направление на консервация, реставрация на фасади и други дейности за възстановяване на първоначалния вид на паметника. Изпълнението на тези процеси в строителството се подsigуряват от дейности, при които в редица случаи се извършва заселяване на обектите. Продължителните възстановителни дейности, особено в градове, които са основни носители на културното и историческо наследство, са нежелан източник на замърсяване, шум и нарушаване на естествения им архитектурен облик. Тези негативни ефекти може да се минимизират, като приложената замрежваща защита върху скелетата се изпълни от тип покривала / мрежи с принтирани фасади. Съвременното ниво на отпечатване на образите върху защитни покрития е с висока степен на прецизност и качество, като това много успешно превръща строителните площи в височина в част от екстериора на града.

**Keywords:** културно-историческото наследство; ремонт; предпазни мрежи за скелета;

### 1. Трудов травматизъм в сектор строителство – челни позиции, спрямо други производствени сектори на икономиката

В международен план статистиката по сектори сочи, че строителството е водещо при всички нива на трудов травматизъм спрямо останалите сектори на индустрията. Тази тенденция се илюстрира с редица примери като:

✓ По данни на бюро по трудова статистика (BLS) в САЩ строителните дейности са на второ място по брой смъртни травми на работното място от всички индустриални сектори през 2011 година. В допълнение Администрацията по безопасност и здраве при работа (OSHA) оповестява, че всеки пети загинал работник е от строителството [5]. В същия източник е

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посочено, че 20,3% от смъртните случаи на работници през 2013 г. са възникнали в строителството, като четирите водещи причини за смъртните случаи на строителни работници са падане от височина, удар от падащ елемент, токови удари и затискане.

Администрацията по безопасност и здраве при работа посочва, че 37% от смъртните случаи при строителни произшествия са свързани с падания (от височини, падания по стълби, падане на скеле и такива в следствие на контакт с оборудване или строителна механизация).

✓ В австралийската стратегия за здравословни и безопасни условия на труд за периода 2012–2022 г. строителната индустрия е залегнала като водещ приоритет, поради големия брой и честота на смъртните случаи и сериозните наранявания от спецификата на работата. Анализът за период от няколко години показва, че строителството трайно е сред първите няколко индустрии с най-висок риск от реализиране на сериозни травми на работното място.

Профилът на строителните дейности показва, че между 2003 и 2013 година в Австралия 28% от загиналите са поради падане от височина. Успоредно с това най-често срещаните травми, свързани с работата по брой заети за 2015 година, показва че нараняванията в следствие на падания и удари от предмети са около 50% [6]. В законовия регламент, който определя параметрите на управлението на риска по направления в Австралия (WHS), много от строителните скелета се класифицират като високорискови съоръжения.

✓ Характеристика на строителните аварии в Турция показва аналогична зависимост на процентното участие на основните видове инциденти. В рамките на общия обем на строителните аварии, падането от височина е 54,7% от всички настъпили събития, на второ място са травми от удари от падащ предмет (13,2%) [7].

Проучванията на актуалното състояние на проблема в световен и национален план, представени в публикации по темата [3, 4], потвърждават представените примери за челните позиции на строителните дейности в тази негативна класация за държави от Европейското икономическо пространство и България.

## **2. Нормативна обосновка на принципите за безопасни условия на труд**

Оценка на риска при процедурите за дейностите, извършвани в сектор строителство, се определя с национални регламенти, наредби за приложение и ръководства за елементи на превенция [8-12].

България е хармонизирала националните си наредби с Европейските такива, които са:

✓ Директива 2001/45/ЕО на Европейския Парламент и на Съвета от 27 юни 2001 г., относно минималните изисквания за сигурност и защита на здравето при използване на работни съоръжения от работниците по време на работа;

✓ Директива на Съвета 89/656/ЕИО от 30 ноември 1989 г. относно минималните изисквания за безопасността и здравето на работниците при използването на лични предпазни средства на работното място;

✓ Директива 92/57/ЕИО за минимални изисквания за безопасност и опазване на здравето на работещите на временни и мобилни строителни площадки е транспонирана напълно с Наредба за изменение и допълнение на Наредба № 7 от 1999 г. [11]. В следствие същата е изведена в отделна Наредба № 2 за минималните изисквания за здравословни и безопасни условия на труд при извършване на строителни и монтажни работи [10].

✓ Рамковата директива 89/391/ЕИО на Съвета от 12 юни 1989 г. за въвеждане на мерки за подкрепа на подобренията за безопасност и здраве на



работещите при работа, транспонирана напълно основно със Закона за здравословните и безопасни условия на труд (ЗЗБУТ) от 1997 г. [8] и с няколко отделни текста, намиращи се в подзаконови нормативни актове. Това е и Законната рамка за хармонизиране на социалните въпроси в Европейската икономическа зона и за въвеждането на мерки за стимулиране подобряването на безопасността и здравето на работниците при работа.

Съгласно член 6 от директива 89/391/ЕИО, работодателят е длъжен да оценява рисковете за работниците от подхлъзване и падане от височина, и да предприема необходимите мерки за избягване и намаляване на тези рискове на основата на общите принципи за превенция.

Оценка на риска и процедурите по прилагането на наредбите са обезпечени със ЗАКОН за здравословни и безопасни условия на труд, [ЗЗБУТ, Обнародван в ДВ, бр. 124 от 23.12.1997]. Оценяването на риска като процедурни правила е разписано в НАРЕДБА № 5 [9] и обхваща работните процеси, работното оборудване, помещенията, работните места, организацията на труда, използваните продукти, други странични фактори, пораждащи риск.

### **3. Основни функции на строителните скелета при изпълнението на строителни и ремонтно-възстановителни дейности**

Строителното скеле е преместваемо съоръжение с конструкция, отнасяща се към групата на тези с временно предназначение. Фасадните скелета са временни конструкции, които спадат към групата на работните скелета. С тях се обезпечават безопасността на изпълнението на строително-монтажните и ремонтно-възстановителни дейности „на ниво по височина“ [1, 2]. Основната им задача е да осигурят фронт във височина за работниците, оборудването и материалите при строителство, монтаж, експлоатация, ремонт и разрушаване (вкл. при строителни процеси като почистване, реставрация, саниране, реконструкция и други) на сгради и съоръжения.

По същество това са строителни площадки развити във вертикалната работна равнина на сградите и съоръженията. Респективно при тях са в сила изискванията, с които на национално ниво се обезпечават безопасните условия на труд [13]. Те са основа на решенията, с които може да се намалят и/или премахнат предпоставките за възникване на инциденти, застрашаващи живота и здравето на персонала на обекта. Отнесено към строителните дейности във височина при временните съоръжения за строителство това налага засилване на контрола върху оборудването им, обезпечавашо безопасността и комфорта при извършване на процесите. Стандартните продуктови гами на елементите на странична защита включват предпазни бордови дъски и парапети - при всички скелета клас А - без покривало [14]. Тези типови обезопасителни пакети са насочени към превенция на трудов травматизъм, дължащ се на падане на работещите от височина. За предотвратяване на инциденти от падащи строителни материали, оборудване и елементи при работа върху фасадни скелета, добра практика е затварянето на работния фронт с механична защита - прилагане на системни решения от клас В [14]. Предпазната повърхнина се реализира с растерни покривала, квалифицирани според степента на пропускливост на въздушни течения като покривала с висока плътност (с минимално процентно запълване) и като мрежи (или платнище) с процентна степен на пропускливост [1]. Най-често прилаганите покриващи материали са мрежи от полиетиленови монофиламенти или моноконци, тъкан полиетилен и перфорирани повърхности от поливинилхлорид.

#### 4. Урбанистични практики при строителните обекти

В 21 век пространствата, в които се извършва строителство и ремонт, изискват планиране на градската среда с интегриране на елементите на строителната площадка. Това включва и нов модернистичен подход при **скеле системите**, особено в случаите, когато времето за изпълнение на строително монтажните или ремонтни дейности е с голяма продължителност, както е при значителни инвестиционни проекти [15] и такива, които касаят реставриране на паметници на световното културно наследство.

Фасадните скелета са пространства, които чрез елементите си за защита на работещите, **предполагат** сериозно развитие в екстериорното си вписване на населените места. Това се дължи на възможностите, които предоставят съвременните способи за отпечатване на висококачествени образи, с които се декорират и брандират предпазните мрежи. Високотехнологичните процедури на принтиране, съчетани с покритията, които гарантират дълготрайност на атмосферни влияния, намалена чувствителност на ултравиолетови лъчи и при необходимост защита от пожарни въздействия съгласно [7], предлагат продължително решение на проблема с обектите, които са предмет на дългосрочно възстановяване.

#### 5. Фасадните скелета с растерни защитни покрития - решение за историческите градове и архитектурното културно наследство

Обектите на наследството са носители на историческото и социално развитие на обществата. Предизвикателство е да бъдат запазени в условията на икономически и екологични заплахи без системна поддръжка и реставрация.



**Фиг. 1.** Бургас [16], и Париж [17], - нарушаване на параметрите на екстериора през периода на ремонт и реставриране на фасада на сграда

Консервацията на културното наследство (в частност на архитектурното такова) позволяват да се съхрани достъпът до миналото и националната идентичност, като се спазват принципите на устойчиво развитие на обществата. Паметниците на наследството са със съществен принос в урбанистичната среда на населените места, особено в градове, които са носители на историческия потенциал на съответните епохи. Респективно всяка технологична намеса, с която се цели рехабилитацията им, нарушава екстериорния облик на средата, като оказва влияние върху съседните пространства за определен период от време. В допълнение реставрационните практики изискват прецизни строителни дейности, чието изпълнение предполага значителни периоди от време, особено при паметници на

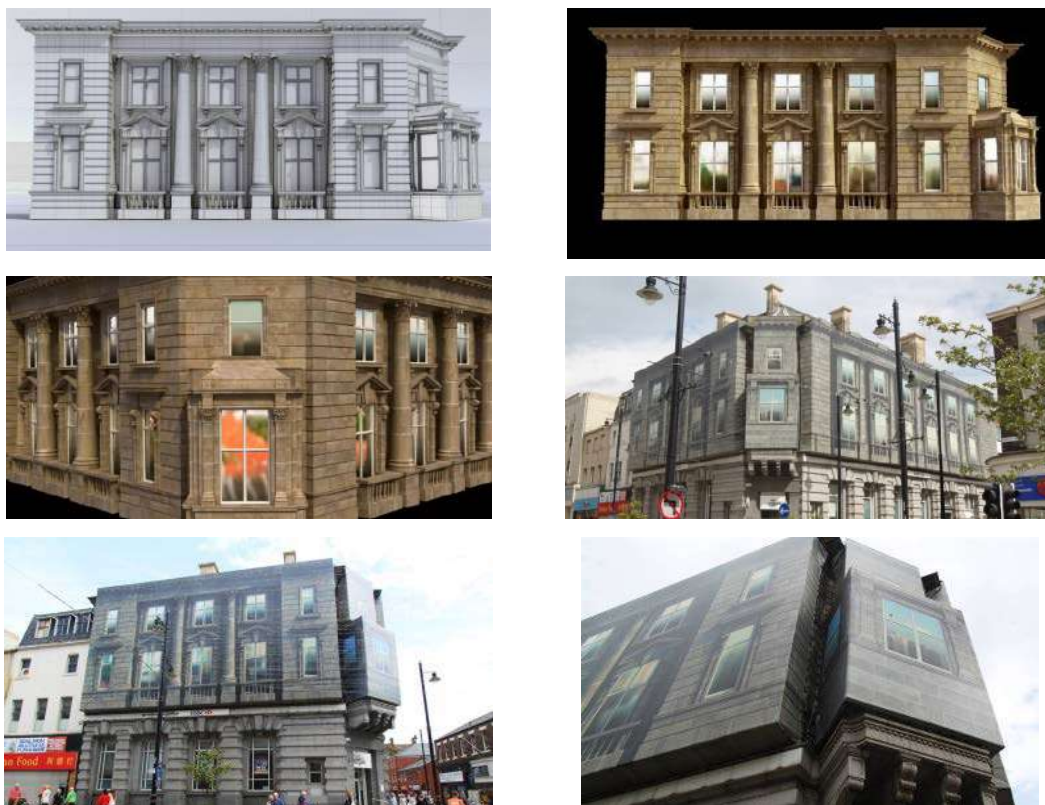
културата от типа на религиозни сгради, дворци, историческа жилищна архитектура и други.

Независимо от продължителността на възстановителните дейности при сградния фонд е недопустимо пространството в зоната на реновиране, да се превръща в строителна площадка, която нарушава историческата и архитектурна идентичност на съответното населено място – фиг.1 [16]

Решаването на този проблем трябва да се търси с въвеждането на нови подходи за интегриране на консервацията на наследството, съчетани със спазването на правилата и утвърдените практики за безопасност на екипите, които извършват възстановителните работи.

Строителните площадки от фасадни **скеле системи**, развити във височина по външните и вътрешни повърхности на сградите, са основен елемент на строителните процедури при реставрирането на сгради – паметници на културата. Предпазните растерни покрития предоставят ефективна защита от трудов травматизъм, като освен основната им задача за превенция на трудов травматизъм, материалната защита на фасадните скелета изолира от прах и шум заобикалящото пространство. Именно материалната защита **чрез** временните съоръжения за строителство е ключов елемент, при който, чрез въвеждане на съвременни технологии, се осигуряват множество вариантни възможности за вписване на строителния обект в урбанистичния стил на заобикалящата го среда за целия етап на ремонтно-възстановителния процес.

Идейни решения трябва да се търсят в мащабните възможности на съвременния пълноцветен печат с растерна графика, скоростта на принтиране и обхвата на пространствата, които могат да бъдат покрити [18] (фиг.2).



**Фиг. 2.** Проектиране и широкоформатно пълноцветно отпечатване на изображението върху PVC мрежа от 320gsm на фирма Imagegroup.  
Сграда на HSBC Bank – Sunderland [18]





Фиг. 3. Проект с векторна графика на изображението на фасадата на Salisbury Guildhall [19]

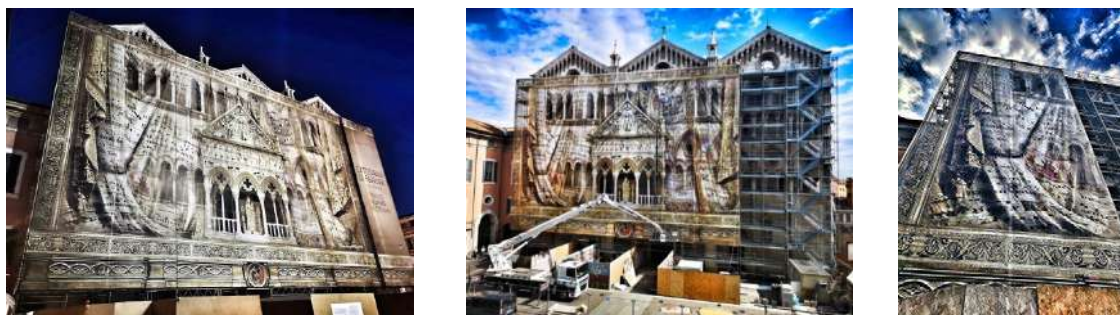
Съществено предимство е, че фирмите производители на рекламните материали, предлагат монтаж и поддръжка на покривалата, и мрежите [20]. С този подход гарантират за доброто ниво на фиксиране на повърхността и запазване на геометричната пропорционалност и последователност на изображенията.

Прецизността и гъвкавостта за различен размер на изображението, както и дълготрайността, и устойчивостта на цветовете на печата, дават възможност широкоформатното принтиране върху предпазни мрежи да отговори с индивидуални решения, които се развиват в няколко основни посоки:

✓ Изображения с художествена стойност, най-често представящи същността на обекта чрез сюрреалистичен подход на образа, който кореспондира с архитектурното и историческото му значение за обществото – фиг. 4, 5 и 6.



Фиг. 4. Музеят Магрит в Брюксел -  
покритието е с размери 21 на 75 метра широчина [21, 22]



**Фиг. 5.** Покриващата защита на скелето на катедралата във Ферара – покритието микроперфорирана PVC с площ над 1000 кв. метра, изображението е дигитален печат с резолюция, която достига 1200 dpi при много големи формати [23]



**Фиг. 6.** Банкетната къща (последната останала сграда на двореца Уайтхол) в Лондон – покритието е по проект на Northover & Brown, като темата е „революционна сграда“ и играеща с илюзия на Trompe L'oeil, ремонтът 2016 и фасадата 2019 г. [24]

✓ Изображения, с които се възпроизвежда **оригиналният** вид на фасадата, която е обект на реставрация. Най-често използвани решения при исторически сгради със сложна орнаментика като дворци, резиденции, кметства и други – фиг. 7.



**Фиг. 7.** а) Предпазна мрежа за скеле с изображение на сграда на l'Hôtel de Ville de Troyes , която е предмет на консервация [25],  
б) Принтирана фасада за ремонта на музея на Jacquemart-André [26]

✓ Пространствени художествени инсталации - широкоформатните визуализации достигат много добро ниво на естетика и пространствен реализъм [3]. Това позволява някои обекти да се затворят изцяло, което ги прави практически „упаковани“ с изображения **по** всичките си фасади. Да „облечеш“ сграда в



пространството и да композираш форма, и обем, е проектно решение, което дава възможност да се работи едновременно на повече от един фронт – фиг. 8 и 9.



Фиг. 8. Пространствена реплика на фасадата на Старата Опера във Франкфурт по време на ремонта ѝ и след него [27, 28].



a)



b)



c)

Фиг. 9. а) Оригиналната фасада на историческото кметство на Калгари [29].  
б) Начало на рехабилитацията – 2017 г. - изграждане на фасадното заскеляване [30] и  
с) „обличане“ на скелето с реплики на фасадите 2018 г. [31]

✓ **Фасада - „пр. реплика“ - фасада** – решение при случаите на частично заскеляване. Това дава възможност в периода на възстановителните дейности сградата да запази функционалността си за посещения, културни мероприятия и други [32, 33, 34]. В тези случаи трябва фото-репликата и монтажът ѝ в много голяма степен да осигурят **оптическа съвместимост** на двата образа от покрита и непокритата фасада на сградата, като същевременно границите между двете трябва да е плавна и ненатрапчива в пространството, особено през деня – фиг. 10 и 11.



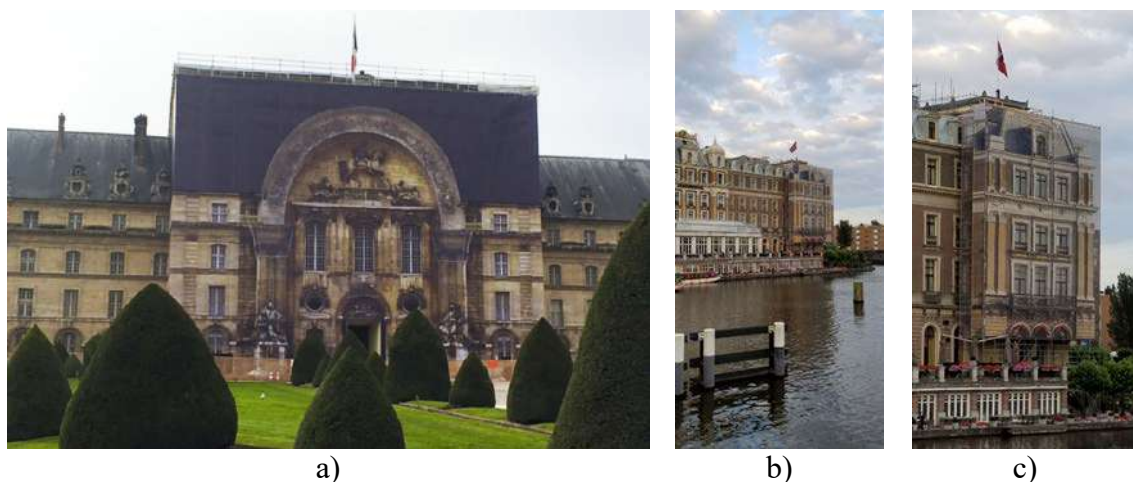
a)



b)

Фиг. 10. а) Somerset House в Лондон – частично е пресъздадена неокласическата му фасада [32]; б) Обновяване на павилиона Дюфур (лявото крило на фасадата на Версайския дворец). Покритието на скелето е частично и така сградата остава в активна експлоатация. Графиката е финансирана от компанията Dior [34]





**Фиг. 11.** а) Реплика с площ  $1400 \text{ m}^2$  за частично заскелена фасада, отпечатана върху микроперфориран винил по поръчка на фирма Layher [33]  
б) сграда в Амстердам – обектът е в частична експлоатация и в) копие на сградата върху покритието [35]

#### 4. Заключение

През 1968 година Христо Явашев и Жан Клод (Christo and Jeanne-Claude) опаковат сградата на галерията в Берн, като илюстрират форма на изкуство, при което строителните инженери стават част от творческия процес.

През 2013 година френската модна къща „Диор“ финансира отпечатване на изображение за покриване на скелето, изградено, за да се подsigури обновяването на павилиона Дарфур.

През 2020 година идеята за „обличането“ на сгради и съоръжения по време на изпълнението, ремонта или реновирането им е вариант, с който **изборът** на предпазни мрежи за скелета придобива художествена стойност, като това трансформира строителните площадки във височина в естетична компонента на урбанистичната среда на населените места през времето на реставриране на обектите.

Подобно предимство би трябвало да се препоръчва при управлението на процедурите по консервация на историческите архитектурни паметници на културата, защото така се съхранява оптическата идентичност на обектите по време на възстановяването им.

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## INVESTIGATION OF OLD STEEL RIVETED BRIDGES, PART OF THE RAILWAY NETWORK IN BULGARIA.

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**Abstract:** *According to current data provided by National railway infrastructure company, a large number of old steel bridges (331 steel bridges, most of which are riveted, with a total length of 11201.70 m and a total area of 356409 m<sup>2</sup>) are operated in the railway network in Bulgaria, which due to their serious age will have to be rehabilitated or replaced with new ones in the near future. Simultaneous replacement of a large number of old facilities with new ones is a financial challenge even for more advanced and developed countries in Europe. Bringing the old structures to the modern operating conditions in order to provide the necessary load-bearing capacity and functionality is a serious and difficult task, given the higher modern requirements. In this publication we consider two types of old steel bridge structures, which are a representative sample of the facilities on our railway network. A method for bringing them to the modern conditions for operation and at the same time increasing their bearing capacity is also proposed.*

**Key words:** *old steel bridges, riveted bridges, composite bridges*

### 1. Investigation of a real old steel riveted bridge

A bridge with a main structure riveted, full-walled, simply supported beam and an opening of 15,000 m is investigated. The facility is situated on the railway line Sofia - Karlovo - Varna per km. 22 + 730 (between the stations Yana and Stolnik) and it is built in 1929 – Figure 1. An inspection and measurements were performed and partial restoration of the structural documentation of the bridge in a volume sufficient to carry out the planned calculations, which concentrate mainly on the superstructure [6].

The existing position of the given facility marked with V0 was studied. The bearing capacity of bending in the elastic stage of the cross section in the middle was determined, taking into account the weakening of the rivets. It corresponds to class 3 in accordance with BDS EN 1993-1-1. Table 1 presents the geometric and inertial characteristics of the cross section for the various variants described below. The calculations were performed with steel characteristics defined in [6]. The steel studied there is characterized as the closest to Baustahl St 37 ( $t < 40\text{mm}$ ) with the following characteristics: tensile strength is  $f_y = 230 \div 260 \text{ N/mm}^2$ , tensile strength  $f_u = 338, 2 \div 409,3 \text{ N/mm}^2$ , ultimate elongation at break  $\varepsilon_u = 27 \div 30\%$ , modulus of elasticity determined by Least squares method statistical processing -  $E_a = 197 \div 211 \text{ GPa}$ .

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**Fig.1.** Bridge on the railway line Sofia - Karlovo at km. 22 + 730 near to Jana station

**Table 1** Geometric and inertial characteristics

| Variants    | $A_s$<br>[cm <sup>2</sup> ] | $J_s$<br>[cm <sup>4</sup> ] | $n_0$ | $n_{AL}$ | $J_{s,0}$<br>[cm <sup>4</sup> ] | $J_{s,m}$<br>[cm <sup>4</sup> ] | $W_{int,0}$<br>[cm <sup>3</sup> ] | $W_{int,m}$<br>[cm <sup>3</sup> ] |
|-------------|-----------------------------|-----------------------------|-------|----------|---------------------------------|---------------------------------|-----------------------------------|-----------------------------------|
| 1           | 2                           | 3                           | 4     | 5        | 6                               | 7                               | 8                                 | 9                                 |
| Variants V0 | 628.4                       | 2829217                     | -     | -        | -                               | -                               | 28251                             | -                                 |
| Variant V1  | 628.4                       | 2829217                     | 6.163 | 14.549   | 5164715                         | 4258231                         | 40127                             | 38281                             |
| Variant V2  | 628.4                       | 2829217                     | 6.163 | 14.549   | 5164715                         | 4173144                         | 40127                             | 37516                             |

When rehabilitating a railway in order to bring it up to modern requirements for operation, the old open road structure for a given bridge structure should be replaced with a closed one. In addition to design and operational advantages, this replacement also leads to the protection of the elements located below the level of the road structure from leaks and the realization of a corrosive environment. The construction of such a structure leads to an increase in its own weight and in combination with higher values of payloads according to modern design standards adversely affects the ratio of computational impact to load-bearing capacity for the relevant part of the structural elements. Reduction of this adverse effect can be sought by combining the new reinforced concrete road structure for joint work with elements of the old, by implementing an adequate structural connection (combining the reinforced concrete slab with elements of the old steel structure by means of dowel connection or by high-strength prestressed steel bolts ). [11]

This publication examines two options for replacing the road structure of the bridge with reinforced concrete, combined to work with the main beams: Option V1 - reinforced concrete slab for ballast passage (Fig.2a) and Option V2 - ballastless passage with reinforced concrete slab 220mm (Fig.2b with rail track on a monolithic basis RHEDA2000® [7]). The results are compared with Option V0 - current situation with "open" type of road structure and wooden sleepers, stepping directly on the main beams. For new reinforced concrete elements is accepted concrete C35 / 45 in accordance with BDS EN 1992-1-1 and BDS EN 206-1 / NA.



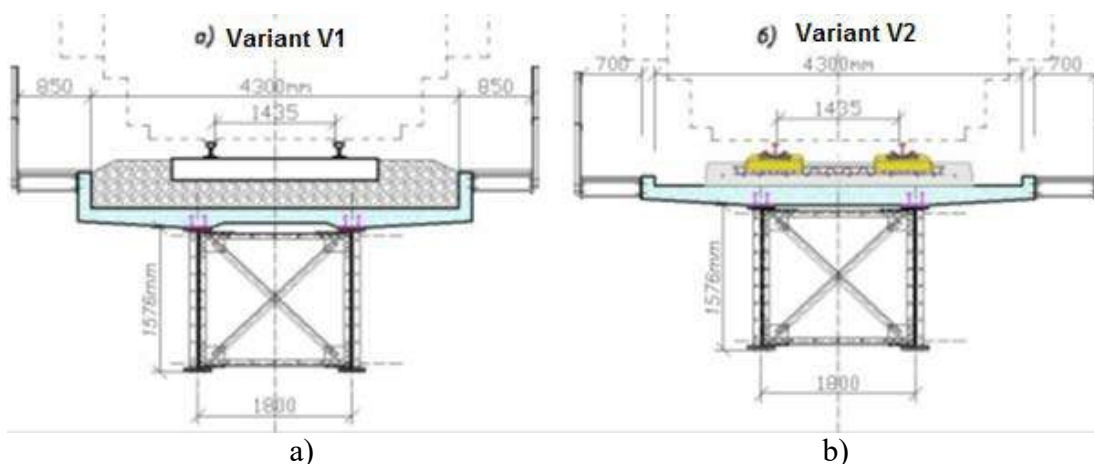


**Fig.2.** Options for construction of reinforced concrete road structure of closed type.  
a) reinforced concrete slab for ballast passage; b) ballastless passage with reinforced concrete slab 220mm – Rheda 2000 system

The relevant values of the bending moment in the field are determined by the own weight “G1” from the first stage (before the concrete has gained strength) and “G2” from the second stage when the concrete has gained strength, load model “LM71” and “SW / 2” (“LM71” is authoritative) in accordance with BDS EN 1991-2. A variant with reduced coefficients  $\alpha = 1.00$  and  $\gamma_Q = 1.30$  was also studied [5]. In Germany, according to [5], it is allowed to reduce the values of LM71 when calculating existing old bridges, but in our country such recommendations are missing. The drying of the concrete “SH” modeled with relative deformation in the reinforced concrete slab  $\varepsilon_{sh} = 0.00032$  was reported, where the temperature difference  $T_k = + 10$  Co between the reinforced concrete slab and the steel beam with coefficient of combination 0.60 - “TP” - is unfavorable. positive for the plate, “TM” - negative for the plate.

Table 2 presents the calculated values of the load own weight [kN / m'] for different parts of the superstructure: gk1 is the own weight of the steel beam; gk21 is the dead weight of the reinforced concrete load-bearing road structure; gk22 is the own weight of the railway (for V1 the rail-traverse grate, for V2 the parts of the track on a monolithic basis); gk23 is the own weight of the ballast prism for V1; gk24 is the own weight of additional parts - railing, etc. Compared to V0, variant V1 increases its own weight by about 4 times, and compared to variant V2, the increase is about 3 times. From the point of view of reducing the dynamic effects of payloads, this is favorable.

**Table 2** Net weight - calculated values

| Variants   | gk1<br>[kN/m'] | gk21<br>[kN/m'] | gk22<br>[kN/m'] | gk23<br>[kN/m'] | gk24<br>[kN/m'] | g<br>[kN/m'] |
|------------|----------------|-----------------|-----------------|-----------------|-----------------|--------------|
| 1          | 2              | 3               | 4               | 5               | 6               | 7            |
| Variant V0 | 7.933          | -               | 3.480           | -               | -               | 11.413       |
| Variant V1 | 7.933          | 13.802          | 6.240           | 19.170          | 2.110           | 49.255       |
| Variant V2 | 7.933          | 12.302          | 11.800          | -               | 2.110           | 34.145       |

**Table 3** Normal stress check for lower edge of steel tension section in the middle of the span.

| Variants   | coefficients                 | $\sigma_{g1}$<br>[N/mm <sup>2</sup> ] | $\sigma_{g2, no}$<br>[N/mm <sup>2</sup> ] | $\sigma_{g2, no}$<br>[N/mm <sup>2</sup> ] | $\sigma_{LM71}$<br>[N/mm <sup>2</sup> ] | $\sigma_{Ed}$<br>[N/mm <sup>2</sup> ] | $f_{yd}$<br>[N/mm <sup>2</sup> ] | $\eta_1 = \sigma_{Ed}/f_{yd}$ |
|------------|------------------------------|---------------------------------------|-------------------------------------------|-------------------------------------------|-----------------------------------------|---------------------------------------|----------------------------------|-------------------------------|
| 1          | 2                            | 3                                     | 4                                         | 5                                         | 6                                       | 7                                     | 8                                | 9                             |
| Variant V0 | $\alpha=1,21; \gamma_Q=1,45$ | 15.34                                 | -                                         | -                                         | 165.09                                  | 180.43                                | 228.57                           | 0.789                         |
|            | $\alpha=1,00; \gamma_Q=1,30$ | 15.34                                 | -                                         | -                                         | 122.33                                  | 137.67                                | 228.57                           | 0.602                         |
| Variant V1 | $\alpha=1,21; \gamma_Q=1,45$ | 25.85                                 | 24.13                                     | 24.93                                     | 137.88                                  | 203.28                                | 228.57                           | 0.889                         |
|            | $\alpha=1,00; \gamma_Q=1,30$ | 25.85                                 | 24.13                                     | 24.93                                     | 102.16                                  | 152.94                                | 228.57                           | 0.669                         |
| Variant V2 | $\alpha=1,21; \gamma_Q=1,45$ | 25.21                                 | 11.57                                     | 11.95                                     | 137.88                                  | 185.08                                | 228.57                           | 0.810                         |
|            | $\alpha=1,00; \gamma_Q=1,30$ | 25.21                                 | 11.57                                     | 11.95                                     | 102.16                                  | 139.32                                | 228.57                           | 0.610                         |

The results of the check for the lower edge of the steel cross section are systematized in Table 3. It is noteworthy that even at the total value of the payload (1.21; 1.45)  $\alpha = 1.21$  and  $\gamma_Q = 1.45$  the bearing capacity with respect to the tensile stresses in the lower edge is provided.

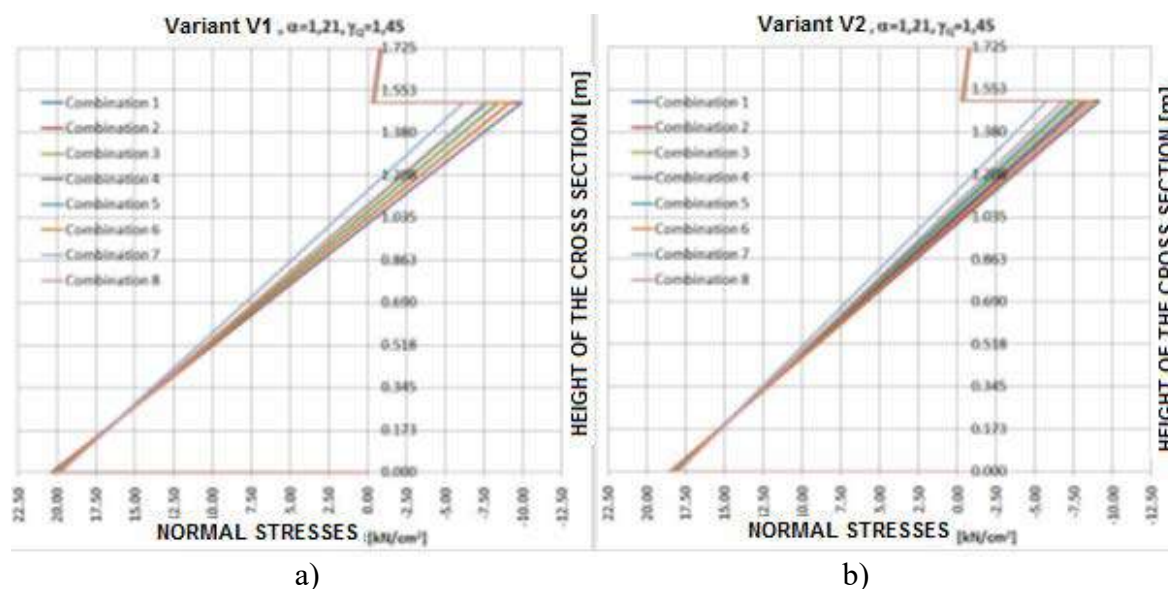


**Fig.3.** Diagrams of normal stresses for the cross section in the middle a)for Option V1; b) Option V2.

## 2. Investigation of old steel truss bridge bridge. [1]

From what has been stated in the previous chapter, it is clear that the transition from an open type of road structure to a closed type, combined to work with the main system, is an effective method for old steel riveted bridges with I beams. In order to clarify more precisely the effects of the proposed approach for bringing old steel bridge structures to modern operating conditions, an old steel riveted bridge with truss main girders was studied. The facility is part of the line Sofia - Mezdra - Varna in the interstation Kurilo - Rebrovo at km.17 + 536. The bridge was built in 1895 and reinforced with a chain link along the lower flange in 1988. (according to a project developed by Prof. Dr. Eng. Atanas Uzunov).

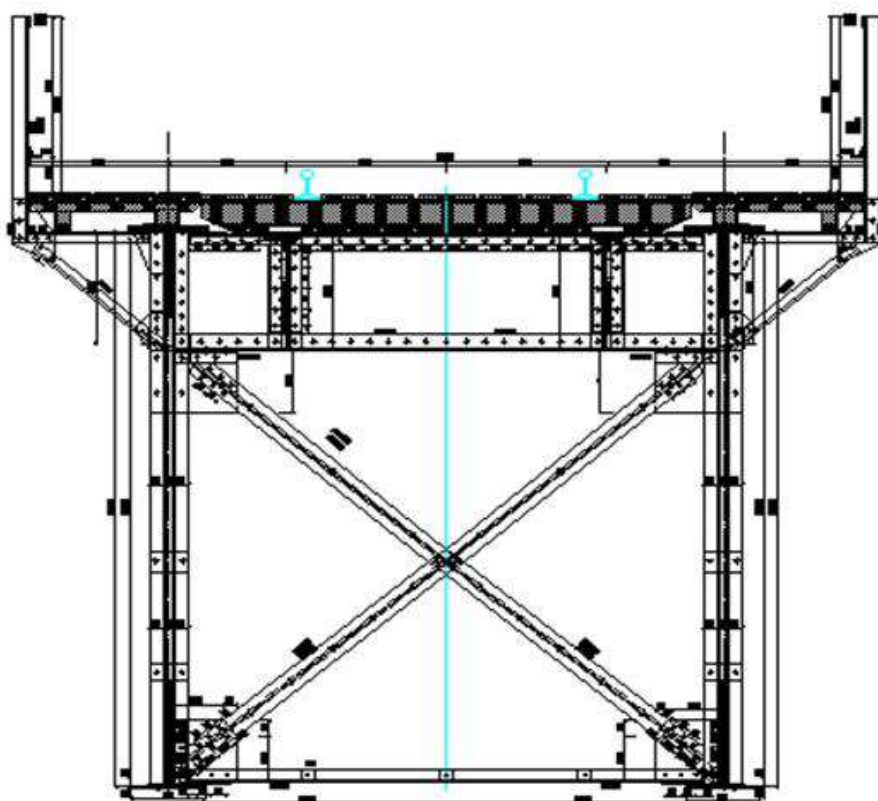


Fig.4. Variant V0 of Cross section of the bridge

### 2.1. Investigation of the existing situation - Option V0.

For the existing situation - variant V0 the bearing capacity of part of the elements of the truss is studied respectively in terms of tension and pressure (stability). Weakening of the cross sections of the respective rivet holes was observed in the elements subjected to tension. The cross sections of the elements correspond to class 3 in accordance with BNS EN 1993-1-1 [6], where the determination of the bearing capacity in the elastic stage is correct. The maximum normal stresses in the elements of the road structure (longitudinal and transverse beam) are determined, whose values significantly exceed the calculated strength of the material - Table 6. Table 4 and Table 5 present the geometrical and inertial characteristics of the cross sections for the different elements of the road and of the main structure of the bridge. Fig. 5 shows a scheme of the main beam on which the investigated elements are marked.





**Table 5** Characteristics of the studied elements of road bridge structure

The modeling of the structure was performed with the calculation software according to FEM SAP2000. Fig. 7 shows the numerical model of FEM of the existing structure. The elements of the main and road structure are modeled with beam frame-end elements. The support is in accordance with the existing bearing devices - Fig.6. Vertical loads LM71 and SW / 2 in accordance with the requirements of BNS EN 1991-2. The horizontal longitudinal inertial forces from starting and stopping of the railway vehicles, as well as the horizontal transverse loads from tortuous forces are also taken into account.



**Fig.6.** Bearing devices - movable and fixed bearing for the two adjacent spans

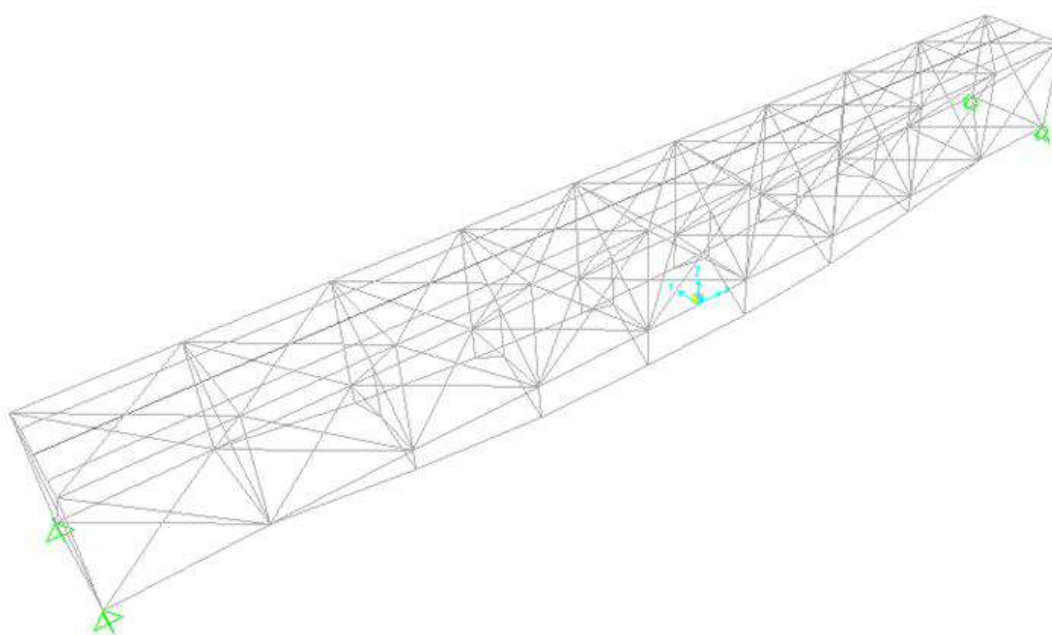


**Fig.7.** Computational 3D model of FEM of the existing facility - variant V0

For variant V0, the computational forces in the elements of the road structure and the stresses determined by (1) are determined. It is clear from Table 6 that their values significantly exceed the strength of the material - (for steel class S235). The bearing capacity of the longitudinal beam is  $\sim 4.2$  times less, and of the transverse  $\sim 2.4$  times less than the required (at  $\{\alpha = 1,21; \gamma_Q = 1,45\}$ ). With such a serious discrepancy between computational effort and load-bearing capacity, very serious reinforcement of transverse and longitudinal beams is required, which is problematic and would not be effective

enough. This is a serious argument determining the need to replace the road structure with a new one, which is the subject of this study.

**Table 6** Calculation efforts and stresses in the elements of the road bridge

| element           | $M_y$ | $M_z$ | $N$  | $\sigma_{Ed}$      | $\eta_1 = \sigma_{Ed}/f_{yd}$ |
|-------------------|-------|-------|------|--------------------|-------------------------------|
|                   | kNm   | kNm   | kN   | kN/cm <sup>2</sup> | -                             |
| longitudinal beam | 490   | 64    | 2092 | 94.22              | 4.210                         |
| cross beam        | 1052  | 13    | 222  | 53.16              | 2.375                         |

$$\sigma_{1,2} = \frac{N}{A} \pm \frac{M_y}{W_z} \pm \frac{M_z}{W_y} \quad (1)$$

$M_{y,Ed}; M_{z,Ed}; N_{Ed}$  - Calculation forces

For the existing situation - variant V0, calculations were performed to determine the calculated value of the axial force in the elements (tension / pressure) of the main truss at a set of its values. The reduction of  $\alpha$  is logical in view of the fact that this ratio takes into account the future increase in payloads within the service life of 100 years. At the same time, when repairing an old structure, the expected residual service life is relatively shorter than the standards in EN 1990 for bridges of 100 years. (usually 15-35 years) and is limited, for example, by the residual resource of fatigue for a number of critical elements. In Table 7, the computational checks for the selected elements are systematized, ignoring the small bending moments that occur due to the elastic clamping of the elements in the nodes. The computational check is reduced to satisfying the inequality between the maximum computational axial force  $N_{Ed}$  and the corresponding bearing capacity  $N_{Rd}$ :

$$\eta_1 = \frac{N_{Ed}}{N_{Rd}} \leq 1,000 \quad (2)$$

The design capacity of the elements  $N_{Rd}$  is determined as follows:

➤ bearing capacity for resistance to centric pressure::

$$N_{b,Rd} = \frac{\chi \cdot A \cdot f_y}{\gamma_{M1}} \quad (3)$$

$\chi$  – dislocation coefficient

$A$  – cross-sectional area

$f_y$  - yield strength

$\gamma_{M1}$  – private coefficient of safety in case of dislocation

➤ bearing capacity of centric tension - the smaller of the two  $N_{u,Rd}$ ,  $N_{pl,Rd}$ :

Computational bearing capacity of the gross cross section for tensile strength:

$$N_{u,Rd} = \frac{0,9 \cdot A_{net} \cdot f_u}{\gamma_{M2}} \quad (4)$$

Calculation load capacity of the net cross section with deduction of the rivet holes::

$$N_{pl,Rd} = \frac{A \cdot f_y}{\gamma_{M0}} \quad (5)$$

$A_{net}$  – net cross-sectional area

$A$  – gross cross-sectional area

$f_u$  - breaking limit, tensile strength

$\gamma_{M0}$ ,  $\gamma_{M2}$  – private safety factors



**Table 6** Calculation checks at different values of  $\alpha$  and  $\gamma$  - for variant V0

| element<br>$\alpha=1,21$<br>$\gamma=1,45$ | $N_{Rd_v}$<br>[kN] | $N_{Ed}$<br>[kN] | $\eta_1 =$<br>$N_{Ed}/N_{Rd}$ | element<br>$\alpha=1,00$<br>$\gamma=1,30$ | $N_{Rd}$<br>[kN] | $N_{Ed}$<br>[kN] | $\eta_1 =$<br>$N_{Ed}/N_{Rd}$ |
|-------------------------------------------|--------------------|------------------|-------------------------------|-------------------------------------------|------------------|------------------|-------------------------------|
| U5                                        | 5630.93            | 3446.69          | 0.612                         | U5                                        | 5630.93          | 2652.92          | 0.471                         |
| O5                                        | 4732.47            | -3279.07         | 0.693                         | O5                                        | 4732.47          | -2442.20         | 0.516                         |
| D1                                        | 4045.59            | 2931.26          | 0.725                         | D1                                        | 4045.59          | 2296.40          | 0.568                         |
| V0                                        | 2698.40            | -2146.73         | 0.796                         | V0                                        | 2698.40          | -1662.10         | 0.616                         |
| V1                                        | 1868.30            | -1546.13         | 0.828                         | V1                                        | 1868.30          | -1282.90         | 0.687                         |
| D3                                        | 1845.50            | 1537.40          | 0.833                         | D3                                        | 1845.50          | 1257.60          | 0.681                         |
| D5                                        | 995.33             | 432.60           | 0.435                         | D5                                        | 995.33           | 385.80           | 0.388                         |
| D5'                                       | 878.17             | 513.10           | 0.584                         | D5'                                       | 878.17           | 377.30           | 0.430                         |
| S                                         | 2307.01            | 1696.00          | 0.735                         | S                                         | 2307.01          | 1305.80          | 0.566                         |
| I                                         | 764.13             | -91.30           | 0.119                         | I                                         | 764.13           | -69.60           | 0.091                         |

$N_{Ed,comb}$  - изчислително усилие в елемента от комбинацията за крайно гранично състояние със съответните групи товари според БДС EN 1991-2.

## 2.2. Investigation on the proposed option - Option V1 and Option V1.

A variant in which the road structure is replaced with a reinforced concrete slab of concrete class C35 / 45 (BDS EN 1992-1-1 and BDS EN 206-1 / NA.), Combined for joint work with the upper flange of the steel riveted truss by dowel connection, providing full interaction in the sense of EN 1994 (full interaction) and rail on a monolithic basis - for example RHEDA2000® was investigated. Regarding the location of the connecting means, two variants are considered - location of the connecting means around the nodes of the truss V1 and along the entire length of the upper belt V2. A numerical model of FEM has been developed for each of the variants, reflecting the nature of the connection between the reinforced concrete slab and the steel upper flange of the main truss structure.

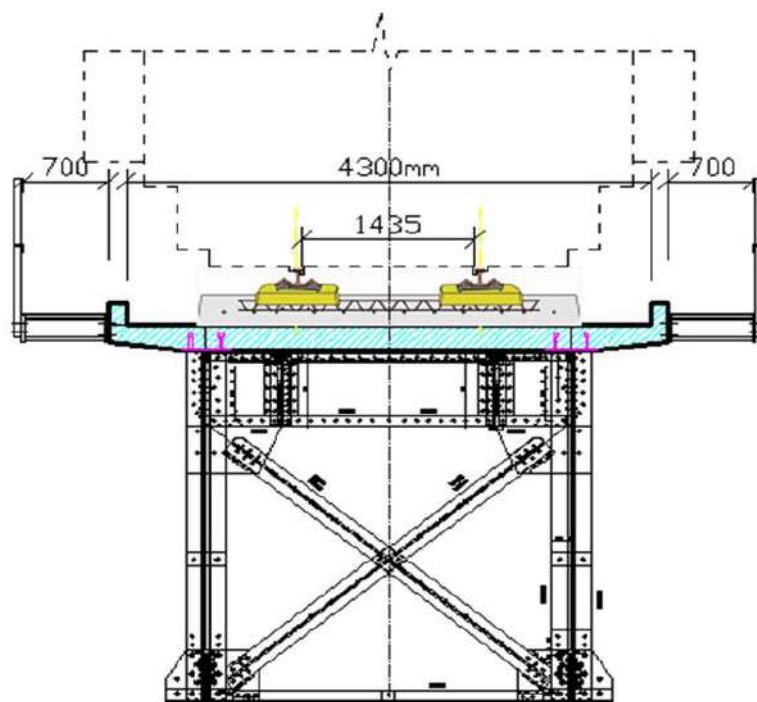


**Fig.8.** Cross section of the bridge - crossing with a ballastless railway track

In the studied variant with a reinforced concrete slab, it is modeled with shell finite elements (Fig. 9). The connection between the concrete slab and the upper flange is realized according to the location of the dowels. In variant V1, which provides for the location of the dowels to be in the area around the nodes, the flat elements are connected to the linear ones there, unlike variant V2 in which the connection is along the entire length of the elements (calculated simulated by degrees of all relevant assemblies of flange elements and the reinforced concrete slab).

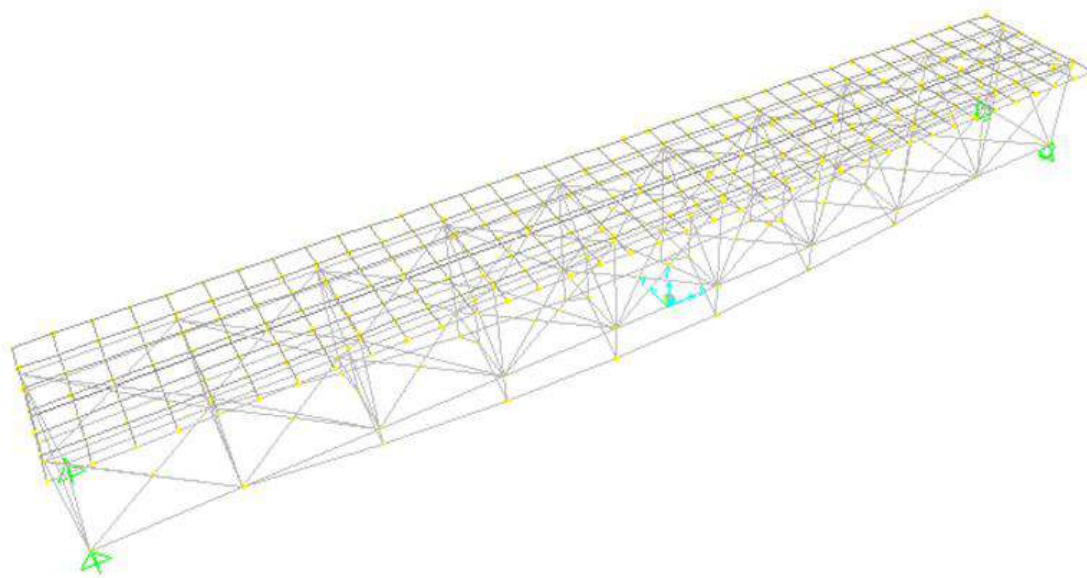


**Fig.9.** Calculation model - variant V1, V2

The rheological processes of drying and creep are reported as different modulus of elasticity  $E_{c, al}$  - for creep and  $E_{c, sh}$  - for drying, introduced by means of the translation numbers ( $n_{al}$  - for creep,  $n_{sh}$  - for dry) are introduced in the calculation model.

$$n_i = n_o [1 + \psi \cdot \beta(t, t_o)] \quad (6)$$

$$n_i = \frac{E_a}{E_{ci}} \Rightarrow E_{ci} = \frac{E_a}{n_i} \quad (6)$$

As in variant V0 and for variants V1 and V2, calculations were performed to determine the calculated value of the axial force in the elements (tension / pressure) of the main truss at a set of its values. In Table 7, Table 8, the calculation checks are systematized for the selected elements in both variants, again ignoring the small bending moments that occur due to the elastic tension of the elements in the nodes.[9],[10]

**Table 7** Normal axial forces and calculation checks at  $\alpha = 1.21$  and  $\gamma = 1.45$  - for variants V1 and V2

| Element | Variant:<br>$\alpha=1,21$<br>$\gamma=1,45$ | "LM71"+"G1"+"G2no"+"SH" | "LM1"+"G1"+"G2na"+"SH" | "LM1"+"G1"+"G2na"+"SH"+"TM15" | "LM1"+"G1"+"G2na"+"SH"+"TP15" | "LM1"+"G1"+"G2no"+"SH"+"TM15" | "LM1"+"G1"+"G2no"+"SH"+"TP15" | "LM1"+"G1"+"G2no" | "LM1"+"G1"+"G2na" | NRd<br>[kN] | $\eta_1 = N_{ed}/N_{Rd}$ |
|---------|--------------------------------------------|-------------------------|------------------------|-------------------------------|-------------------------------|-------------------------------|-------------------------------|-------------------|-------------------|-------------|--------------------------|
| O5      | V1                                         | -2025.89                | -2112.97               | -1779.97                      | -2445.97                      | -1692.89                      | -2358.89                      | -1265.19          | -1352.27          | 4732.47     | 0.517                    |
|         | V2                                         | -2011.80                | -2098.87               | -2436.82                      | -2436.82                      | -1673.85                      | -2349.75                      | -1240.40          | -1327.47          | 4732.47     | 0.515                    |
| U5      | V1                                         | 4034.36                 | 4033.85                | 4028.99                       | 4028.99                       | 4039.22                       | 4029.50                       | 4045.70           | 4045.19           | 5630.93     | 0.718                    |
|         | V2                                         | 4040.69                 | 4040.18                | 4035.14                       | 4035.14                       | 4045.73                       | 4035.65                       | 4051.79           | 4051.28           | 5630.93     | 0.720                    |
| D1      | V1                                         | 3632.09                 | 3637.35                | 3653.10                       | 3653.10                       | 3616.34                       | 3647.84                       | 3598.49           | 3608.75           | 4045.59     | 0.903                    |
|         | V2                                         | 3609.71                 | 3614.97                | 3633.98                       | 3633.98                       | 3590.70                       | 3628.71                       | 3563.11           | 3568.37           | 4045.59     | 0.898                    |
| V0      | V1                                         | -2659.45                | -2659.58               | -2653.25                      | -2653.25                      | -2665.77                      | -2653.12                      | -2672.45          | -2672.58          | 2698.40     | 0.990                    |
|         | V2                                         | -2668.57                | -2668.71               | -2662.59                      | -2662.59                      | -2674.69                      | -2662.45                      | -2671.43          | -2671.57          | 2698.40     | 0.991                    |
| V1      | V1                                         | -2014.11                | -2016.67               | -2013.70                      | -2013.70                      | -2017.08                      | -2011.14                      | -2025.91          | -2028.47          | 1868.30     | 1.086                    |
|         | V2                                         | -2004.99                | -2007.56               | -2007.32                      | -2007.32                      | -2005.22                      | -2004.76                      | -1993.29          | -1995.86          | 1868.30     | 1.075                    |
| D3      | V1                                         | 1895.65                 | 1895.92                | 1892.95                       | 1892.95                       | 1898.62                       | 1892.68                       | 1903.05           | 1903.32           | 1845.50     | 1.081                    |
|         | V2                                         | 1884.73                 | 1885.00                | 1881.45                       | 1881.45                       | 1888.27                       | 1881.18                       | 1893.63           | 1893.90           | 1845.50     | 1.026                    |
| D5      | V1                                         | 501.58                  | 498.31                 | 486.88                        | 486.88                        | 513.01                        | 490.15                        | 528.08            | 524.81            | 995.33      | 0.531                    |
|         | V2                                         | 493.08                  | 489.81                 | 477.92                        | 477.92                        | 504.97                        | 481.19                        | 522.28            | 519.01            | 995.33      | 0.525                    |
| D5'     | V1                                         | 457.04                  | 453.85                 | 442.33                        | 442.33                        | 468.56                        | 445.52                        | 482.54            | 479.35            | 878.17      | 0.549                    |
|         | V2                                         | 449.41                  | 446.22                 | 434.88                        | 434.88                        | 460.75                        | 438.07                        | 477.61            | 474.42            | 878.17      | 0.544                    |
| I       | V1                                         | -101.29                 | -101.70                | -101.70                       | -101.70                       | -101.29                       | -101.29                       | -98.29            | -98.70            | 764.13      | 0.133                    |
|         | V2                                         | -101.37                 | -101.77                | -101.77                       | -101.77                       | -101.37                       | -101.37                       | -97.87            | -98.27            | 764.13      | 0.133                    |
| S       | V1                                         | 1814.91                 | 1822.20                | 1822.20                       | 1822.20                       | 1814.91                       | 1814.91                       | 1751.91           | 1759.20           | 2307.01     | 0.790                    |
|         | V2                                         | 1853.82                 | 1861.11                | 1861.11                       | 1861.11                       | 1853.82                       | 1853.82                       | 1788.02           | 1795.31           | 2307.01     | 0.807                    |



**Table 8** Normal axial cutting forces and calculation checks at  $\alpha = 1,00$  and  $\gamma = 1,30$   
- for variants V1 and V2

| Элемент | Вариант<br>$\alpha=1,00$<br>$\gamma=1,30$ | "LM71"+"G1"+"G2no"+"SH" | "LM1"+"G1"+"G2nal"+"SH" | "LM1"+"G1"+"G2nal"+"SH"+"TM<br>15" | "LM1"+"G1"+"G2nal"+"SH"+"TP1<br>5" | "LM1"+"G1"+"G2no"+"SH"+"TM1<br>5" | "LM1"+"G1"+"G2no"+"SH"+"TP1<br>5" | "LM1"+"G1"+"G2no" | "LM1"+"G1"+"G2nal" | NRd<br>[kN] | $\eta_1 = N_{Ed}/N_{Rd}$ |
|---------|-------------------------------------------|-------------------------|-------------------------|------------------------------------|------------------------------------|-----------------------------------|-----------------------------------|-------------------|--------------------|-------------|--------------------------|
| O5      | V1                                        | -1907.34                | -1994.42                | -1661.42                           | -2327.42                           | -1574.34                          | -2240.34                          | -1146.64          | -1233.72           | 4732.47     | 0.492                    |
|         | V2                                        | -1899.67                | -1986.75                | -1648.80                           | -2324.70                           | -1561.72                          | -2237.62                          | -1128.27          | -1215.35           | 4732.47     | 0.491                    |
| U5      | V1                                        | 651.86                  | 651.34                  | 656.20                             | 646.48                             | 656.72                            | 647.00                            | 663.20            | 3252.03            | 5630.93     | 0.578                    |
|         | V2                                        | 3245.95                 | 3245.44                 | 3250.48                            | 3240.40                            | 3250.99                           | 3240.91                           | 3257.05           | 3256.54            | 5630.93     | 0.578                    |
| D1      | V1                                        | 2928.02                 | 2933.29                 | 2917.54                            | 2949.04                            | 2912.27                           | 2943.77                           | 2894.42           | 2899.69            | 4045.59     | 0.729                    |
|         | V2                                        | 2914.81                 | 2920.07                 | 2901.07                            | 2939.08                            | 2895.80                           | 2933.82                           | 2868.21           | 2873.47            | 4045.59     | 0.726                    |
| V0      | V1                                        | -2128.58                | -2128.72                | -2135.04                           | -2122.39                           | -2134.91                          | -2122.25                          | -2141.58          | -2141.72           | 2698.40     | 0.794                    |
|         | V2                                        | -2137.97                | -2138.10                | -2144.22                           | -2131.98                           | -2144.09                          | -2131.85                          | -2140.83          | -2140.96           | 2698.40     | 0.795                    |
| V1      | V1                                        | -1606.56                | -1609.12                | -1612.09                           | -1606.15                           | -1609.53                          | -1603.59                          | -1618.36          | -1620.92           | 1868.30     | 0.868                    |
|         | V2                                        | -1605.86                | -1608.42                | -1608.66                           | -1608.19                           | -1606.09                          | -1605.62                          | -1594.16          | -1596.72           | 1868.30     | 0.861                    |
| D3      | V1                                        | 1493.70                 | 1493.97                 | 1496.94                            | 1491.00                            | 1496.67                           | 1490.73                           | 1501.10           | 1501.37            | 1845.50     | 0.814                    |
|         | V2                                        | 1485.22                 | 1485.49                 | 1489.03                            | 1481.94                            | 1488.76                           | 1481.67                           | 1494.12           | 1494.39            | 1845.50     | 0.810                    |
| D5      | V1                                        | 363.05                  | 359.78                  | 371.21                             | 348.35                             | 374.48                            | 351.62                            | 389.55            | 386.28             | 995.33      | 0.391                    |
|         | V2                                        | 356.05                  | 352.78                  | 364.67                             | 340.90                             | 367.94                            | 344.16                            | 385.25            | 381.98             | 995.33      | 0.387                    |
| D5'     | V1                                        | 330.60                  | 327.42                  | 338.94                             | 315.90                             | 342.12                            | 319.08                            | 356.10            | 352.92             | 878.17      | 0.406                    |
|         | V2                                        | 324.25                  | 321.06                  | 332.40                             | 309.72                             | 335.59                            | 312.91                            | 352.45            | 349.26             | 878.17      | 0.401                    |
| I       | V1                                        | -81.98                  | -82.39                  | -82.39                             | -82.39                             | -81.98                            | -81.98                            | -78.98            | -79.39             | 764.13      | 0.108                    |
|         | V2                                        | -82.17                  | -82.58                  | -82.58                             | -82.58                             | -82.17                            | -82.17                            | -78.67            | -79.08             | 764.13      | 0.108                    |
| S       | V1                                        | 1484.74                 | 1492.03                 | 1492.03                            | 1492.03                            | 1484.74                           | 1484.74                           | 1421.74           | 1429.03            | 2307.01     | 0.647                    |
|         | V2                                        | 1514.29                 | 1521.58                 | 1521.58                            | 1521.58                            | 1514.29                           | 1514.29                           | 1448.49           | 1455.78            | 2307.01     | 0.660                    |

Given the calculations made above and the results obtained from them, the need to replace the existing road structure is imperative, as its elements are significantly overloaded. The two considered options for replacement of the road bridge structure differ in the location of the mechanical connecting dowels, uniting the concrete slab and the main beam. In variant V1 they are concentrated in the nodes of the truss bridge, while in variant V2 they are located along the entire length of the upper flange. In the case of a truss construction, the transfer of axial forces between the elements is realized in the nodes and therefore the differences in the bearing capacities of the elements (estimated by the ratio  $\eta_1 = N_{Ed} / N_{Rd} \leq 1,0$ ) at V1 and V2 vary in relatively small limits - about 2%. The concentration of the dowels around the nodes of the truss is expedient especially in the cases when the groin belt is composed of riveted plates and rolled profiles.

In the present situation V0, after the reinforcing with a additional truss at the lower flange in 1988, the bearing capacity of the investigated elements is satisfied with  $\max \eta_1 = 0,833 \{ \alpha = 1,21; \gamma_Q = 1,45 \}$  and  $\max \eta_1 = 0,687 \{ \alpha = 1,00; \gamma_Q = 1,30 \}$ . For V1 and V2, despite an increase in dead load of 2.6 times,  $\max \eta_1$  decreases by 26% (it should be noted that the private load factor for dead load in Eurocode is 1.35 and can be reduced

for an old structure if done precisely measured) and in case  $\{\alpha = 1,21; \gamma_Q = 1,45\}$  it will be necessary to strengthen part of the elements of the main structure, while in this case  $\{\alpha = 1,00; \gamma_Q = 1,30\}$  the bearing capacity of all elements of the main truss is satisfied. [9],[10]

**Table 9** max  $\eta_1$  for truss beam

| Variant | max $\eta_1$                     |                                  | $R_g/R_{go}$ |
|---------|----------------------------------|----------------------------------|--------------|
|         | $\alpha=1.21$<br>$\gamma_Q=1.45$ | $\alpha=1.00$<br>$\gamma_Q=1.30$ |              |
| V0      | 0.833                            | 0.687                            | 1.00         |
| V1      | 1.086                            | 0.868                            | 2.61         |
| V2      | 1.075                            | 0.861                            | 2.61         |

The main difference in the replacement of the road bridge structure in accordance with V1 or V2 is that the upper flange of the main truss begins to work at non-centric pressure, but at the same time the axial compressive forces compared to variant V0 decrease by about  $\Delta = 35 \div 40\%$  (Table 10).

**Table 10** Comparison of the compressive forces in element O5 of the upper flange at  $\{\alpha = 1,21; \gamma_Q = 1,45\}$

| Element O5 при $\alpha=1.21 \gamma_Q=1.45$ |          |          |          |
|--------------------------------------------|----------|----------|----------|
| -                                          | V0       | V1       | V2       |
| $N_{Ed}$ [kN]                              | -3279.07 | -2445.97 | -2327.42 |
| $\Delta$ [%]                               | 0.0      | 34.1     | 40.9     |

### 3. Conclusion

In a number of cases (represented by the old riveted bridge discussed in this publication) due to the specifics of detailing and the increased values of the horizontal longitudinal and transverse loads standardized in Eurocode, it is established that the longitudinal and transverse beams of similar open road structures for railway bridges are with significantly insufficient bearing capacity (in this case  $3.0 \div 4.0$  times for longitudinal and  $2.0 \div 2.5$  times for transverse beams). Despite the conservatism of the standardized loads, this circumstance leads to an increased risk of unacceptable deformations or destruction of elements of the road structure when passing heavier or higher speed railway trains and all related possible adverse consequences.

The main problems with the proposed design concept for the replacement of the road structure with reinforced concrete in combined action are the increased dead load, as well as the strict requirements regarding seismic response and inertial forces when starting and stopping in accordance with modern Eurocode standards, which may lead to the need to replace the bearings and the possible need for partial reinforcement of the lower structure, as well as modifications to it in order to maintain the level in the case of a ballast bed.

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## POSSIBILITIES FOR APPLICATION OF BALLASELESS TRACK ON OLD STEEL BRIDGES

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**Abstract:** Nowadays, the modernization of railway networks around the world is on a huge scale. The ballasted railway track is still the most popular and preferred option due to its advantages and many years of experience with it. However, structural solutions with reinforced concrete slab and / or asphalt concrete layer are increasingly used, especially when it comes to the crossing of the railway over a bridge structure and / or tunnel. The ballastless track is designed in such a way that there is almost no maintenance work, as the reinforced concrete slab replaces the ballast prism and assumes its function to distribute the stresses generated by the rolling stock to the substructure. Slab track also proves to be quite suitable for application to old steel bridge facilities when modernizing a railway section. Then, instead of replacing the bridges with new equipment, a reinforced concrete slab can be made for joint work with the beams, and in the implementation of an adequate connection between the two elements to obtain a combined cross section and the slab to serve as a basis for ballastless track.

The application of this system on steel bridges is not a common practice and design solutions for its are not approved in practice. For this reason, the present publication aims to analyze the different versions of slab track systems on steel bridges. There is possibility when slab track is designed on old steel bridges, the reinforced concrete slab of track to combine to work with the steel structure and obtain a combined type of reinforced concrete structure.

**Key words:** old steel bridges, ballastless track, slab track, superstructure of railway

### 1. Introduction

In recent decades, operating speeds on railways around the world have increased significantly, which has significantly increased the requirements for the geometry of the railway and bridge facilities. This is the reason why there is such a great development of various types of ballastless track systems. Despite the good characteristics of the ballast superstructure of the railway and the long-term experience in construction and its maintenance, more and more railway administrations are implementing various ballastless superstructures on their railways. The conventional ballast superstructure of the railway needs significant maintenance due to the disruption of the geometry of the railway, due to disturbances in the structure of the ballast, which makes it unsuitable for high-speed railway lines. This is the reason why this type of superstructures is being replaced by those with more durable materials, such as concrete and asphalt concrete. Improving the quality of the railway geometry through unconventional systems such as ballastless ones plays an important role not only in the comfort of travel in passenger transport, but also even more in the safety and reliability of operation, as well as in reducing the cost of maintenance.

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This publication presents various solutions for slab track systems, designed and implemented on bridge structures in recent years, and presents a comparative analysis between the two types of railway superstructures.

## 2. Ballast and ballastless railway track [1]

The slab track provides very good structural stability of the superstructure elements and the geometry of the railway track in comparison with the conventional ballast superstructure, at the expense of the increased noise emissions. Other important advantages of the ballastless construction are: the significant reduction of the need for maintenance and the long service life. Although the ballast railway track is considered a better solution for the implementation of earthworks, due to the lower cost of implementation, on the other hand, although the road on a monolithic basis is more expensive to build, the necessary maintenance over the years is many times lower. Also, the service life of the structure is longer than that of the conventional ballast railway track.

Some studies [2] carried out over the years have shown that ballastless track system is cost-effective if the cost of construction does not exceed by more than 30% the cost of construction of a conventional ballast track. The Rheda system is the most commonly used ballastless construction in Germany and the price of this system is about 1.5 times higher than the standard ballast track. Rheda system is widely used due to many years of experience [2], [3]. Independence from the higher cost of the ballast superstructure is on average by 20 ÷ 50%, due to the minimal need for maintenance, it is offered after a solution, compared to a ballast track, in the long run. Some studies in the past have been shown by calculating the cost of maintaining tracks in the range of 100 years for tunnels and 40 years for superstructures on earth structures that are larger. For example, for the Cologne / Rhine-Main high-speed line, maintenance costs are 30% higher than pre-construction calculations. Previously, ballastless systems performed their function adequately, without unacceptable deviations, after a long time without significant maintenance compared to a conventional ballast track system [2]. Figure 1 shows the results of the research conducted for the Austrian railway network, in accordance with the ballastless railway is created more efficient in the long run.

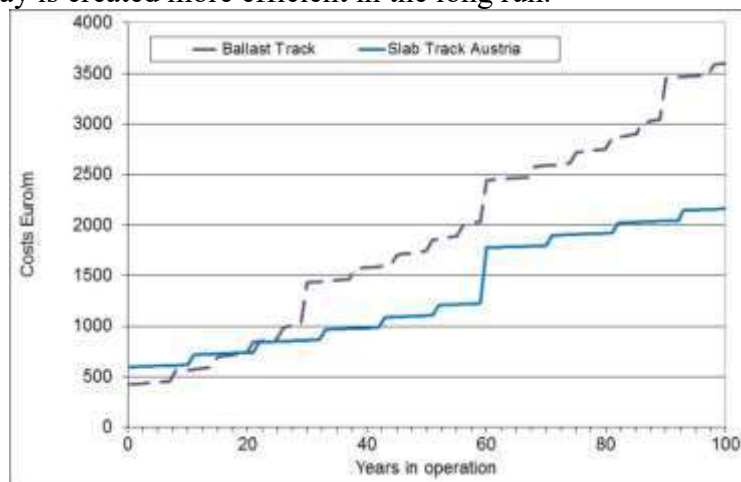


Fig.1. Comparison between ballast and ballastless railway track in time [1]

**Table 1** Differences between ballast and ballastless railway track in time

| BALLASTLES TRACK                                                                                                                                                                                                                                                                                                                                                                                                 | BALLAST TRACK                                                                                                                                                                                                                                                                                                                                                                                                                                                        |
|------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|----------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|
| <p>ADVANTAGES :</p> <ul style="list-style-type: none"> <li>- Reduction of structural height and weight</li> <li>- Reduced maintenance</li> <li>- Increased service life</li> <li>- Increased rigidity in the horizontal plane</li> </ul>                                                                                                                                                                         | <ul style="list-style-type: none"> <li>- low constructive price</li> <li>- not high performance requirements</li> <li>- low maintenance cost</li> <li>- high noise absorption</li> <li>- working together many elements of the track superstructure leads to a large voltage reduction [7]</li> </ul>                                                                                                                                                                |
| <p>DISADVANTAGES:</p> <ul style="list-style-type: none"> <li>- Require transition zone</li> <li>- more expensive to execution</li> <li>- high performance requirements for construction</li> <li>- more expensive and difficult repair works</li> <li>- highly qualified equipment and staff for execution work</li> <li>- greater noise pollution</li> <li>- require hydraulic bounded layer (300mm)</li> </ul> | <ul style="list-style-type: none"> <li>- Many elements of the railway superstructure</li> <li>- high construction height and weight</li> <li>- unstable geometry of the superstructure</li> <li>- more maintenance work</li> <li>- uneven subsidence, as well hidden subsidence of the sleepers</li> <li>- defects in weak points on the road – joints, curves with small radii</li> <li>- shorter service life compared to with ballastless track system</li> </ul> |

## 2.1. Ballastless track

The ballastless track construction is mainly characterized by the replacement of the standard ballast prism with a reinforced concrete or asphalt layer. Concrete is the widely used material for this type of construction all over the world and only in some special cases is it replaced with asphalt as the basis of the ballastless track. This is mainly due to the relatively low construction tolerances for the accuracy of execution. Various structures of ballastless track superstructures can be seen mainly in the transport facilities on high-speed railways, with a wide variety of design solutions. Each of the systems has a different bending stiffness and they must be selected depending on the stiffness of the soil layers. The whole structure is mainly composed from five layers as show in Fig.2. There are:

Subgrade, Frost Protection Layer, Hydraulically Bonded Layer, Concrete (Asphalt) Layer and the rail. [1], [8].

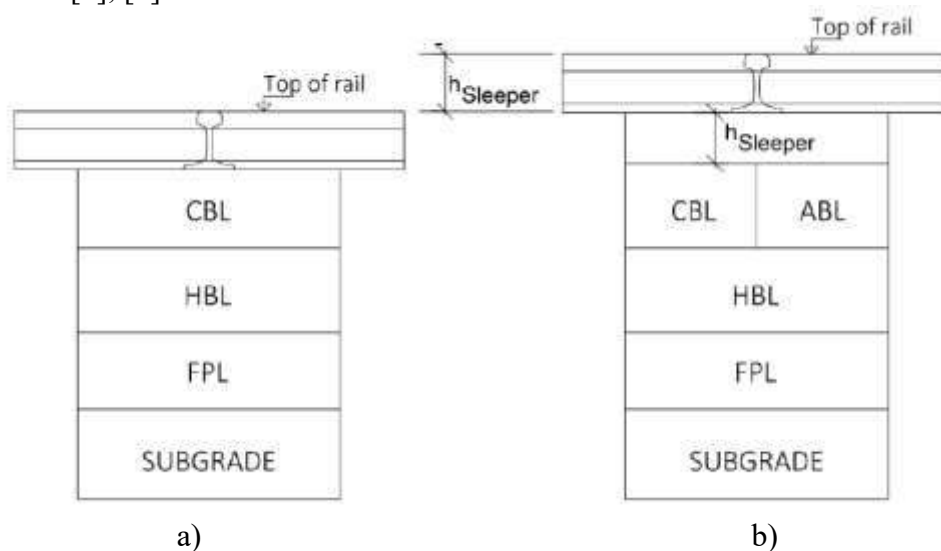


**Fig.2.** Construction for ballastless tracks: a) structure without sleepers, b) with the sleeper. [1],[4]

## 2.2. Constructive principles in the ballastless track

There are some specific requirements for this type of construction, which should be taken into account when designing and implementing it. These requirements are mainly related to: the ground base, the choice of the type of superstructure, the lower layers supporting the superstructure, the location of the site - tunnels and bridges and the requirements related to the interaction facility – railway track. Attention should also be paid to transition areas, available materials, traffic, axle load of rolling stock, noise requirements, maintenance level, construction cost, weather conditions, signaling and catenary, and travel comfort. [1], [5]. The biggest advantage of this type of systems is the durability of the superstructure over time, as the expected service life is 60 years in compliance with all requirements for the quality of the geometry of the track. In addition, these requirements must be adapted to the specific relationship between the operating speed and the base load of the rolling stock and meet the following requirements throughout the service life of the structure:

- Provided lateral rigidity of the track
- Optimal noise pollution and vibration generation
- The permanent placement of the slab track construction on the bearing concrete plate must be without deformations
- Minimum railway maintenance requirements



### 2.3. Various types of slab track systems

Regarding the support of the rails, the ballastless railway track is divided into 2 main types:

1. Railway track with discrete support (fig.3a))
2. Railway track with continuous support - Embedded rail structure (ERS) Netherlands (fig.3b))

In terms of laying, they are:

1. Cast structures on site
2. Pre-prepared in factory conditions reinforced concrete slabs, which are laid on site without wet processes (or with minimal ones).



**Fig.3.** Ballastless track: a) with discrete support б) with continuous support

With regard to the substructure they are:

1. Track on excavations and embankments
2. Track on artificial facilities - bridges, tunnels, reinforced concrete troughs, etc.

### 2.4. Ballastless track on a ground base

Depending on the construction of the substructure, ballastless systems have different features. The Rheda2000 construction (Fig. 4) is one of the most common slab track system. The basic construction consists of modified two-plate sleepers, which are securely and reliably embedded in a monolithic reinforced concrete slab. Highly elastic fastenings ensure the vertical movement of the rails, which is essential for load distribution and travel comfort.

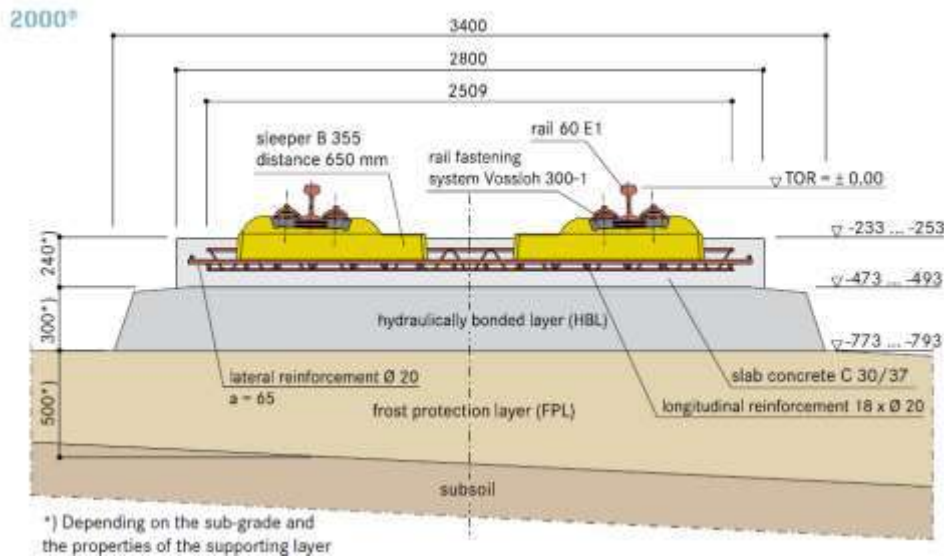
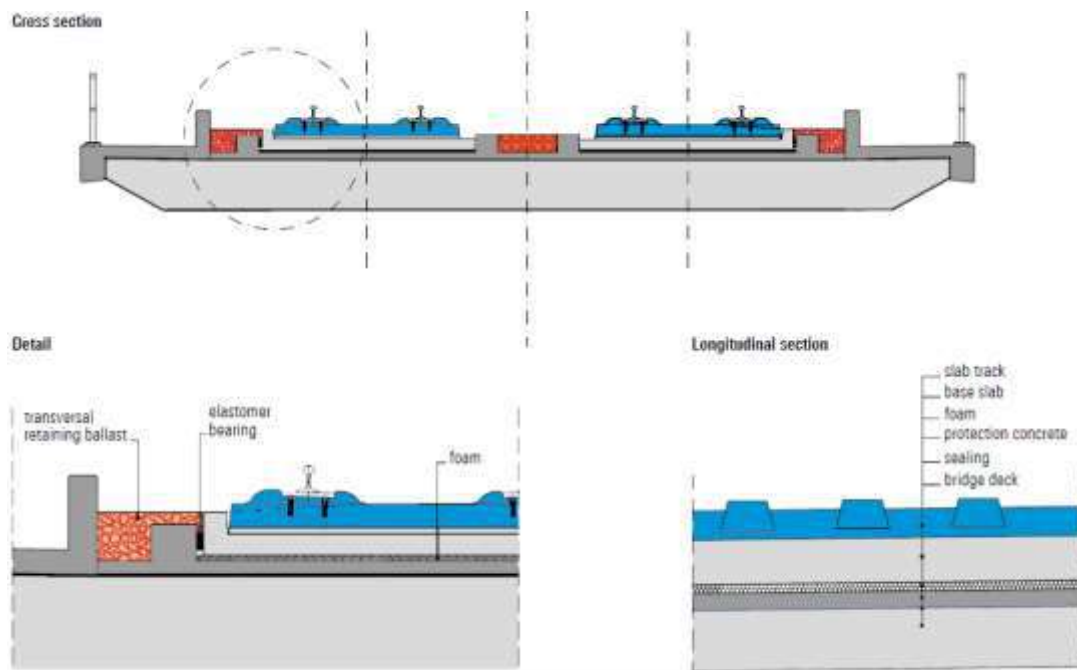


Fig.4. Rheda 2000 [1],[5]

## 2.5. Ballastless track on bridge structures

Bridge structures provide the desired rigid base without subsidence, but the generalized displacements of the bridge must be adequately taken into account in terms of their impact on the railway superstructure. Due to the temperature changes in the bridge structure, longitudinal deformations occur. The moving load causes deformations in the bridge beams and lifting is realized by turning at the ends of the beams. If the bridge is reinforced concrete or a combination of drying and creep deformations must also be taken into account. Subsequently, the railway superstructure must be adapted to the specific supporting conditions of the bridge. The rotation of the beams in the end supports must be limited to 2 %. Lifting forces from bent rails near the joints of the bridge should be checked to prevent damage to the fasteners and not to exceed the allowable stresses of the rail steel. In addition, longitudinal dilatations must be assumed without causing rail stresses higher than normalized (eg in EN1991-2). [1]

Regarding the constructive design and detailing of the ballastless railway on bridges, two main cases are distinguished - for openings up to 25 m and for openings larger than 25 m. For short bridges (up to 25 m) - types of structures without bearings are preferred - frame, in which the railway can be built continuously on the structure. The ballast-free railway is separated from the structure by a material providing low longitudinal stiffness and rigid foam, in order to prevent the transmission of longitudinal forces (Fig. 5).



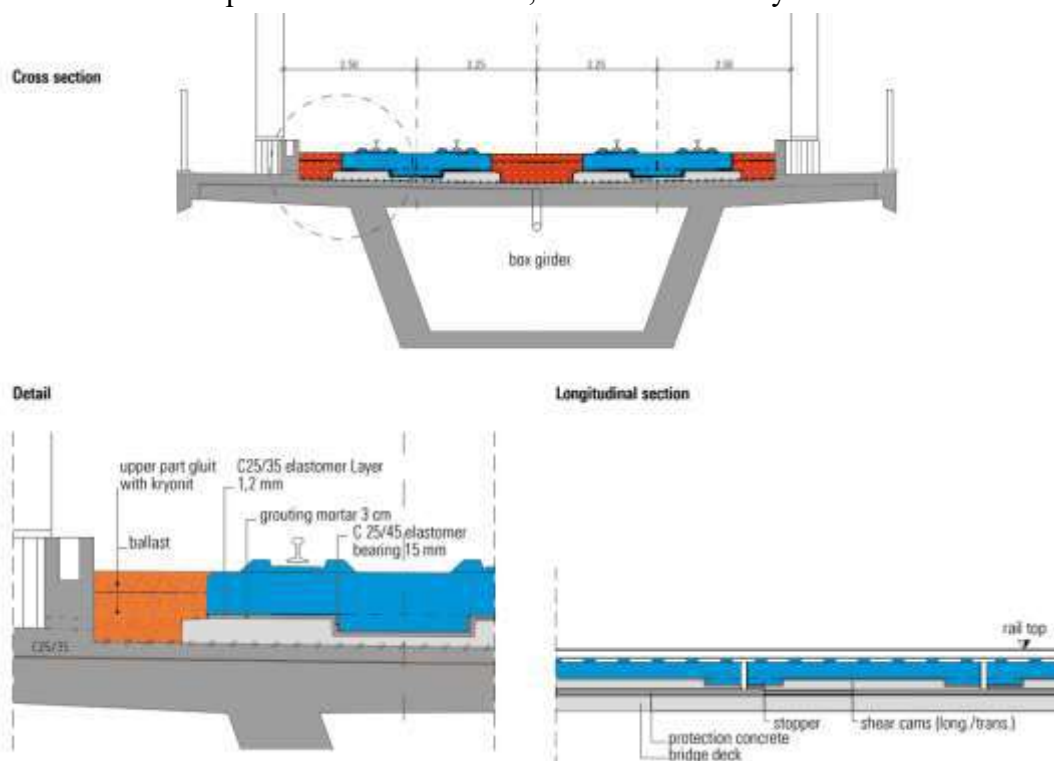
**Фиг.5.** Безбаластов железен път при мост с отвор до 25м [6]

In the case of long bridges (over 25 m), the track must be connected to the bridge structure. This is usually realized by means of retaining devices (Fig. 6). The design and details should ensure the bends in terms of temperature changes and displacements of the structure in the joints. Drying and creep of concrete must also be taken into account. In the same way as for bridges with a ballast prism, for bridges with openings exceeding a certain length, it is necessary to design expansion devices that ensure the realization of the corresponding horizontal deformation at the supports. These expansion devices depend on the length and the temperature-continuous block and on the rotation of the ends of the bridge structure and can be performed on a transitional structure to the bridge.

In the case of temperature continuously railway track on bridges, significant forces in the rails and relative displacements can develop from temperature changes, accelerating and stopping forces, as well as from drying and creeping of the concrete. The stresses caused by the interaction between the bridge structure and the railway superstructure must be limited, as they appear as additional stresses to those that occur under normal conditions (EN1991-2). In general, the problems described above that could arise in this case can be solved by implementing devices for linear expansion of the rails. For high-speed lines, this is not an optimal solution due to the fact that these devices cause a local change in the vertical stiffness of the track and the geometry of the railway, which may lead to the need for more intensive maintenance. An alternative effective solution in these cases is the use of the so-called. Zero Longitudinal Retention (LNC) fasteners for a specified track length.

This innovative method is currently being developed and introduced in some projects,

which has proven its effectiveness, but is not currently standardized.



Фиг.6. Ballastless railway track at a bridge with an opening of more than 25 m [6]

### 3. Ballastless track on old steel bridges

When applying a ballastless track on long steel bridges, three alternatives have been proposed (Fig. 7, Fig. 8 and Fig. 9) [4].

#### 3.1. Variant 1

In this variant (Fig. 7) we have a structural solution that is identical to that of a reinforced concrete bridge. The slab is combined to work with the crossbeams as a combined reinforced concrete slab. The slab track is connected to the slab by means of retaining devices. The thickness of the slab is at least 200 mm, and the reinforced concrete retaining devices (800x800 mm) are located along the slab, ensuring the stability of the ballastless track superstructure on the bridge facility.



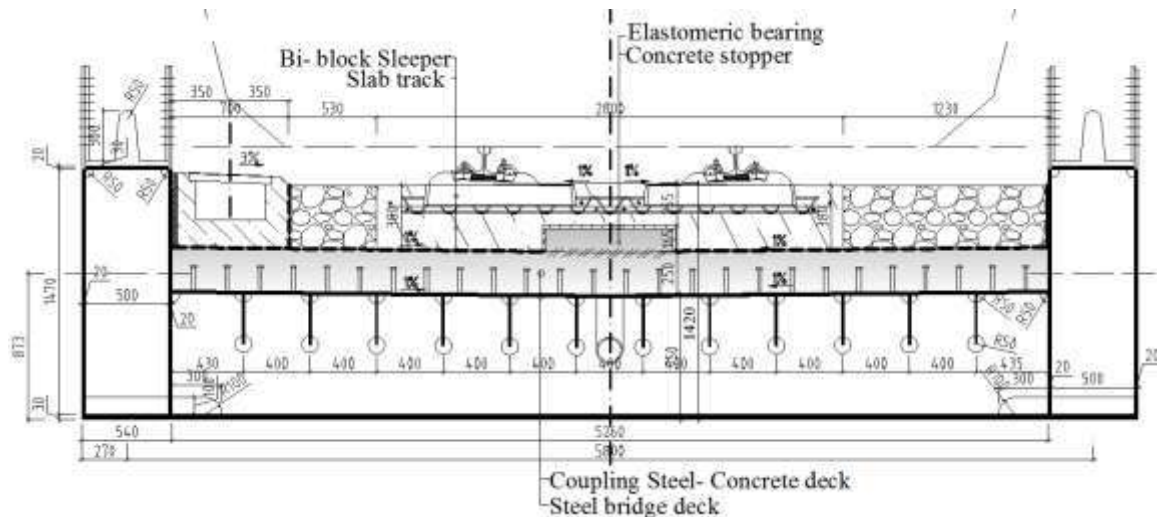


Fig.7. Variant 1 [4]

### 3.2. Variant 2

The reinforced concrete retaining devices 150 mm high are directly connected to the steel orthotropic plate, as they serve for the stable position of the ballastless system. There are elastomeric pads between the slab track superstructure and the reinforced concrete retaining devices (800x800mm).

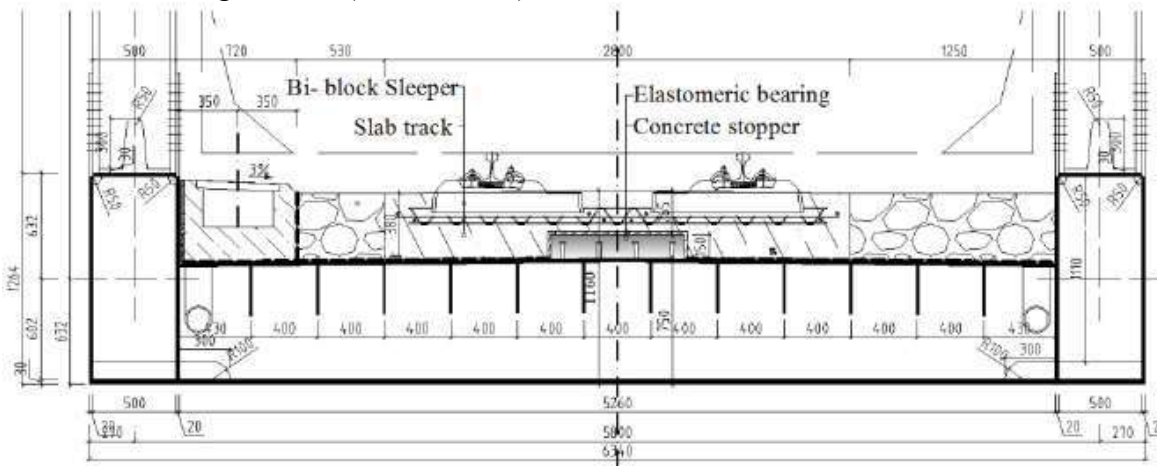


Fig.8. Variant 2 [4]

### 3.3. Variant 3

For steel bridges with smaller openings (up to 35 m), beam bridges with a combined steel-reinforced concrete slab with a road below, a road in the middle or a road above are used, as shown in Fig.9. This variant is similar to variant 1 (Fig. 7), as here the thickness of the slab is at least 200 mm, and the reinforced concrete retaining devices (800x800 mm) together with elastomer pads ensure the stability of the ballastless railway superstructure on the facility.

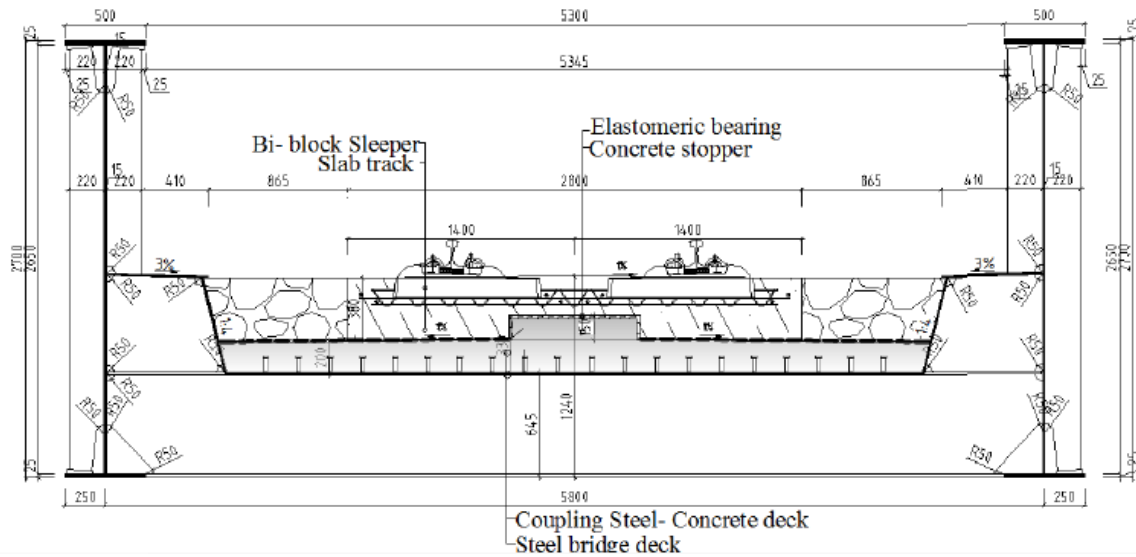


Fig.9. Variant 3[4]

#### 4. Conclusion

The application of a ballastless track on steel bridges is not a common practice and no standard solutions have been developed. In connection with the many existing steel railway bridges in the railway network not only in Bulgaria but also in Europe and possibly future modernization of the respective railway routes, it is essential to bring the steel bridges in a form satisfying modern operating conditions for high-speed railways. The purpose of this publication is to show various design options that meet modern operational requirements. Numerical models should be made, real objects and field tests should be performed in order to establish the appropriateness of the options shown above.

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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.**





## RESTAURATION METHOD WITH ELEMENTS OF RECONSTRUCTION BASED ON THE WORK OF ART - BRIDGE STRUCTURE IN OCRKAVLJE

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**Abstract:** *In order to preserve architectural heritage, some of the known methods of protection must be implemented to preserve it and protect it from further deterioration. Any reconstruction can be performed with a considerable degree of certainty if there exists an original document which provides arguments to perform physical protection through a methodological procedure. In the case of an infrastructural building, whose function remained unchanged irrespective of the passage of time, and then the authenticity is completely definite. The stone bridge structure near Ocrkavlje (Foča) has been deteriorating for a long time, probably because there have not been sufficient arguments for restoration. Restoration, replacing the missing elements, became definitive after the discovery of the watercolor depicting the condition of this bridge structure, which is an artwork of E. Loydolt, an Austro-Hungarian officer with a great talent for painting. Since the geomorphology of the terrain remained unchanged for hundreds of years, it was not difficult to detect the key rocks that served as the support of the bridge. The method of restoration with elements of reconstruction is implemented in the work. Previously, it is necessary to perform conservation and structural rehabilitation in situ in order to stabilize the existing.*

**Key words:** *Traditional stone bridge constructions, Vernacular architecture, Ottoman infrastructural architecture, Built heritage protection methods.*

### 1. Introduction

The analysis of bridge structures, which belong to infrastructural buildings, would lead us to the starting point, i.e. to the archetypal values established as early as in the Roman period [15].

However, the subject of this paper are small and medium span bridges on the Imperial Road that led from the south of Herzegovina to Constantinople during the Ottoman rule on the territory of the present-day Bosnia and Herzegovina. It is interesting to note that the route of the road was the same, and that after the long rule of Rome in this area, according to Dolabella's maps, it can be determined that the route of the road is the same, whereby the direction has been changed [2] [4] [11] [12] [6].

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The Imperial Road was a route that connected the entire territory of Bosnia, and the bridges that were built on that road were mainly built in places where certain bridge structures were built in the Roman period, which were destroyed over the centuries, damaged by the ravages of time or, which was also established, deliberately demolished to preserve the integrity of the Kingdom of Bosnia in the Middle Ages (Fig. 1) [1] [3] [7] [9].

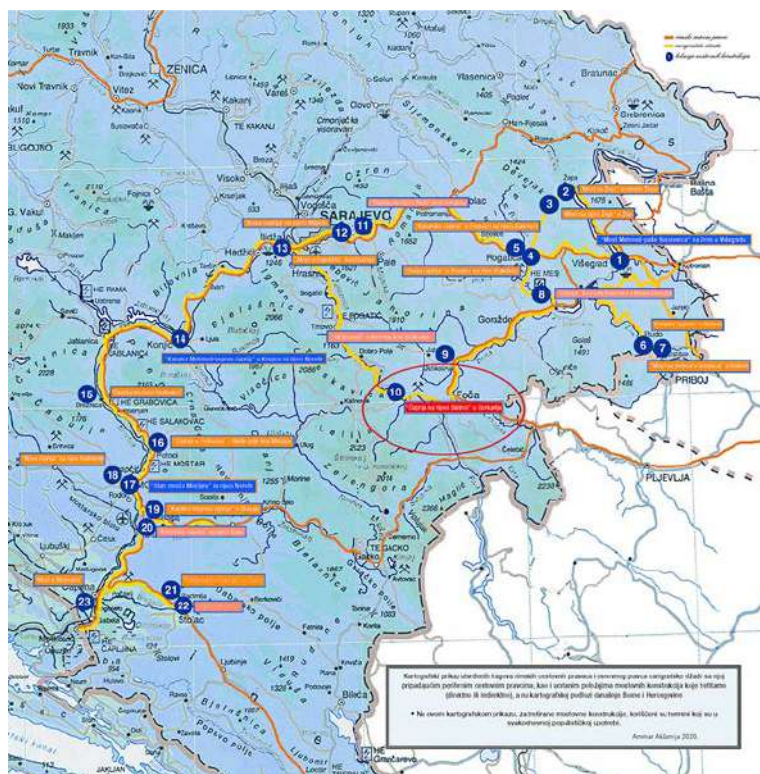


**Fig.1.** Cartographic presentation of the position of bridge structures on Roman and Ottoman road routes - the position of the bridge structure in Ocrkavlje on the present territory of Bosnia and Herzegovina is emphasized in red

We can also argue the vernacularity of these creations, because it is quite certain that they are works of architectural heritage that exist today, and their builders constructed them by transferring traditional knowledge in a belief that nothing but function should be an integral part of the final bridge structure. Precisely because of this, all these, conditionally speaking, new bridge structures, created in the Ottoman period, which we consider small and medium span bridge structures, are timeless and their archetype, as a bridge paradigm, goes far back in time. Vernacularity is the work of anonymous builders, whose intention to build structures (in our case, infrastructural buildings) was to make these structures functional, without pretending to be the bearers of character or any stylistic distinctions [4] [13].

Precisely because of that, the bridge constructions on the Imperial Road - *Constantinople džada* still bear the names such as the *Roman bridge in Ilidža* near Sarajevo (although it was built in the Ottoman period).

Experience up to date, in the field of protection of architectural heritage demonstrates a different approach and methods of possible protection of those bridge stone structures that belong to the group of cultural and historical monuments. Reasons, but not justifications, can be found in the cultural-historical significance of a certain bridge structure, its condition, the effect of unfavorable conditions that are permanently adverse to

its integrity, as well as conditions that may have historical, scientific, socio-economic character, and especially those conditioned by the contemporary life.

The aim of this paper is to show that the originality of a certain heritage, as suggested by G. De Angelis d'Ossat, can be carried out by comparison with contemporary monuments, and that in our case there is an art painting, which can be used as a valid document, because it represents an objective recording of a historical moment.

The aim is also to document and analyze, through surviving examples, that large arches which correspond in dimensions to some other arches on more representative existing bridge structures at other locations can be assumed on the infrastructural building located near the *Čengiće tower* in Ocrkavlje (Municipality of Foča), which at that time was the tallest building in the Balkans (Fig. 2.)



**Fig. 2** Bridge Construction in Ocrkavlje with the tower and mansion of Cengić family  
Existing situation - the remaining part of the upstream facade (left); Watercolor by  
Eduardo Loidolt from 1882 (right)

The research is focused on the presentation of a stone bridge structure in the immediate vicinity of the town of Foča, on the river Bistrica in Ocrkavlje. The creation of the *Čengiće tower*, which can also be seen in the painting, is related to the construction of this stone bridge structure. It was a part of the access sequence and was on the property of the Čengiće family, so that in the census from 1503 it is recorded as a *material property*. As the change of social order, but also the influence of the Western civilization brought changes in the areas of *begluks* and land ownership, and as feudal society was gradually replaced by early capitalism, this bridge structure experienced a kind of degradation and almost completely dilapidated over time. The Čengiće tower, even though not being the subject of this paper, suffered the same fate [4].

Numerous architectural theorists and conservators have dealt with this bridge structure, but, as will be seen from the results, lacking adequate documentation analyzes have mostly resulted in unsuccessful assumptions (Fig. 3.).



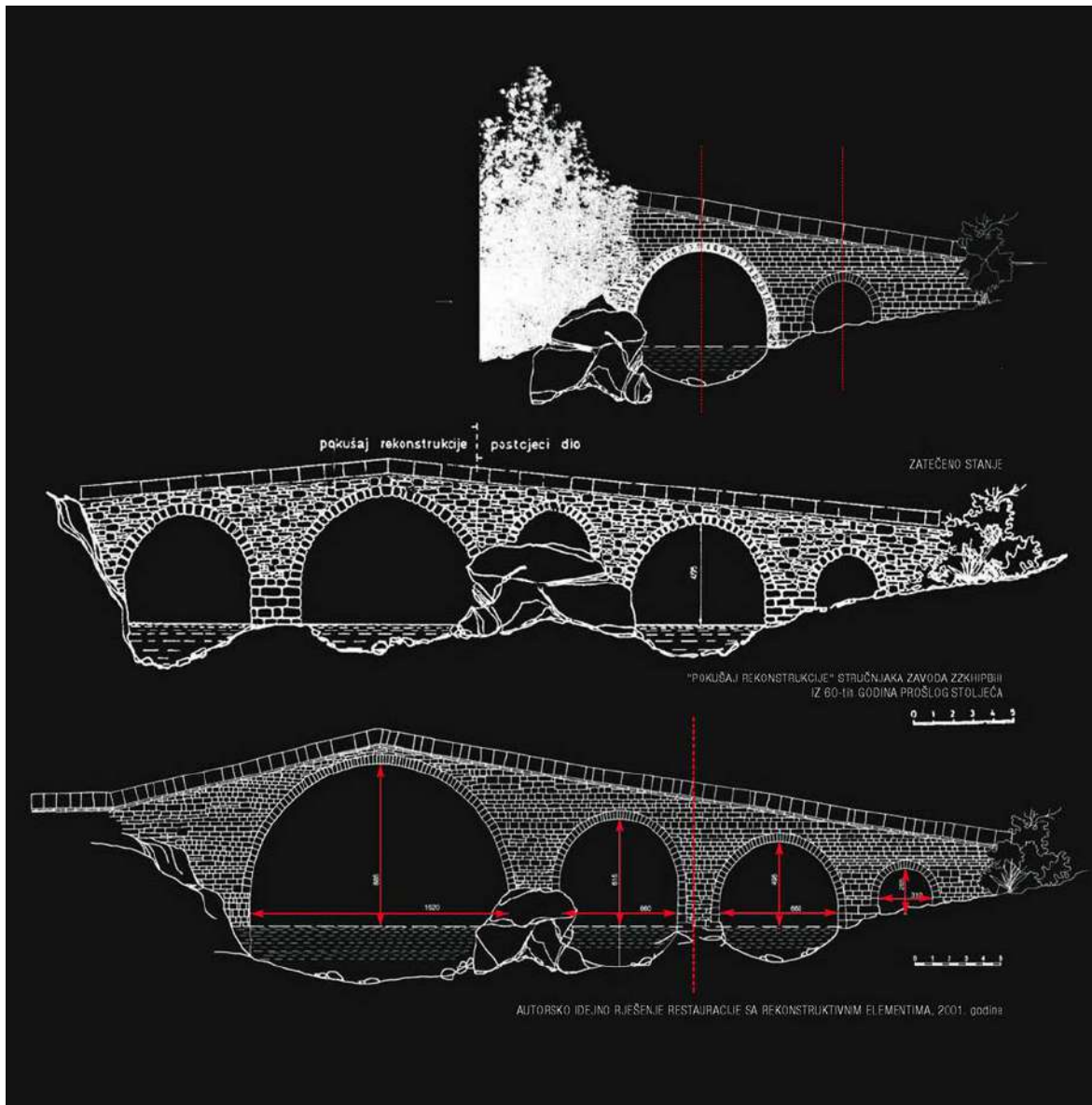


**Fig.3.** Top to bottom: Existing situation; an attempt of reconstruction by the experts of the Institute ZZKHIPBiH from the 60's. of the last century; Author's conceptual design of the restoration with reconstructive elements in 2001.

## 2. Discussion on bridge construction

Going down the left connecting branch to the Istanbul road, which led from Rogatica along the valley of the rivers Rakitnica and Prača towards Goražde and Foča, one came to Miljevina, i.e. the village of Ocrkavlje where the partially preserved remains of once grand stone bridge structure from the Ottoman period are located [1]. Based on the precisely made and preserved watercolour by Eduard Loidolt, an officer of the Austro-Hungarian army and, at a later date, an academic painter, we learn that this bridge structure had five spans [8]. From other documents we learn that during the presence of Austria-Hungary in these parts most of the structure collapsed, because due to dilapidation and lack of protection it was prone to fall, as that a wooden structure was constructed instead of the stone one [5]. Proximity and orientation of this bridge structure towards the tower of the Čengić on Rataj indicates the existence of this bridge structure at the time this extraordinary housing complex with the, probably, tallest tower in Bosnia was being built [4]. Kreševljaković recorded that this tower was built by the brothers Ahmed-pasha and



Osman-pasha, who lived at turn of 17<sup>th</sup> and 18<sup>th</sup> centuries [9]. The field inspection reveals the preserved remains of a bridge structure, comprising one large and one small arch, of uncertain age, without any prominent stylistic characteristics (Fig. 4).

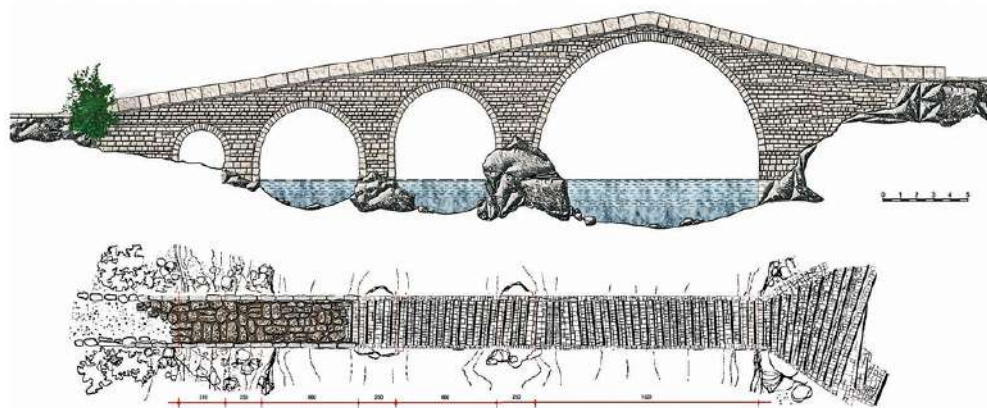


**Fig. 4.** Bridge structure of four spans on the river Bistrica in Ocrkavlje – Downstream front view and layout – Conceptual design of reconstruction; Original reconstruction with elements of structural characterization of materials and ambient, based on the Eduard Loidolt watercolour [4].

It can be observed that this bridge structure was built of somewhat more precisely dressed stone blocks of various formats, most likely from the material that could be collected on or near the site, with joints between stone blocks that are not consistently radially reduced to the center, but exhibit smaller or larger deviations. On the front walls there is a tendency of horizontal arrangement of stone blocks, but without considerable precision and consistency, where the blocks that are stacked next to the vaults are placed so as to approximately follow the arched structure and thus create the impression of the existence of some other sufficiently pronounced arch. The presence of plastics is visible on the front arches themselves, which are set back by about 5 cm in relation to the surfaces of the front walls, both on the downstream and upstream side of the bridge structure. The level of the bridge deck of the surviving part of this bridge structure constantly rises, and based on the observation of the already mentioned, preserved watercolor by Eduard Loidolt, we can state with certainty that it formed an apex above the span/vault with the largest radius. When it concerns the foundations of the bridge structure, it is evident that the builders, in order to facilitate the foundation construction, skillfully used solid large rocks in the riverbed, which is also present in the Loidolt's watercolor. The above data, as well as the knowledge that this was a very wealthy and respectable family, i.e. that this road branch did not extend beyond the tower and the mansion give us the right to assume that the brothers Ahmed Pasha and Osman Pasha in the late 17<sup>th</sup> century, in order to ensure the construction and normal communication between the tower and the mansion across the river Bistrica and the road to Foča, trade and administrative center, hired more experienced local builders for the construction of this bridge structure and road route. Also, it can be emphasized that these are well-accomplished local builders who knew how and in what way to bridge almost forty meters of riverbed, use natural rocks for foundations, while defining a specific respectable architectural design which, with a more modest stylistic approach, still relies on the Ottoman school of stone bridge construction.

The structural and compositional value of this stone bridge structure, before collapsing, was placed on three larger piers made of stone blocks, which were founded in a relatively shallow riverbed on the existing stone mass. Unfortunately, the builders did not feel the need to shape these piers in an elongated shape and place them in the direction of

the watercourse with pointed ends that would have split the water mass and reduced the pressure of the torrent, which, most likely, in addition to other oversights, led to the collapse of the bridge structure (Fig. 5).

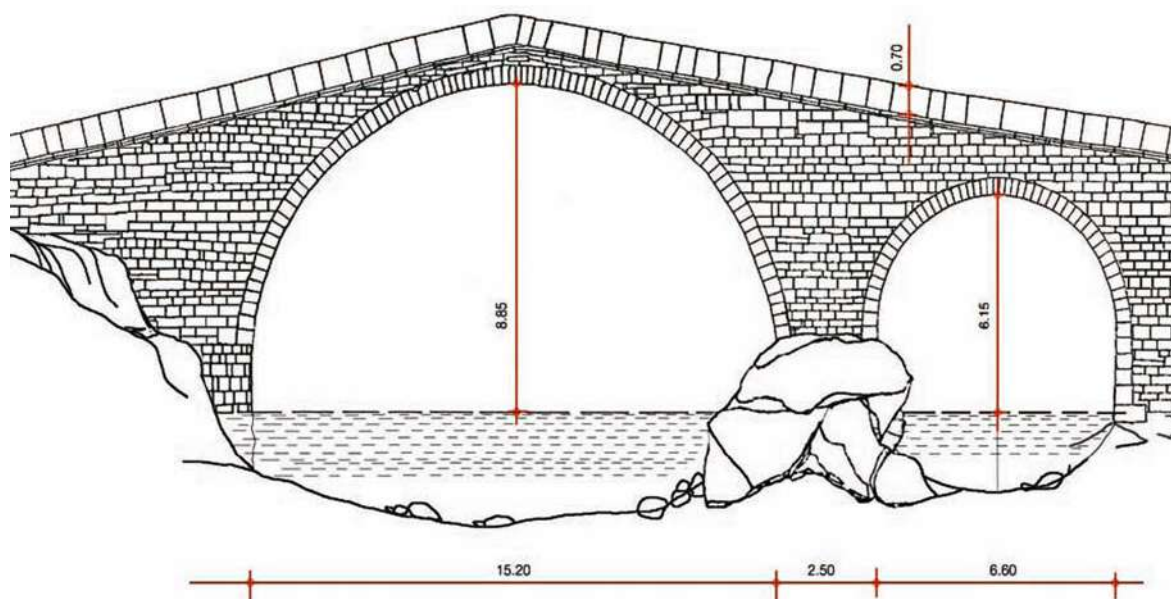


**Fig. 5.** Conceptual solution - author's reconstruction of the third and fourth arches - upstream front view [4].

Based on our research, which is basically different from the others, using the architectural restoration with reconstructive elements, we determined that the width of these piers was about 3.6 meters, and their length was about 2.5 meters [4]. Considering the total length and width of the bridge structure, this proportion of the piers guaranteed the basic stability of the structure at normal water levels, and the already mentioned lack of buttresses caused a higher vulnerability to possible torrents that were very common in this region. The piers, depending on the size of the radius and the compositional arrangement, obtained rounded pier ribs, which also, within the mentioned dependence factors, are supported by the front walls along approximately three quarters of the height of the building above the normal water table.

The transition, in places where it exists, from the vertical part of the pillar to the rounded one is accentuated by a profiled cornice. We cannot escape the impression of another deficiency, and it is a matter of securing better bridge structure supports on the right, deeper and vertical shore in the form of protective wing walls, upstream and downstream, made of stone blocks, which could prevent collapse of the bank earth mass and secure the structural value of the access ramp. Three arches of unequal width, ranging from 6.60 to 15.20 meters in span rested on the described piers. Along the left, shallower bank is the fourth arch, below which, at normal water levels, no water flowed and whose span is lower than half the smallest span across the water.

In shape, these arches are not the classic Ottoman pointed arches, but due to the relatively small eccentricity of the center of curvature; they approached the semicircular shape, which is also one of the characteristics of bridge structures in the Ottoman period. The total length of the bridge structure was about 55 meters, and consisted, starting from the left to the right bank, of the following structural dimensions: width of the embankment of about 8 meters, first span 3.10 meters, first pier 2.50 meters, second span 6.60 meters, the second pier 2.50 meters, the third span 6.60 meters, the third pier 2.50 meters, the fourth span 15.20 meters and the width of the second embankment ramp about 8 meters. The height of the bridge structure at its highest point above the normal water level above the middle of the fourth span was 10.85 meters, from where the level descended in a slight



slope towards the left bank and a more pronounced and shorter slope towards the right bank. The width of the bridge structure was 4.5 *aršins*, or 3.60 meters, of which the parapet fences - *korkaluks* occupied, most likely, two times 0.20 meters (Fig. 6).

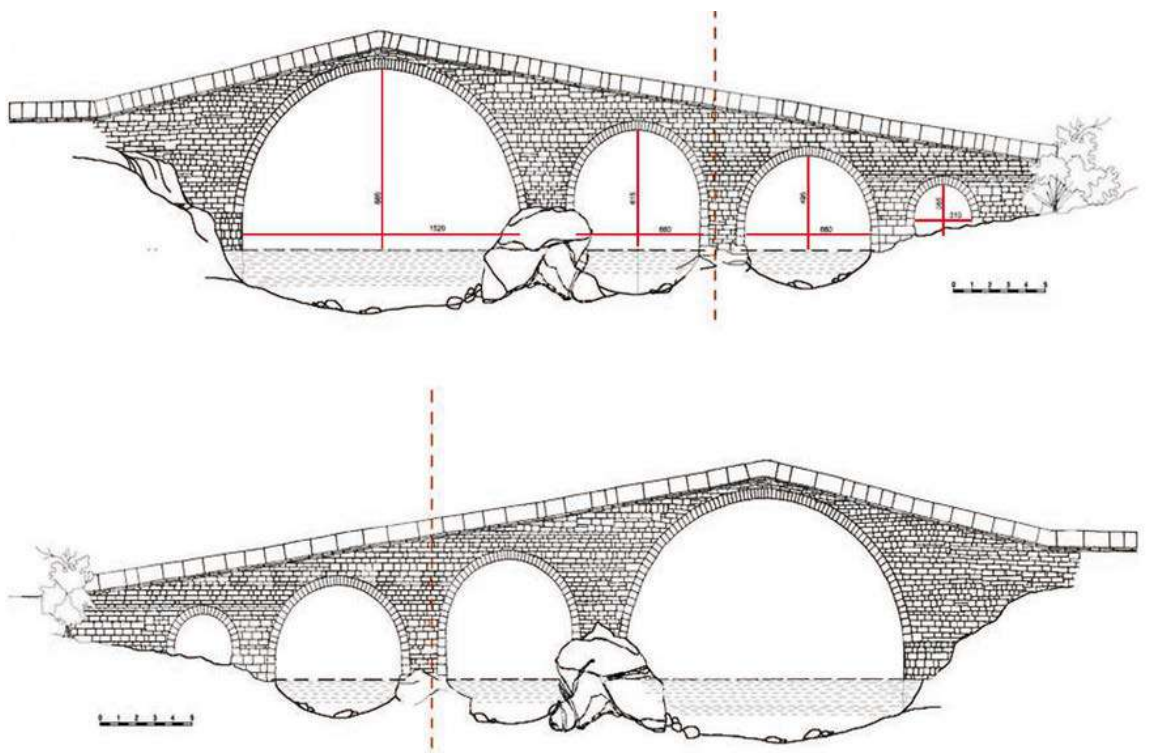


**Fig. 6.** Upstream view - Conceptual design of the reconstruction; Downstream view - Conceptual design of the reconstruction [4].

The thickness of the vaulted structures, at the largest span, was about 0.50 meters. The level of the bridge structure on the facades was outlined by a simple profile of the cornice, about 0.10 meters high, which supported parapet fence. The relatively correct arrangement of the pillars and the treatment of the wing walls on the sides towards the edges of the riverbed, i.e. towards the bank earthen hull, led to a specific design of the size and height of the openings which resulted in a specific façade appearance. The accentuated fourth span, semi circularly arched like the other spans, differs from the third and second by 8.60 meters, which is the approximate value of the height of the fourth span, and the first span is almost 50% lower than the second and third (Fig. 7).

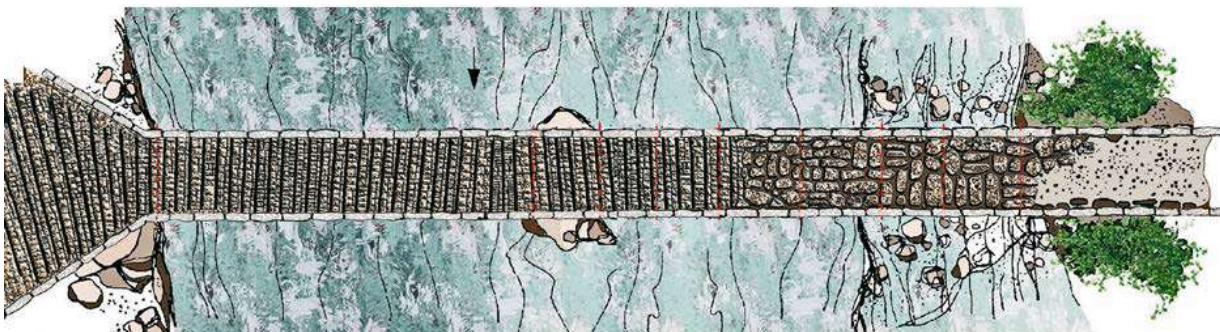


**Fig. 7.** Ground plan of stone bridge construction - author's reconstruction with elements of structural characterization of materials and surrounding [4]

with each opening's width. The listed span heights were as follows: first opening 2.65 meters, second opening 4.45 meters, third opening 6.15 meters and fourth opening 8.85 meters. Eventually, we must state the exceptional moderation in the structural proportions of mutually used structural elements on the bridge under the given conditions, which were defined by the character of location and use value, and that such a building could be realized by builders with many years of experience in construction, sound analytical and constructive approach towards the existing facilities from that and earlier periods in relation to the requirements of the client, financial conditions and available material.

The remains of the bridge structure are not proclaimed as an legal cultural monument so they are not into a register of the List of cultural monuments, only the listed reference number in the plan archives is p – 285 [14].

The proposed appearance and structure represent a completely new, reconstructed design that differs in many ways from the reconstruction design from the Plan archives of the Republic Institute for the Protection of Cultural, Historical and Natural Heritage, as well as the cited material in all available literature [14]. Namely, from historical data on this bridge structure we learn that it is a multi-arched stone bridge structure of very interestingly structured arch openings, which in the design of the Institute, in our opinion, are quite illogically arranged, and where the shape, number and position of river pillars did not reflect the actual situation *in situ*.

The method of **restoration** seemed to us the most suitable for this restoration work, where more than two thirds of the bridge structure had to be replaced. However, for the entire restoration process as an integral procedure, given the percentage of collapsed material that is much higher than preserved, it would be logical to use elements of **facsimile reconstruction**. Reconstruction is a procedure that is normally used in the protection of architectural heritage, when the monument is to be protected and restored in the event that it is completely or mostly destroyed. Unfortunately, the missing documentation on this building rules out the method of restoration precisely because the Venice Charter in Article 9 states that “restoration ends where the presumption begins ...” [16].

Reconstruction, regardless of its scientific foundation, is often adopted as a method of choice where there are disproportionately more restored than unrestored parts. **The question also arises as to how much, based on one watercolour, which evidently has a documentary value; a certain reconstruction can be considered a facsimile** (or documentary). We found a foothold and arguments in favour of our **reconstructive hypothesis** in the knowledge of the cause and consequence connection between the structural elements of the bridge structure and the relief of the riverbed on the one hand, and the remains of the original bridge structure and Loidolt's watercolor on the other. Some specific inner energy that made us think of a crossword puzzle where the unknown lies in the intertwining of known facts drove us to attempt this **reconstruction**, which obviously gave good results, by the virtue of the fact that the known parameters effortlessly matched the newly designed. By observing the cross-section of the riverbed, we come to the conclusion that the possible arrangement of river pillars is directly related to the arrangement of natural rocks in the riverbed, and that in our reconstruction the positions of river pillars and natural rocks in the riverbed completely coincided.

And finally, what absolutely confirmed our hypothesis stated in the general text about this bridge structure, that it was **a monumental building of medium span** whose largest arch we compared by a comparative method with other bridge structures of that period - obtaining a surprising result that its shape and proportion the largest arch coincides with the largest arch of the Arslanagi's bridge structure in Trebinje from the second half of the 16<sup>th</sup> century [4].

Certainly, the uniqueness of this bridge structure lies in its asymmetry, which we through our reconstruction explain by the cross section through the riverbed and the levels of access ramps on both sides. The asymmetry can be seen not only in the position of the arches and vaults, i.e. from the frontal appearance of the bridge structure, but also in the position of the carriageway that follows a series of three arches that rise slightly as the slope line rises to the middle of the fourth



largest arch when it steeply descends to the other riverbank. The assumption is that the structure of the road also differed, so on the slightly sloping side, which is at the same time twice as long, we probably have only classic cobblestones as paving, which can be concluded from the remains. The steep, shorter side, most probably featured thresholds, for slower descent or easier ascent (it is more likely than the assumption that those were stairs). This hypothesis has no material grounds, except for comparing it to the carriageways of the contemporary bridge structures (*Stari most* in Mostar and *Arslanagića most* in Trebinje) with the same or similar slope expressed in percent.

The reconstruction procedure resulted from three implemented and described methods, those being:

1. Analysis according to E. Loidolt watercolours (analytical-research method);
2. Comparison of the riverbed cross-section and position of pillars with superimposing axes (reconstructive-graphical method);
3. Comparison of the bridge structure with other similar structures (comparative method),

Which resulted in the conceptual design we have presented, and which found its realization based on the known parameters in the **facsimile reconstruction**.

We will make a small digression here and mention the installation of timber beams on buildings of monumental value. Namely, they were first cut with an electric saw and then finished with ax strokes that were supposed to remain visible, in order to create an image of manual processing. We have the same problem with stone dressing.

As this type of copying is an almost immoral act of pure hypocrisy, we are resolutely against such pretentious imitations.

### 3. Final remarks and Conclusion

#### 3.1 Results

First of the result of the paper proved that the originality of a certain heritage, as suggested by G. De Angelis d'Ossat, can be carried out by comparison with contemporary monuments, of course using the historical method, and that in our case there is an art painting by T. Loidolt. In our case there is an art painting, which can be used as a valid document, because it represents an objective recording of a historical moment.

The result is also that the situation is documented and analyzed, through surviving examples, that on the infrastructural building located in the immediate vicinity of the Čengić tower, which at that time was the tallest structure in the Balkans, we can assume large arches, corresponding by their dimensions to some other arches on existing bridge structures.

#### 3.2. Conclusion

The condition found on this stone bridge structure is nowadays only a confirmation that there was once a monumental bridge structure. An *in-situ* analysis determined the exact position of this structure, and based on its remains, the following was concluded:

- Only plastics which differentiate arches and vaults from the front wall surfaces is present on the stone bridge structure.
- There are no traces of pillars, which can be explained by the character of the river Bistrica and the strength of its current throughout the year.
- The bridge structure was asymmetrical, which is the result of architectural creativity, since the builder relied on the geomorphology of the terrain and based the supports on the rocks rising from the river.
- As a result of asymmetry, we also have a carriageway that is designed differently, so that the steeper part, which traverses above the largest arch, has a kind of thresholds (similar to the *Old*

- Parapet fence - *Korkaluk* is, judging by the remains, made of the same material as the rest of the bridge structure.

The materialization, but also the construction of this bridge structure is probably a consequence of its current degradation. Namely, as already explained, the stone was collected from the environment, but it was very poorly and insufficiently dressed, so that the tangential surfaces are uneven, and the mortar, as the basic binder in the frontal surfaces, was washed away over time. As it is usual in the analysis of degradation, especially in the case when there are existing traces, which is the situation with this bridge structure, the process of active protection requires interventions in the form of protection methods that will primarily eliminate existing problems in the current state and then follow up with interventions in order to compensate for what is missing. Therefore, the existing condition should be conserved and structurally repaired, and then the restoration procedure should be carried out, which, precisely because of the way we approach the solution of the problem, also possesses some elements of reconstruction.

Although the proposed procedure would be of great importance for the preservation of the traditional vernacular heritage, it is not planned for the time being. Unfortunately, and probably due to the lack of attractiveness, this stone bridge structure is not listed as a cultural monument, but is recorded on the list of the current Federal Institute for the Protection of Monuments, under number P - 285.

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## FACSIMILE RECONSTRUCTION BASED ON PHOTO DOCUMENTATION, USING IMAGE PROCESSING SOFTWARE PACKAGE

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**Abstract:** *Using the photo documentation from the family archives, the facsimile reconstruction of the three-arch stone bridge construction in Rogatica was created. At present, this bridge construction does not exist and therefore this way of work, as a unique though documented analysis, was a significant contribution to the retrieval of certain traditional values. Urban transformation that resulted from a significant change in the town's urban matrix should not be a problem if there is documentation that will confirm the structure and appearance, as well as the materialization of the forcefully demolished heritage. Interest in traditional values remains an obligation, despite the fact that the so-called Karanska Bridge was demolished near the end of the Second World War. It can be claimed with confidence that the facsimile reconstruction is the method of choice in the case when this kind of degradation has already occurred. Many examples from practice describe this kind of reconstruction, taking account of the significance of the reconstructed objects, the memory of the place, the spirit of the place, spirit of the age compared to the existing environment, traditional values, monumentality and the necessity of the existence of these traditional structures in the given space.*

*In this case, reasons for unaccomplished reconstruction are twofold. On the one hand, they pertain to modest original accomplishments, which were the result of local craftsmen's construction, thus of the vernacular architecture, which is instrumental rather than monumental. On the other, the stone bridge constructions in Rogatica (two in total) date back to the 16<sup>th</sup> and 17<sup>th</sup> century (thus to the Ottoman period); the end of the Second World War and their demolition were followed by the socialist period, which can presently be characterized as amnesiac in terms of everything that is traditional. Obviously, this bridge construction was not an object of interest. The socialist period was followed by the war in Bosnia, and everything that occurred in the post-socialist period can be described as the state of general anarchy and a lack of sufficient resources for reconstruction. A possible facsimile reconstruction is only a modest contribution of the present moment and our own attitude toward tradition.*

**Key words:** *Architectural heritage, Rogatica, stone bridge constructions, Karanska Bridge, vernacular architecture, traditional bridge constructions.*

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## 1. Introduction

The data at our disposal [2] [3] [4] on this three-arch stone bridge construction were very scarce, since it was demolished near the end of the Second World War, during German troops' withdrawal in 1944. It is even referred to by different names in different sources. Thus, we encounter names "Kanarska", "Karanska" and "Kamarska" Bridge, while Rogatica inhabitants also used the name "Upper Bridge" (since it is upstream of the "Lower Bridge"), which was not used in the available literature. Čelić and Mujezinović, as well as Ayverdi, describe the bridge construction and provide data on the size of arches, structure of vertical walls, form of the piers, parapet, etc. [3] [1]. Further research reveals that these authors took these data, without the citation, from a text by Alija Bejtić [2] who, in his monograph "Rogatica" cites Austro-Hungarian and Ottoman-Turkish sources and provides part of the accurate dimensions and basic characteristics of the bridge construction. Thus, Bejtić writes that this three-arch stone bridge construction had two arches of the same size, 4.20 meter wide, and one arch (left) 2.70 meter wide. The same text says that, in terms of shape, this stone bridge construction had *no style characteristics*, although it claims that the almost perfectly regular semicircular shape of the arches started from the bottom of the pier, and therefore appeared as a segment of the semicircle at higher water levels, which also resulted from the height of the bridge compared to the river level, of somewhat higher than two meters [2].

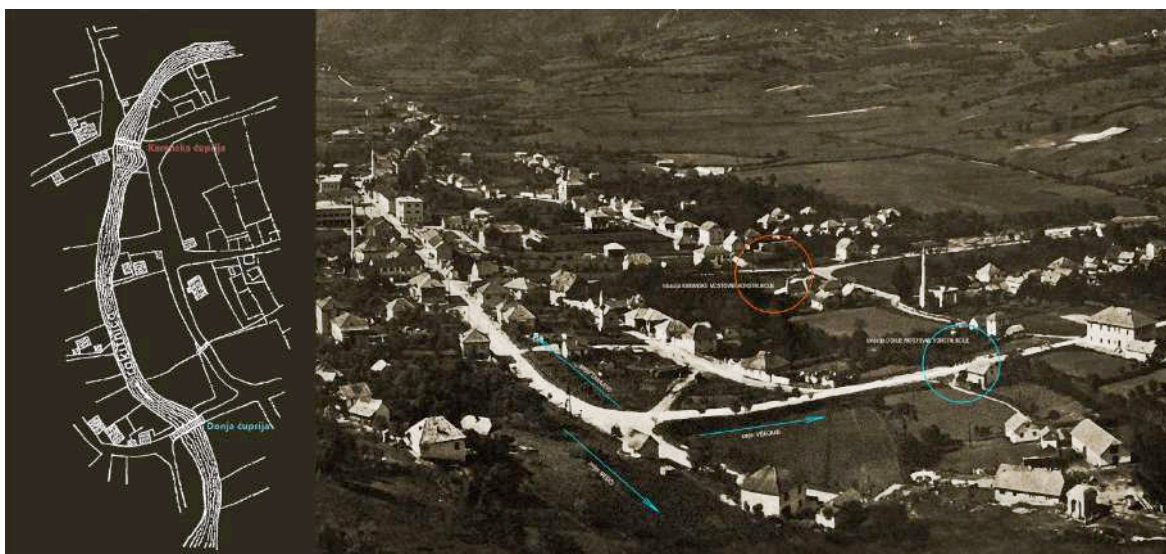


**Fig. 1.** Scheme of the position of bridge constructions on the Rakitnica river in Rogatica. The photograph from the 1950s reveals the position of the bridge constructions. *Karanska Bridge* is located at the frontier of Krušćica and Gračanica neighbourhoods, and is marked in red.

We could not find the detailed documentation on this bridge construction; however, since the mentioned text by Alija Bejtić literally says that the bridge was around 16 meter long and 3 meter wide, we felt the need to thoroughly study these data and attempt to develop a conceptual design of the reconstruction according to the existing dimensional parameters and the available photo documentation.



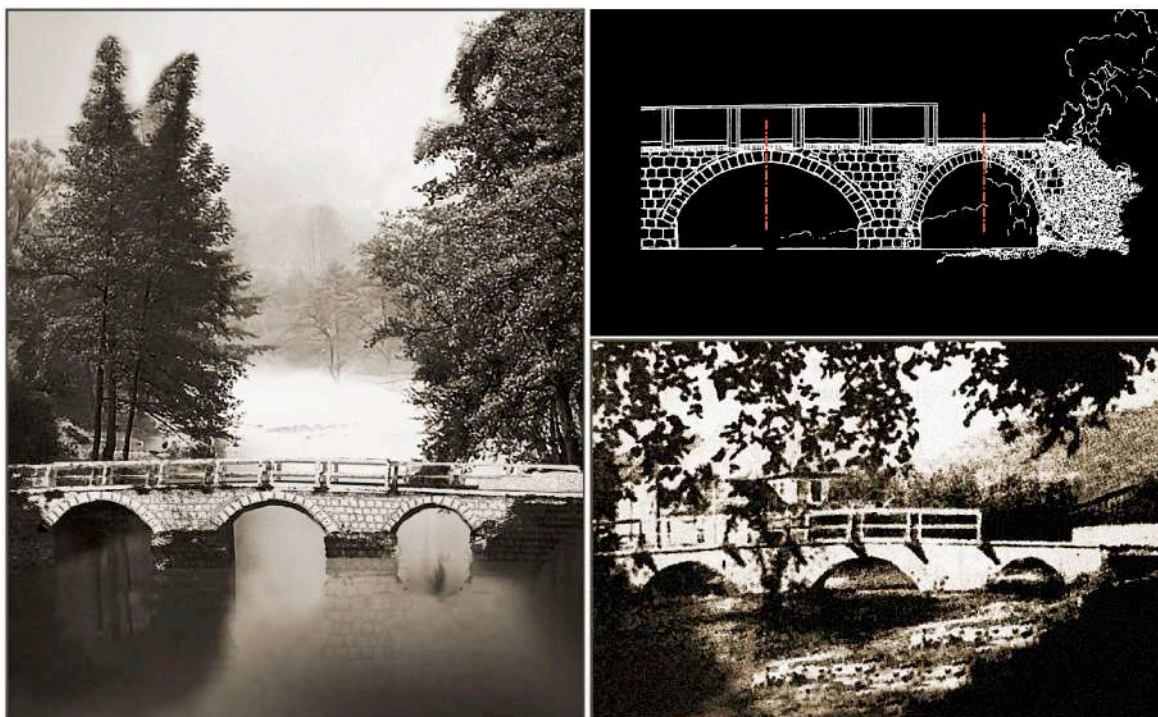


**Fig. 2.** Presentation of the part of the procedure of processing illustrating material of different origin (on the left: processed photograph and on the bottom right: processed reproduction from printed material), and a drawing of part of the conceptual design of the bridge construction (top right) developed based on the condition and measurement conducted within the process of photo-digital analytic approach.

We have collected many photographs that had never been published, photo-documentation from the press, and we also surveyed older inhabitants, who provided us with numerous interesting data. Thanks to this survey we obtained the explanation of the problem and answers to questions that we believed would remain unanswered. This example made use of, and first promoted, the modern method of photo-digital analytic approach to the already existing light drawings (photographs) and / or reproduced photographic material from some of the offset techniques, as the basic source for obtaining the sizes and parameters of the unknown construction values in the form of linear drawing (Fig. 2. and 3.) and based on some of the already known parameters. We have used such a principle of visual research aimed at obtaining proper parameters on other bridge constructions as well, during our work on conceptual designs of reconstructions when it was the necessary, or the only possible way. The newly created linear digital recording was subjected to possible corrections of perspectives and angular deformities, and proportional relations were established using the already known dimensions. In this way, we obtained all, previously unknown, values of the elements of the bridge construction and it was fully reconstructed. This method will be described in more detail in the section on protection principles.

Amazingly, using this process of assembling the known elements we managed to obtain the accurate length of the bridge construction of 16 meters and 30 centimetres, which is certainly the accurate expression for “about 16 meters” [2].

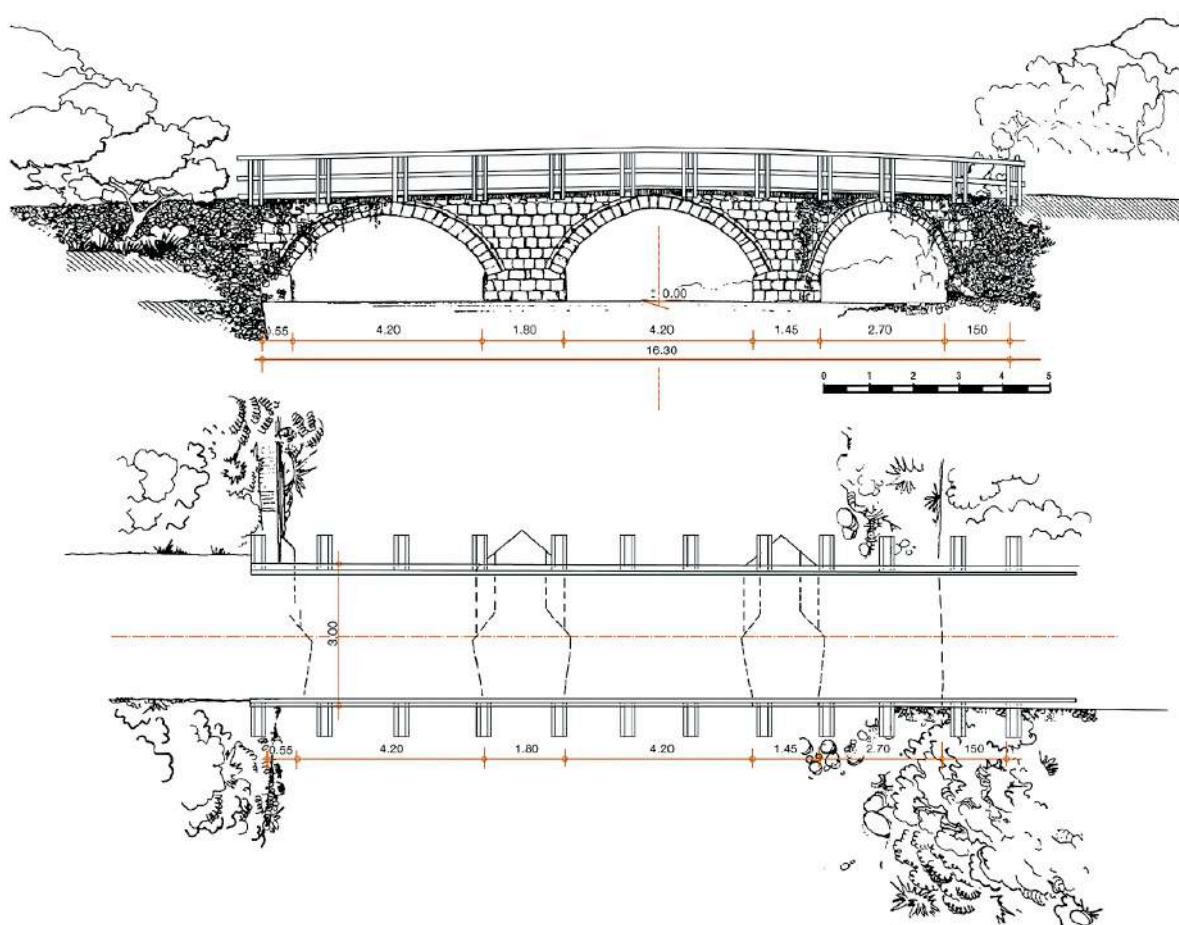


**Fig. 3.** Three-arch stone bridge construction on the Rakitnica River in Rogatica.

Upstream façade and layout – Conceptual design of reconstruction – Authorial reconstruction using the methodology of photo-digital analytic approach to linear drawing based on the used documentary material, survey findings and description by Alija Bejtić

The detailed measurement made after entering the known values resulted in the insight, as we already pointed out, into the other possible dimensions of the elements of this stone bridge construction (Fig. 3.). We discover that two piers had the following dimensions: the left one – 1.80 meters and the right one – 1.45 meters; we can also explain the different width of these piers by different dimensions of the arches starting from them. The abutments were shaped in such a way that the right one had the footing and was only 0.55 meter wide, while the left one did not have the footing and was 1.50 meter wide (Fig. 3.).

Review of the two preserved graphic reproductions from the period of the Ottoman Empire with motifs from Čelebipazar/Rogatica revealed the present slight slope of roadways from the highest point in the central part toward the periphery (land) (Fig. 4. and 5.).



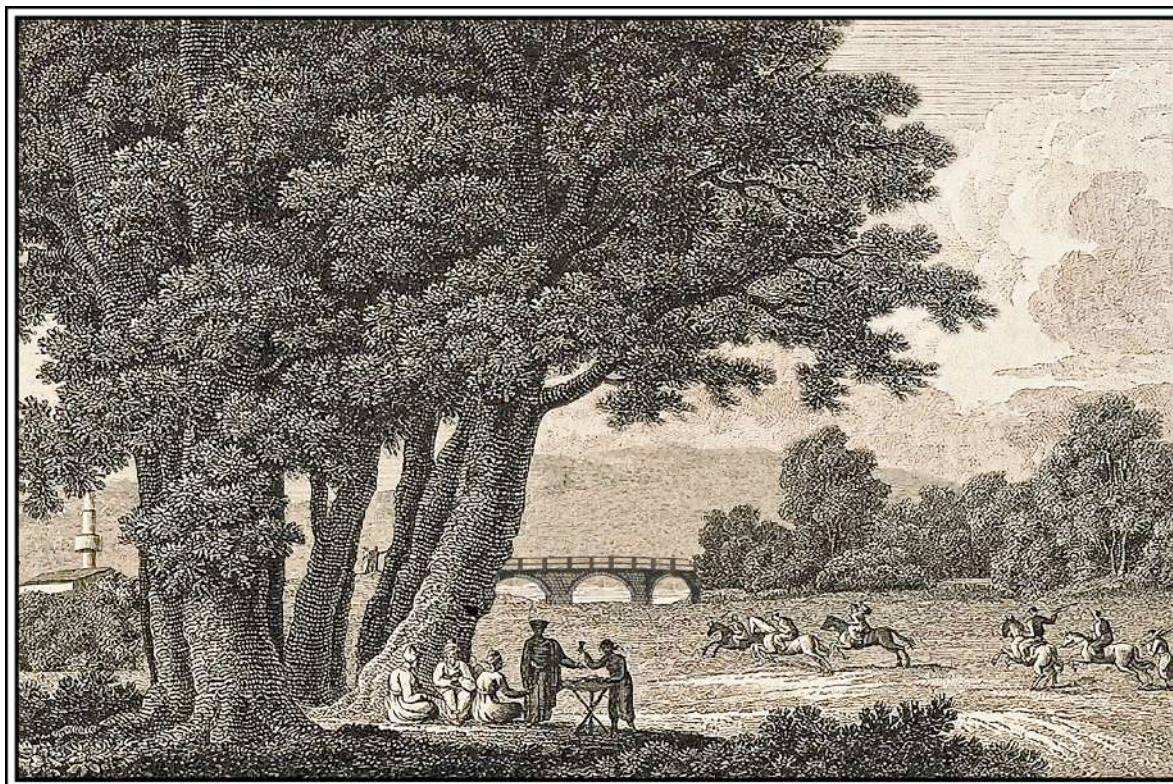


Fig. 4. Unknown author, Čelebipazar/Rogatica, copperplate technique, around 1850.

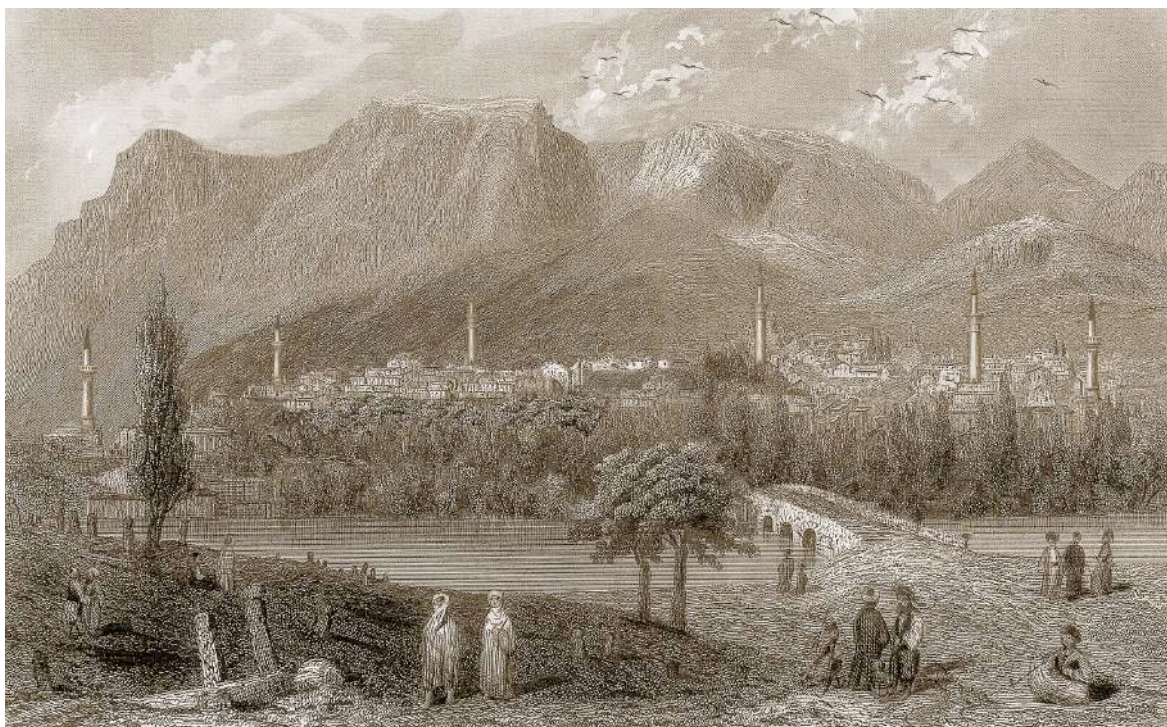


Fig. 5. Unknown author, Čelebipazar/Rogatica – view from the Gračanica neighbourhood, drypoint printing technique, around 1850.

Besides, the immediate inspection in the field established the match between the stone construction roadways, which were on a slight ascent toward the middle, and the heights of access ramps, although the flow regulation of the Rakitnica river was done on this site in the 1960s. It included the construction of underpinning and a new reinforced-concrete bridge construction, inappropriate for the landscape and the existing urban



architecture of this part of the town of Rogatica, which significantly reduced the width of the riverbed, although the heights of the banks remained the same (Fig. 6.).

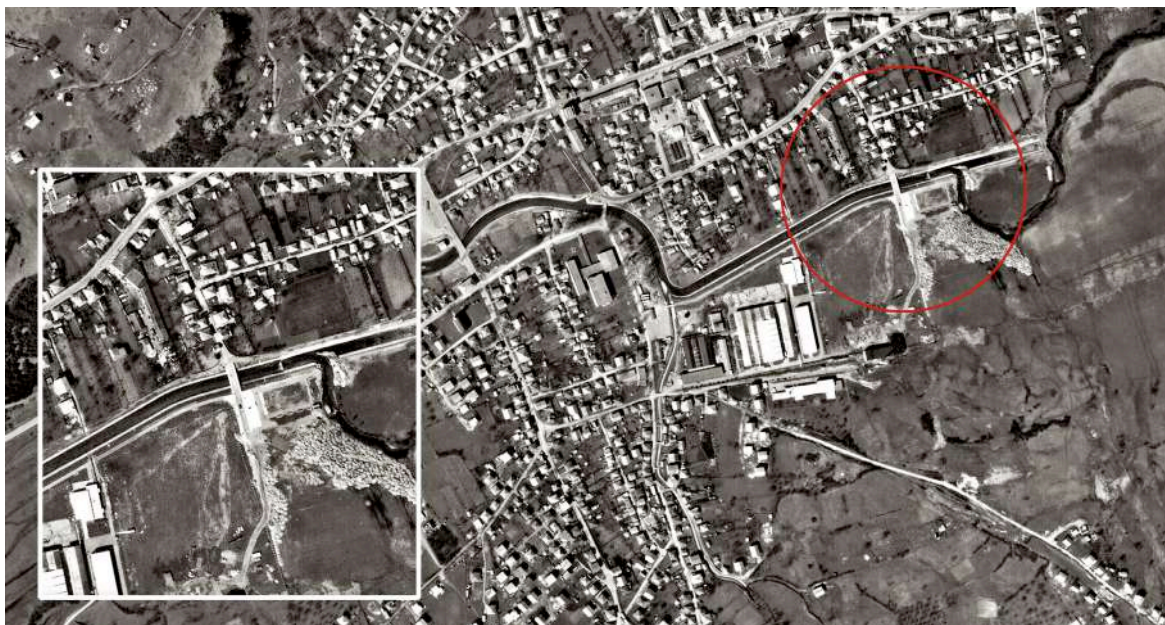


Fig. 6. Aerial photo upon the completion of works on the flow regulation of part of the Rakitnica river through central town zone, from the 1960s.

The survey confirmed our assumption based on the processed documentary photographs (Fig. 2.), that the parapet of the bridge construction was made of wood and that the string course, which followed the roadway line, had openings where wooden material was inserted and fastened to support and hold the parapet. This method was known as early as at the time of Roman bridge building and was also used in the Middle Ages [1]. Such a material origin of the parapet required occasional replacements, and these data lead us to believe that *Karanska* Bridge construction never had a *korkaluk* (parapet, presented in Fig. 5.) made of rock masses. In other words, it can be assumed that in at least one of those pictorial representations, one of the authors allowed himself the freedom of visual presentation of certain elements in space (Fig. 4. and 5.)

## 2. Comparative approach

The comparative method, with other stone bridge constructions of the same category and from the same time period discovered the data on the stone bridge construction in *Konavle near Dubrovnik across the Ljuta River*, where we also encounter a three-arch stone bridge construction without a parapet, with openings in the string course which point to the earlier existence of a protective wooden parapet that no longer exists (Fig. 7.).

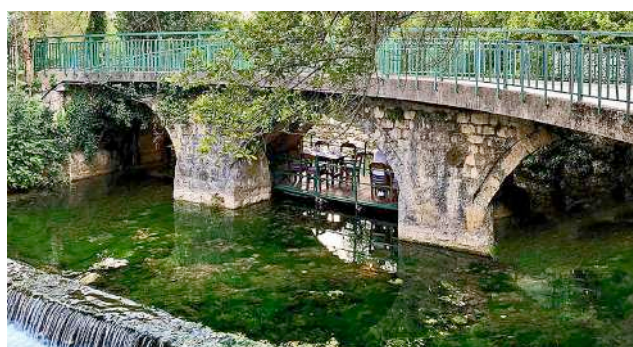


Fig. 7. Stone bridge construction without a parapet on the Ljuta River, Konavle near Dubrovnik



Since this bridge construction is located on the first part of the imperial road/*dzada* which leads from Boka Kotorska and Konavle across Popovo Polje along the Neretva River valley and further on, via Mostar all the way to Sarajevo, Rogatica, Višegrad and further to Istanbul, thus, since both stone bridge constructions are found on the same route, we deduce that this comparison would be realistic.

The following example, although of a more recent date but certainly from the same civilizational circle and with the same empirical knowledge of bridge builders, the stone bridge construction across the *Hendek* creek in front of the Travnik fortress had a wooden construction in the form of parapet. This stone bridge construction is an access ramp to the Old Town from the west side, and there are also other entrances from different sides. In 1957, some work was done on this access construction, and the wooden parapet was replaced by a stucco parapet with the extension of cement mortar (Fig. 8.)



**Fig. 8.** Stone bridge construction where the wooden construction of parapet was subsequently replaced with a walled *korkaluk* (parapet). Entrance into the Old Town in Travnik from the west side.



Besides, comparing the already described form of the arch with the arches of the stone bridge construction at *Plandište near Blažuj* we can conclude that it is almost the same construction of the arch and the vault, where the arch starts directly from the bottom of the pier (Fig. 9).

Based on these comparisons, we could not agree with authors Čelić and Mujezinović that it is a bridge construction without style characteristics of the time [3]. We would define the issue by claiming that it is evident that one cannot speak of the accurate time of construction – which in the European or, more precisely, Western coding of terms indeed refers to style affiliation. Eastern civilizations, including the Ottoman Empire as its obvious representative in our regions, do not recognize such a typology; rather, they simultaneously use several construction techniques, several equally represented forms and it is quite usual that several different „style“ characteristics are present on a single structure. Indeed, the previously adopted techniques and forms are here simply inherited and compiled with new and recently discovered techniques and are by no means excluded as old and outdated, which characterizes vernacular construction. This could be a significant reason why, in terms of style and construction, the same form is encountered in the period from 1400 to 1900, when the style framework was replaced with a strong need for utility.



Fig. 9. Stone bridge construction at Plandište near Blažuj (Sarajevo surroundings)

KARANSKI THREE-ARCH STONE BRIDGE CONSTRUCTION ON THE RAKITNICA RIVER IN ROGATICA – FACSIMILE RECONSTRUCTION – THE BRIDGE CONSTRUCTION HAS NOT BEEN PUT UNDER PROTECTION NOR ENTERED INTO THE REGISTRY – THERE IS NO RECORD NUMBER OF THE PLANOOTHEQUE

### 3. Description of the method of photo-digital analytic approach

We will begin the discussion of the principles of protection in Rogatica with the fact that the town is one of the settlements that have existed from the Roman period through the Bosnian medieval state, the Ottoman Empire and further on. Two three-arch stone bridge constructions were preserved until the outbreak of the Second World War (more accurately, the Lower Bridge construction until 1942, and Karanska Bridge construction until 1944). Both date back to the period of Turkish administration. It is difficult to determine whether it is the 15<sup>th</sup>, 16<sup>th</sup>, or even 17<sup>th</sup> century. There are no preserved graphic or photogrammetric records for either of the stone bridge constructions. There is no evidence apart from traces in *sidžils* (court minutes) of a *vakufnama* (deed of donation of a court decision character) and several old photographs and picture postcards; thus, there is no firm base for the attempt of facsimile reconstruction. The experience of facsimile reconstruction of the city of Warsaw (Poland) as an ambience ensemble, which was defined and adopted through countless photographs from family albums and survey questionnaires encouraged us in our endeavour to attempt to apply the already tested methods, as well as some new ones, which we have just introduced through a given character of digital processing of the existing light recording - photograph, the process of which as a method we will attempt to officially present and systematize for the first time. As was already pointed out, the benefit of this method is that using some known dimensional parameters and their proportional positioning on the digital recording we obtain all or most of the desired values and parameters.

The prerequisite for the functioning of such a methodology is the existence of any kind of the previous fixation of optical image, in the form of manually delineated light drawing by means of *camera obscura* or a template which is a product of the photo-chemical, photo-electronic, photo-digital, etc. transformation process. We can use things ranging from the appropriate forms of drawings made by means of *camera obscura*, through static photographs of any modality to kinetic recordings, also of any modality (classic "film", electronic "video" or digital "DVD" recording). In the absence of such forms, one can also use printed forms of light recordings of all kinds (semitone monochrome and multicolour offset forms, graphic "*Strich*" of highly contrasted monochrome or multicolour forms, serigraph forms, etc.), although the process of further digital processing becomes somewhat more complex. To digitalize such recordings, it is necessary to use certain filters to annul different forms of grids that are constituent part of such forms.

The second prerequisite is the possession of correct and checked parameters from the given structure (at least one). In general, it suffices to have only one correct (reliable) value that pertains to the height or width of a given part, parts or the whole of the structure. Naturally, the greater the number of accurate/correct dimensional parameters the more precise and complete the result of this method will be.

The third prerequisite is knowledge of distortional anomalies that occur during the inappropriate use of "classical" – analogue photo camera in the process of light recording – "shooting" and the removal of the present distortions during digital processing. (Fixations of optical images obtained using cameras for photogrammetric shooting of structures, with proper serving, do not imply the presence of such a character of distortions.). It is also inevitable to be familiar with the work on computers using a software for digital – pixel image processing (any version of Photoshop or Corel painter is recommended).

Manipulative work within this method can be observed in the following stages:



**1. Digitalization of any form of the optical image fixation in a static or kinetic modality.**

Digitalization of static modalities is possible using scanners or digital cameras intended for such kind of reproduction. Digitalization of kinetic modalities implies possession of specialized equipment for this purpose, which is available in different constructive-technological variants depending on whether it is a classical film or video, or else DVD recording. Multicolour or monochrome condition of the recording is irrelevant. The aim is to obtain a digital recording with quality characteristics in terms of its size (height x width x resolution expressed in units "dpi per inch").

**2. Analysis and processing of digital recording.**

In this stage it is essential to determine the "optical condition" of the structure, i.e. detect certain distortional anomalies resulting from the inappropriate angle of shooting or the improper use of lens (this refers to the focal length of the lens compared to the proportional value of camera format). In processing a digital recording, it is necessary to remove any kind of possible distortional anomalies as efficiently as possible and obtain as regular and readable, e.g., upstream façade of a given bridge construction, as possible. It is followed by the suitable processing of tonality to increase or reduce contrast, i.e. the presence or absence of certain tonal colouring to increase the linear-constructive readability.

**3. Work on pre-metric form.**

This stage requires maximally precise work and appropriate equipment, mostly the use of larger monitors. The form of digital recording obtained in the previous stage can be transformed into the form of contrasted linear drawing, although attention should be paid to the quality of transformation in terms of defining the optimum thickness of contour lines depending on the format of digitalized form.

**4. Establishing metric relations and reading of desired values.**

Based on the known numeric value(s) on the selected structure, by correcting (complete diminishing or enlarging, or partial correction of the vertical or horizontal axis) the previously obtained form, we establish the basic metric relations. Alignment of the known dimensional parameters with the linear drawing yields the metric form of the structure, and the only step that remains is reading and entering the needed/required values.

**5. Plotting the final form of the structure.**

The plotting process itself can be carried out in two ways. The first is to make use of the previously obtained linear drawing, possibly with an additional retouch, and the second one is to manually delineate the existing linear drawing to reach the appropriate presentational form. This stage of work is possible both in the previously mentioned pixel software and in the software that operate with vector values.<sup>3</sup>

**4. Concluding remarks**

With respect to this stone bridge construction, it is necessary to add several more general remarks about the choice of method and protection principles. The method of facsimile reconstruction is generally the method of choice, since our basic view is not to make copies without a valid reason for doing so. The "facsimile" method is a method we recommend in the case of substituting a missing segment although in this case the bridge construction would greatly improve the quality of the environment it is located in. Indeed,

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<sup>3</sup> With the expressed good will and elaboration of several examples within the framework of the presented methodology, joint commitment and work with prof. dr. Mehmed Akšamija, we obtained the described results and the definition of the technological processing procedure and the way of drawing metric conclusions. The initial research and application of the results were conducted at the Academy of Fine Arts of University of Sarajevo – photography studio, in 1998.

this town in East Bosnia has, to a great degree, lost its identity and relationship with the past, due to the post-war renewal and construction (we refer to the period after the Second World War) and the uncontrolled demolition of traditional (historical) structures. Re-building this bridge construction on the original location would bring harmony among the several remaining old residential houses and several different religious facilities built in the same or approximately same period. In this way we would create the renewed quality of ambiance as context, which has been significantly devastated, and it would fully honour Article 9. of the Venice Charter.

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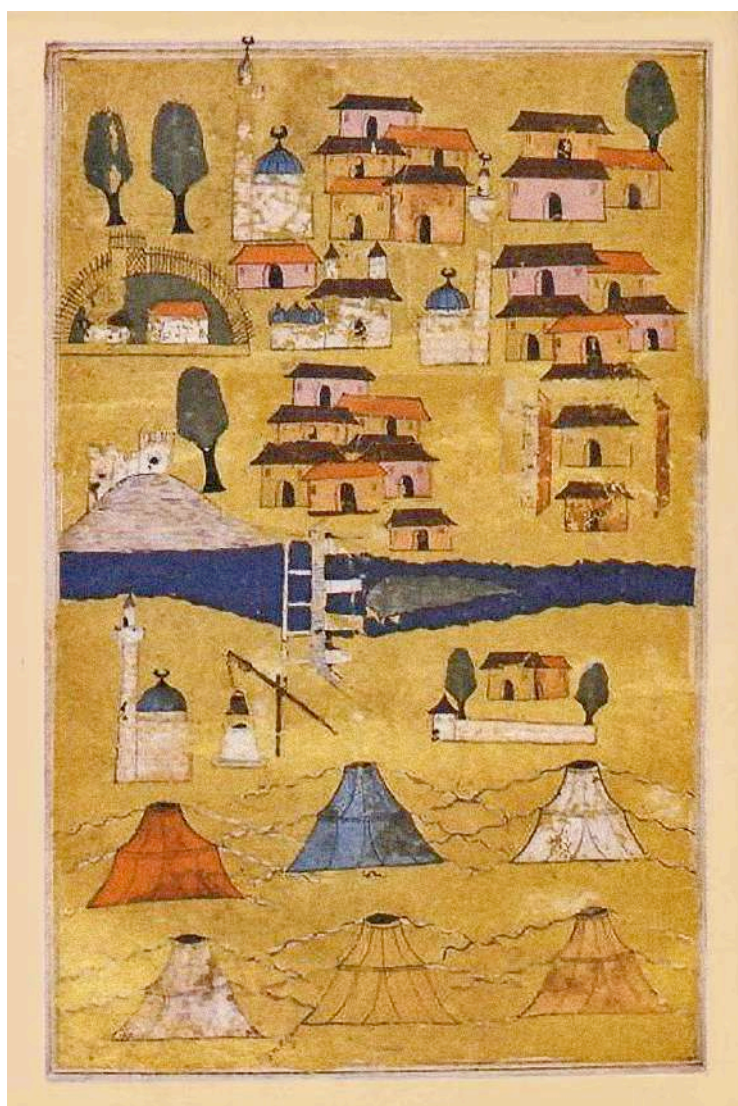


**Fig. 10.** Stone bridge construction with the wooden parapet in a miniature, presumed to depict Čelebipazar/Rogatica, from the late 16<sup>th</sup> century – unknown author





**RECONSTRUCTION OF THE ARCHITECTURAL AND CULTURE  
MONUMENT, "TOBACCO WAREHOUSE" IN KUSTENDIL,  
IN A COMPLEX MUSEUM AND CULTURAL INFORMATION CENTER**

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**Abstract:** *The building is a masterpiece of the industrial architecture in the thirties of XX century. The original design is the work of the architect Rudolf Fischer. The construction of the building is realized in the period 1924-1925. The combination of different construction materials: masonry, reinforced concrete and timber, gives a unique structure. This contribution introduces a structural investigation, using MFE software, into the stability and efficiency of the design of the roof truss of the storage heritage building in Kustendil. A two dimensional model was used in numerical analysis to discuss and highlight structural sense of Austrian architect R. Fischer in the period of 1925-1925. The modelling and the final conclusions are basic for the decision concerning the possible interventions in the building and the possibility to extend the service life of the roof structure.*

**Key words:** *masterpiece, industrial architecture, intervention, structural investigation*

**Introduction**

The building is a model of industrial architecture in the thirties of the twentieth century, which still dominates with its volume in the modern urban structure in the town of Kyustendil. It was designed as a tobacco warehouse by the Austrian architect Rudolf Fischer, who lived in Kyustendil in the period 1909-1934. The building was built in the period 1924-1925. The plan scheme is a one-dimensional structure dissected by the rhythm of the columns. The scheme is symmetrical with an axis of symmetry defined by the projection middle part, covering the stairwells and the entrance of the building. The building is included in the list of buildings declared as cultural monuments from the period 1878 - 1944, approved by the director of NIPC in the town of Kyustendil "with letter №2765/17.07.1979. An immovable cultural property with a category of "local significance" was declared after a meeting with document: №6/10.03.1997 and approval, and was published in SJ, with No.: 25/04.03.1998. After a meeting with document: № RD 09-29/29.01.2008, the proposal of the director of RIM Kyustendil to change the building to

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"national significance" is accepted. The urban situation of the building complex of the Regional Historical Museum of Kyustendil is extremely favorable. The terrain is located near the central part of the city, east of the attractive pedestrian "green" axis, connecting the central square spaces with the station square/railway station and bus station. The architectural volume of the central building is clearly expressed in the silhouettes on the main pedestrian axis - from the west, park and square space - from the north and street - from the east, as well as from the tall buildings of the city or in the panoramic view from the southern hill "Hisarlaka". The facades of the building are flat and treated equally on all four sides (Fig. 1a,b,c,d). The windows are in pairs alternating with flesh. The height of the windows in the different floors is different with a certain gradation. The attic space is illuminated with dormers along the long axis and with windows along the narrow fronts (Fig. 1.1a,b,c,d).

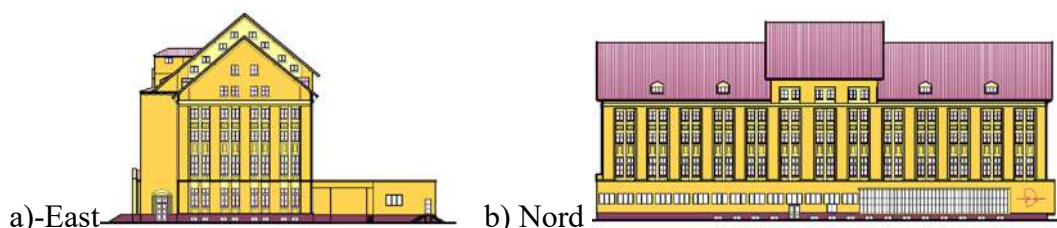


Fig.1.1a,b. East elevation-a), Nord elavation-b)

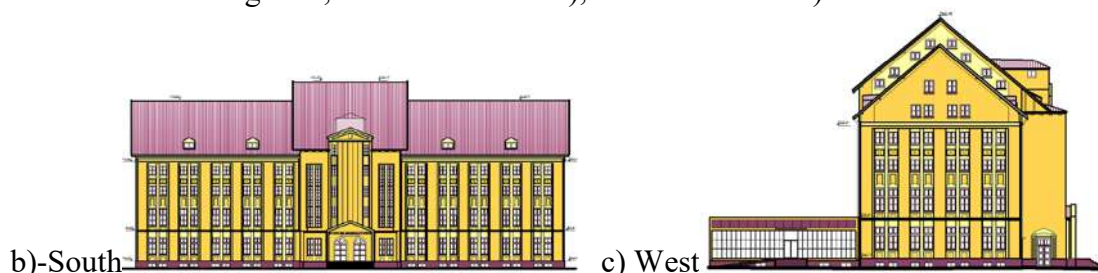


Fig.1.1c,d. South elevation-c), West elavation-d)

The height of the building consists of a semi-dug basement, three main floors, a mezzanine and attics. The presence of a high brick superstructure favors the use of the attic space. The building is designed as one main volume, with a rectangular base and five main floors, basement and attic floor: Basement, elevation -2.92, with:  $H_{light} = 2.75$  m; First floor, elevation -0.00, with:  $H_{light} = 3.96$  m; Second floor, elevation: +4.13, with:  $H_{liht} = 3.88$  m; Third floor, elevation +8.26, with:  $H_{light} = 3,50$  m; Fourth floor, elevation +12.01, with  $H_{light} = 1.95$  m( installation ); Fifth floor, elevation +14.21, with:  $H_{light} = 2.68$  m; Sixth and seventh attic floor: elevation: +17.14 and elevation: +19.73 are from wooden structure (Fig.1.2.).

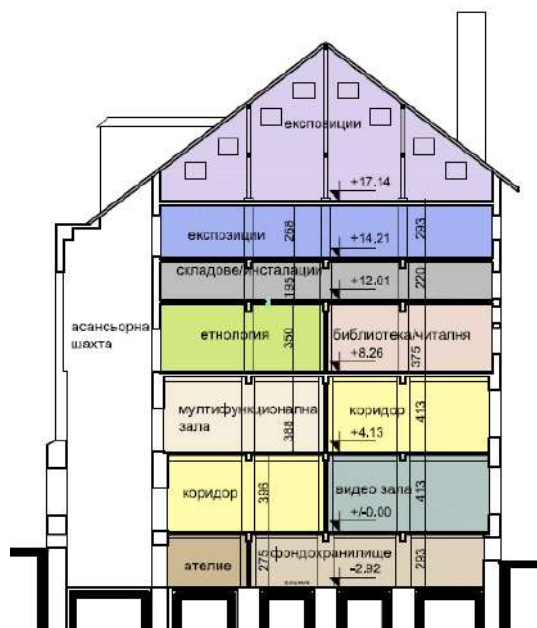


Fig.1.2. Cross section of the Museum building

The building in plan has rectangular dimensions with a width of 18.1 m and a length of 59.79 m and a built-up area of approximately 1082m<sup>2</sup>. In the transverse direction, the columns of the building are located in three wheel bases with dimensions: 4.60 + 4.00 + 4.00 + 4.60 = 17.200m, which is the light spans of the building. In the longitudinal direction, in the ground floor the columns are arranged as follows: 5.00 + 4.40 + 4.40 + 4.40 + 4.10 m (Fig.1.3, Fig. 1.4 and Fig.1.5).



Fig.1.3. Horizontal section: over Ground floor





Fig.1.4. Horizontal section: over 4- floor

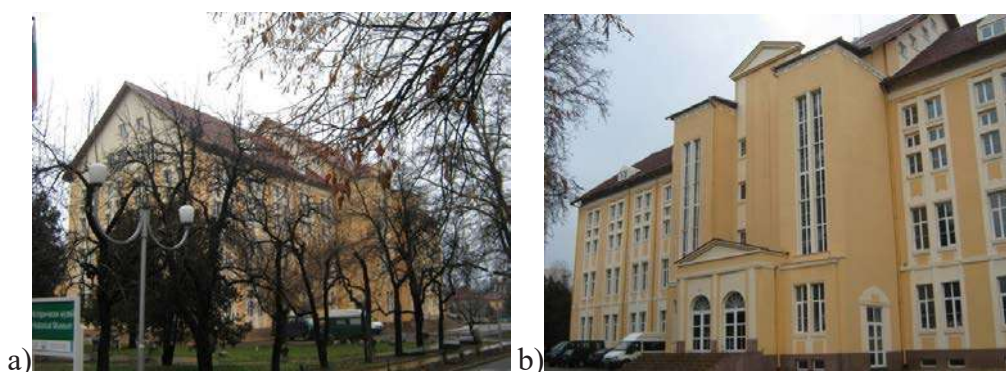


Fig.1.5a,b. General view of the building and the south-east facade

## 2. DESCRIPTION OF THE CONSTRUCTIVE SYSTEM OF BUILDING

The construction of the building is massive with external load-bearing brick walls 80 cm thick in the basement, 60 cm on the first and second floor, 45 cm on the third and fourth floor, 40 cm on the fifth, sixth and seventh floors. The columns are from reinforce concrete with dimensions 78/78 in the basement , and from brick with dimensions 64/64 cm on the first and second floor, 50/50 cm on the third and fourth floor (Fig. 2.1a). In the transverse direction, the columns of the building are located in three wheelbases with dimensions:  $4.60 + 4.00 + 4.00 + 4.60 = 17,200$  m, which is the light spans of the building. In the longitudinal direction, in the first section the columns are arranged as follows:  $5.00 + 4.40 + 4.40 + 4.40 + 4.10$  m. The mezzanine structures at elevation:  $+(-) 0.00$  (slab above the basement) and at elevation:  $+4.13$  m (slab above the first floor) are a beam system with monolithic reinforced concrete slabs and beams, which lie on brick (respectively) RC columns (Fig.2.1b).



**Fig.2.1a,b.** Distribution of the columns in the basement of the building-a), View on the floor and column in the first level of building-b)

At elevation 8.26 m (slab above the second floor), as well as at elevations +12.01 (slab above the third floor) and + 14.21 m (slab above the fourth floor) the structures are made of wooden beams with a caratavan ceiling. The wooden beams are oriented in the longitudinal direction of the building and are supported in the transverse axes of the building on steel double T-beams, which in turn lie on the columns of the building (Fig. 2.2a,b,c). The wooden beams have different sizes varying in the range: 16/18, 18/20, 17/17, 20/20, 17/18, 18/18. The 7 timber beams are located in the end and 5 in the intermediate axes of the building. The steel beams on the second floor are №30 and on the third and fourth are №22 and №24. The floors are made of wooden sheets, placed on a joist structure of the ridge (Fig.2.2a, b,c).



**Fig.2.2a,b,c.** Supporting of the wooden beams on steel double T- beams; Supporting of the steel beams on the columns of the building.

View inside on the floor construction is shown in the Fig.2.3.a,b.



**Fig.2.3a,b.** View on the beams in the floor construction

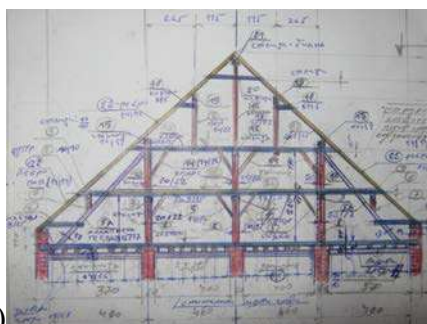
The floor construction of the elevation: + 17.14 m (ceiling slab) is also made of wooden joists, lying on steel beams located in the transverse axes of the building, which rest on the brick (respectively. Concrete columns) (Fig. 2.4a,b).



Fig.2.4a,b. Floor construction of the elevation: +17.14m (ceiling slab), made of wooden beams, lying on steel beams located in the transverse axes of the building, which are supported by the brick (respectively: RC columns).

### 3. DESCRIPTION OF THE CONSTRUCTIVE SYSTEM OF TIMBER ROOF OF BUILDING

The roof structure of the museum in Kyustendil is a gable roof with alpine type and height to the ridge:  $H = 10.10$  m from the level of the ceiling slab. The base of the isosceles configuration of the gable roof is light in size between the enclosing walls: 17,220 m. The supporting structure of the roof is made entirely of wood (Fig. 3.1a, b). The roof structure is made entirely of wood - made of shaped wood. The roof is gabled with a steep slope covered with marseille tiles on a wooden cladding.



a)



b)

Fig. 3.1a, b. The drawing of the the roof truss structure- and View on the timber roof

The main load carrying elements are: binder, columns-with capitals(from diagonal supporting struts), or slopes to each columns, purlins in the longitudinal direction of the building(Fig. 3.2a,b,c) of the three-storey roof structure and also the presence of kings on the third level of the structure (Fig.3.2a,b,c).



a)



b)



Fig. 3.2a,b,c. Binder, columns (kings) anchored on a binder beam; capitals in the longitudinal direction, slopes to each column; Pliers and kings at each level of the three-storey structure of the roof

The roof construction is characterized by the presence of inclined beams at the beginning and end of the main spine of the structure, which start from the floor beam above the wooden floor, pass through the two-piece construction of pliers, located above



the masonry wall and at the level of the first floor joist, reach second level of the structure and, resting on the side at the top of the column of the second level (Fig.3.3a,b, c). The inclined beam (**pos. 5A**), which is supported at its lower end in the main row (**pos. 1**), is supported at its upper end in the two end columns (**pos. 10**). These inclined beams in all wheel-bases are: **15/17 cm**. It is gripped in turn with pliers (**pos.6**) to the roof rib. This small pliers is made of two planks with dimensions: **8/15 cm**. Above this pliers on the inclined beam is located a little column (**pos.7**) with dimensions **14/14 cm**. Above it is a longitudinal purlin (**pos.9**) with dimensions **10/10 cm**, under which a pillow (**pos.8**) with dimensions **10/10 cm** is mounted.



**Fig. 3.3a, b, c.** Masonry wall attics and inclined beams at the beginning and end of the main spine of the structure

The roof ribs resting on the little masonry attics have an independent slope (Fig. 3.4a,b). A 4 cm thick wooden cladding is nailed on them, on which Marseille tiles are arranged. The ribs are located at approximately equal distances from each other, varying in the range of : 40-50-60-70 cm. There are usually 7 ribs in each field.

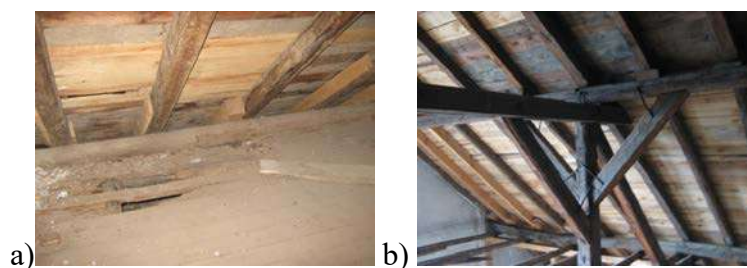


**Fig. 3.4a,b.** Second floor of the roof; ribs supported by pliers

The three-storey wooden structure of the roof starts from a main wooden beam (**pos.1.-binder**), lying on the bricks, respectively on the concrete columns (Fig.3.5). The binder of the roof truss is supported continuously on the beams of the floor joists of the ceiling, which in turn are supported on the transverse steel double T beams with a height of 22 cm. This method of support presupposes the calculation of the binder as a continuous beam, which must be checked for bending obtained from the positive and negative moments obtained in it (in the continuous beam) and also for axial forces obtained in the binder during its participation in the roof truss structure. It consists of separate single-span beams with dimensions: 20/22 cm, fast connected each other. On them in the places of the massive columns, wooden columns (**pos.2-kings**) with dimensions 20/22 cm are mounted(Fig.3.6).



**Fig. 3.5.** Supporting the binder on the floor joist; supporting the floor joist from the ceiling on the steel beams; Supporting a column (king) from the bar truss on a brick column



**Fig. 3.6.** Connection between binder and column (king)

The wooden columns are 3 in the first level and are at a distance of 400 cm from each other in the transverse direction (Fig.3.7).



**Fig. 3.7.** Floor joists of the first new of the spatial structure



**Fig. 3.8.** Connection between column, purlins and pliers on the first level of the roof

Purlins with dimensions: **18/20 cm (pos. 3)** are laid on them in the longitudinal direction. Above the longitudinally oriented purlins, in the transverse direction are mounted wooden pliers (**pos.4**) with dimensions:**2x8/15cm**, which are supported by braces (**pos.5**) in the transverse direction (Fig. 3.8). The diagonal struts have the dimensions of their cross sections: **14/16 (pos.5)**. The pliers (**pos. 4**) surround **the columns** of the second level (**pos. 10** -2 pcs., and pos. 11-1 pcs.), on the side, as the connection between them is with a bolted connection (Fig. 3.9 and Fig. 3.10). The pliers(**pos.4**), which are made of planks with dimensions: **2x8/15**, grip each other at both ends, the inclined beams to the main supporting structure as well as the corresponding roof ribs, which are located in this axis.



**Fig. 3.9.** Connection between column, purlins, pliers and struts on the second level of the roof

Parallel to the pliers on the capitals a wooden beam with circular cross section has been developed: round wood:  $\Phi 12/12\text{cm}$ ; as well as with rectangular section: 14/15 cm; 12/15; 16/15; 18/15 cm. Half of this wooden platform is plastered (Fig. 3.10a, b, c).



Fig. 3.10a, b, c. View on the second level of the roof structure

The clear distance between this floor structure and the lower level is equal to 225 cm. Above this floor structure at a distance of 170 cm from it is located a second level - trimmer joists, parallel to the horizontal pliers - **pos.13(2x8/15)**. Above the central king of the second level, longitudinally on the roof is a purlin situate: (pos.12) with dimensions 18/20 cm. At both ends of the pliers (pos.13) are located longitudinally on the roof purlins: (**pos.15**) with dimensions **16/19** cm (Fig.3.10a, b, c). On these purlins, located next to the contour of the roof, the roof ribs rest laterally. The third level of the roof structure consists of a central "king" (pos. 16) with dimensions **20/22** cm, which at the top supports a ridge purlin: (**pos. 21**) with dimensions: **18/20** cm Fig. 3.11). The central king lies at his lower end on the capital, passing through the pliers - **pos.13**. Under the ridge purlin there is a pliers (**pos. 20**) with dimensions: **2x8/15** cm, which covers the roof ribs, which fall into its axis and is connected to them with a bolted joint. In the longitudinal direction this "king" is reinforced with two diagonal struts: (pos.24) with dimensions 14/14 cm.



Fig. 3.11. Central column (king), roof purlin in the longitudinal direction, diaonal slopes to the central column, pliers under the roof of the three-storey roof structure

At a distance of 195 cm from the central king are located on both sides two hanging kings (**pos. 17**) with dimensions: **20/22** cm. They are supported in the pliers below them(**pos.13**), passing through the joint formed by the two planks: **2x8/15** cm (Fig.3.12). The connection between them is realized with a bolted connection BM20. At its upper end this king (**pos.17**) supports a longitudinal purlin: (pos.19) with dimensions: **16/19**cm. Pos.17 is supported laterally with pliers: (**pos.18**) with dimensions: **2x8/15** cm.





**Fig. 3.12.** Central column (king), side hanging columns (kings), roof purlins in the longitudinal direction, diagonal slopes to the central column, pliers under the roof of the three-storey roof structure on both levels

#### **4. COMPUTATIONAL 3D MODEL OF THE TRUSS CONSTRUCTION OF THE BUILDING**

##### **4.1. Load model**

Loads are adopted pursuant to Ordinance № 3/2004 years. "Basics of designing the structure of the buildings and the impacts on them" for the region of Kyustendil as follows: snow region III with value  $130 \text{ daN/m}^2$  and wind to area II with value  $32 \text{ daN/m}^2$ . Computing values of these values are achieved by multiplying the normative values of the two impacts with load factor:  $=1.50$ . Studied are all occurring in wooden frames species components. Static analysis of the various structural elements of the roof done according to the methods of structural mechanics and verified with FEM program "SAP2000". Static schemes of frames and roof are modeled as plane joint-rod systems. All fixtures in the wooden structure of the roof are filled with bolt-screws. Dimensions of the different components of the roof frame of strength, stability and deformability are carried out according to EC5, 2007. The quality of the wood, when testing the structural elements of the roof frame is adopted as a second category (SI-II). On the Fig.4.1 is demonstrated the Computational 3D model of the truss construction of the museums building. The drawings of the main truss construction is on the Fig. 4.2 Cross sections of the constructive element of the timber truss system is the Tab. 1. As additional information is important to know that: length of the truss is: 17200mm, and high: 10010mm. The length of the roof is: 59,79m, width: 18,10m and high: 10,10m.

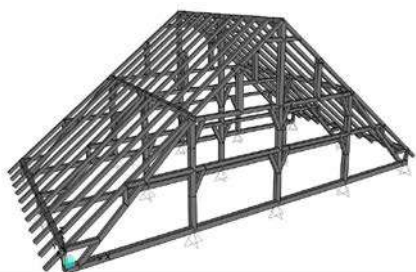


**Fig. 4.1.** Computational 3D model of the truss construction of the museums building in Kystendil.

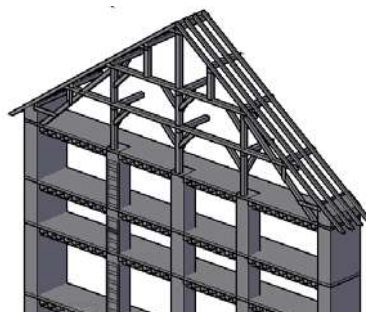


**Fig. 4.2.** The drawings of the truss construction of the roof

**Tab. No. 1: Cross sections of individual structural elements**

| <b>Cross-Section</b><br><b>Průřez</b> | <b>b (cm)</b> | <b>h (cm)</b> |
|---------------------------------------|---------------|---------------|
| Pos.1 -binder                         | 20            | 22            |
| Pos.2-column                          | 20            | 22            |
| Pos.3-purlin                          | 18            | 20            |
| Pos.4-pliers                          | 2x8           | 15            |
| Pos.5/5A-incline beam                 | 14/15         | 16/17         |
| Pos.6-pliers                          | 8             | 15            |
| Pos. 7 -column                        | 14            | 14            |
| Pos. 8-pilow                          | 10            | 10            |
| Pos. 9-purlin                         | 10            | 10            |
| Pos. 10-column                        | 20            | 22            |
| Pos.11-column                         | 20            | 22            |
| Pos.12-purlin                         | 18            | 20            |
| Pos.13 –pliers                        | 2x8           | 15            |
| Pos.14 diagonal bar                   | 14            | 16            |
| Pos.15-purlin                         | 16            | 19            |
| Pos.16-colum                          | 20            | 22            |
| Pos.17-column                         | 20            | 22            |
| Pos.18-pliers                         | 8             | 15            |
| Pos. 19 -purlin                       | 16            | 19            |
| Pos. 20-pliers                        | 2x8           | 15            |
| Pos. 21-purlin                        | 20            | 20            |
| Pos. 22-rib                           | 14            | 14            |
| Pos.23-timber sheets                  | 17            | 17            |
| Pos.24-inclined supports bar          | 16            | 16 r          |

## **5. CALCULATION OF THE CONSTRUCTION ACCORDING TO THE THEORY OF THE SECOND ORDER**

In order to be able to comment on the reserves of the truss supporting structure, we performed a full calculation links with SAP2000 software, according to theory I. and II. order.

**Tab. No.2: Load combinations (EC5, 2007).**

| <b>Load combinations</b>          |
|-----------------------------------|
| 1.35 DL + 1.5 SL                  |
| 1.35DL + 1.5SL+ 0.75WL            |
| 1.35DL+ 1.5WL (pressure) + 0.75SL |
| 1.0DL + 1.5WL (suction)           |
| 1.35DL + 0.75WL                   |

**Tab. No.3: Load values**

| <b>Type</b>     | <b>Load (kN/m<sup>2</sup>)</b> |
|-----------------|--------------------------------|
| Covering        | 0.70                           |
| Snow            | 1.30                           |
| Wind (pressure) | 0.32/-0.32                     |
| Wind (suction)  | -1.00/-1.00                    |

The combinations of loads and values are given in the tables: 2 and 3. Due to the age of the construction were created extraordinary load values, which in this case can count on a 100-year return Scheme of external load: dead load, snow and wind load and are given in fig. 5.1,a,b,c,d.

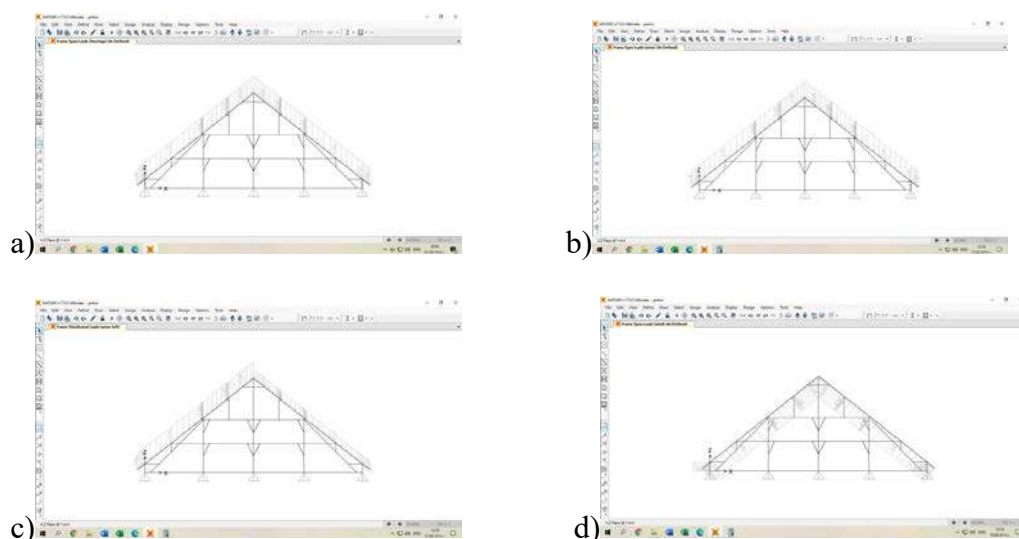


**Fig. 5.1,b,c,d.** Scheme of external load are given

Static scheme and courses of internal forces for dead load  $M$ ,  $Q$ ,  $N$  values according to theory of second order is shown in Fig. 5.2,a,b,c,d.

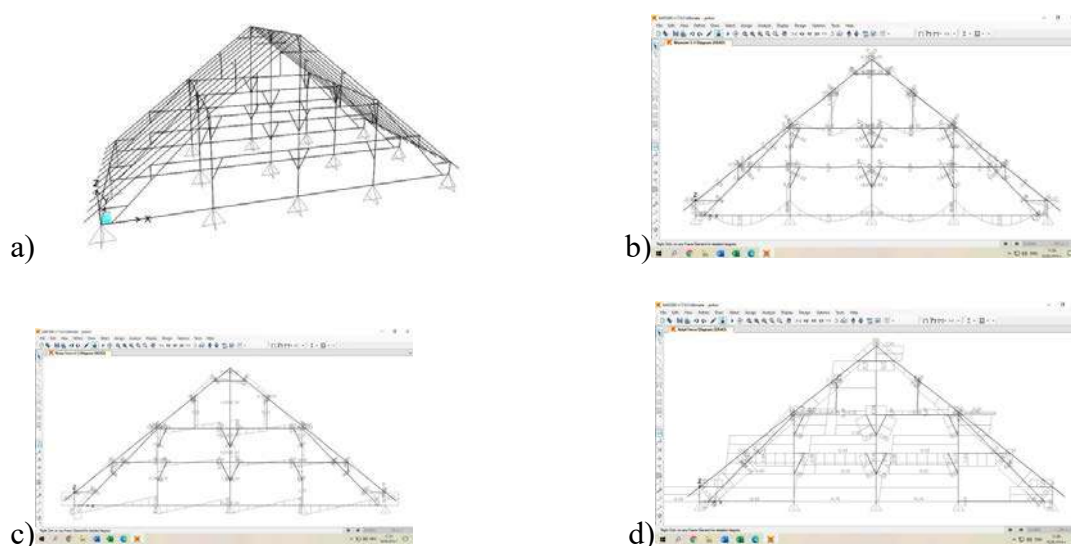


**Fig.5.2a,b,c,d.** Static scheme, Diagrams of bending moments- $M$ , shear forces ( $Q$ ) and normal forces ( $N$ ), according to theory II. Order

### Conclusion

The calculation verified that if there is no further significant damage to the carrier construction by biotic influences, there is no danger of the collapse of the truss of the building in Kustendil. If we think about the constructional feeling of the arch. R.Fischer, we must state that this was considerably advanced[1-20]. Although they had no modern computing resources, they did comprehensively combine various requirements for the functionality of the work, including some protection against biotic influences. From today's



point of view, the building withstood all the effects of a load with a 95-year return and still despite some damage to the truss does not threaten its immediate collapse[1-20].

### **Acknowledgement**

The research was supported within the research project: "Investigation of behavior between historic Timber Roof Structures and Masonry Building, due to load of Wind and Earthquake using MFE", contract number 2/2020-22, granted by the research Fund of Ministry of Education.

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## MECHANICAL ANALYSIS OF A HERITAGE STRUCTURE IN THE CITY CENTER OF SOFIA

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**Abstract:** Recent earthquakes have highlighted worldwide the high seismic vulnerability of the structures, in particular of the old masonry buildings. The scope of vulnerability analysis is to assess the propensity at seismic damage of buildings. In this perspective, the proposed research aims at estimating the seismic vulnerability of a historical building in a district of Sofia in Bulgaria and to improve the seismic behavior with retrofitting interventions. The building of study is a structure of 1908, located on the boulevard "Knyaginya Maria Luiza", near the women market and the Bania Bashi mosque and it belongs to the group of heritage structures in Sofia.

A mechanical analysis has been performed through non-linear analysis, using the 3Muri software and the expected damage has been estimated in terms of fragility curves. The analysis has shown the poor seismic performance of the building, so to require a structural retrofitting intervention with carbon fiber-reinforced polymers sheets and reinforced plaster aiming at improving its seismic capacity.

**Key words:** vulnerability assessment, masonry building, heritage buildings, fragility curves, mechanical analysis, seismic retrofitting.

### 1. Introduction

The proposed research, which has been developed in collaboration between the University of Naples "Federico II" and the University of Structural Engineering & Architecture (VSU) "Lyuben Karavelov" of Sofia, aims at estimating the seismic vulnerability of a heritage structure in the city center of Sofia, through non-linear static analyses performed using the 3Muri software [1].

### 2. Sofia's seismic history

Bulgaria is divided into three seismic zones, characterized by the seismic intensity of the site for an average return period of 1000 years. Sofia is part of seismic zone IX, where earthquakes with maximum acceleration of 0.4g can occur.



In 19<sup>th</sup> century, the city of Sofia was affected by two strong earthquakes: the first one in 1818 with epicentre intensity  $I_0 = \text{VIII-IX MSK}$  and the second in 1858 with  $I_0 = \text{IX-X MSK}$ . During the 20<sup>th</sup> century the strongest events occurred was in 1917, closed to the city of Sofia, with a moment magnitude  $M_w = 5.3$ , and in 1980, 30 km far from the city of Sofia, with  $M_w = 4.3$  [2].

### 3. Building of study

The building of study (*Fig. 1a*) is a structure of the 1908, located on the boulevard “Knyaginya Maria Luiza”, near the women market and the Bania Bashi mosque. This building belong to the group of heritage structures in Sofia.

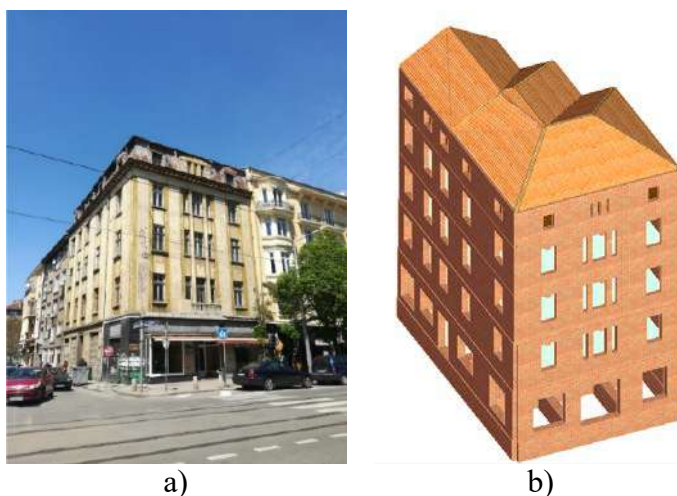


**Fig. 1.** The case study: street view (a), 3D microelement model (b)

It develops on five floors, one underground and it is 18 meters tall with a surface area of  $195 \text{ m}^2$ . The firsts two floors are in reinforced concrete, the third and the fourth with steel beams and hollow flat blocks. The foundation develops from the underground floor and it's  $1,50 \text{ m}^2$  tall, with a thickness of 80 cm. The first two floors present a masonry with a thickness of 60 cm, instead the following 3 floors present a masonry structure with a thickness of 40 cm and 30 cm. Finally, the last floor with the attic is in timber. With all these information, the building has been modelled in 3Muri software (*Fig. 1b*).

### 4. Mechanical analysis

The mechanical method has been applied through non-linear static analyses performed using the 3Muri software. Based on the results of the mechanical analysis, a set of fragility curves [3] have been plotted in order to predict the expected seismic damage in the two main directions, X and Y, of the building. So, for the Damage Limitation (DL) limit state, 100% of probability to be exceeded is reached for an acceleration higher than  $0,07g$ . Moreover, considering a Significant Damage (SD) limit state, the probability of exceeding this limit state is equal to 100% for accelerations higher than  $0,15g$ . Finally, there is for Near Collapse (NC) limit state a probability to be exceeded equal to 100% occurs for acceleration higher than  $0,18g$ . The results above illustrated are presented in Figures 2a and 2b.

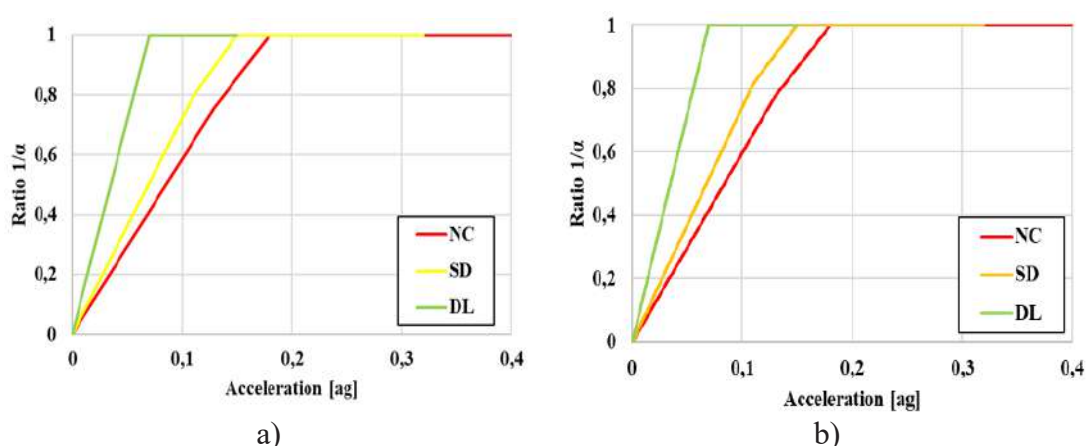


Figure 2. Fragility curves in X (a) and Y (b) directions.

## 5. Seismic retrofitting

The seismic analysis performed have shown that the risk index,  $\alpha$ , intended as the ratio between the capacity acceleration,  $PGA_C$ , and the corresponding demand one  $PGA_D$ , is lower than unit. These results are indicative of a critical response of the case study building, since the seismic demand is greater than the corresponding capacity. So, a structural consolidation has been speculated to increase the values of  $\alpha$ . A seismic retrofitting has been speculated, first using FRP system, and then with cement plaster reinforced with a 10x10 cm electro-welded mesh of  $\phi 8$  bars. To improve the seismic behavior, the consolidation has been speculated only for the walls that were damage with the first analysis. In the software, the element's colour indicates the type of the identified damage (Fig. 3-4).



Figure 3. Wall's deformation legend.

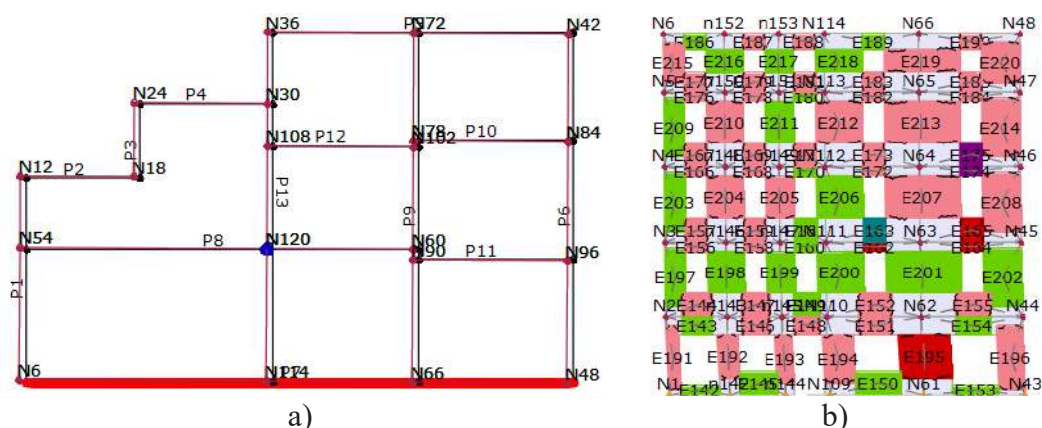


Figure 4. Example of a wall's deformation.

With the use of FRP system all the analysis have been verified for SD and DL limit states. Instead for NC limit state, the results have been not verified, but the behavior of the structure has been improved, infact for this limit state the values of the  $\alpha$  have increased of 0,5 in X and in Y direction. With the use of reinforced plaster all the analysis have been verified for NC, SD and DL limit states, infact all the values of the  $\alpha$  are higher than 1.

Following the comparison between the building behaviours before and after interventions, using the FRP system (Fig. 5) and using reinforced cement plaster (Fig.6). As can be seen from the presented pictures, the two interventions have shown a significantly ductile behaviour of the building, also with a considerable increase of both stiffness and strength.

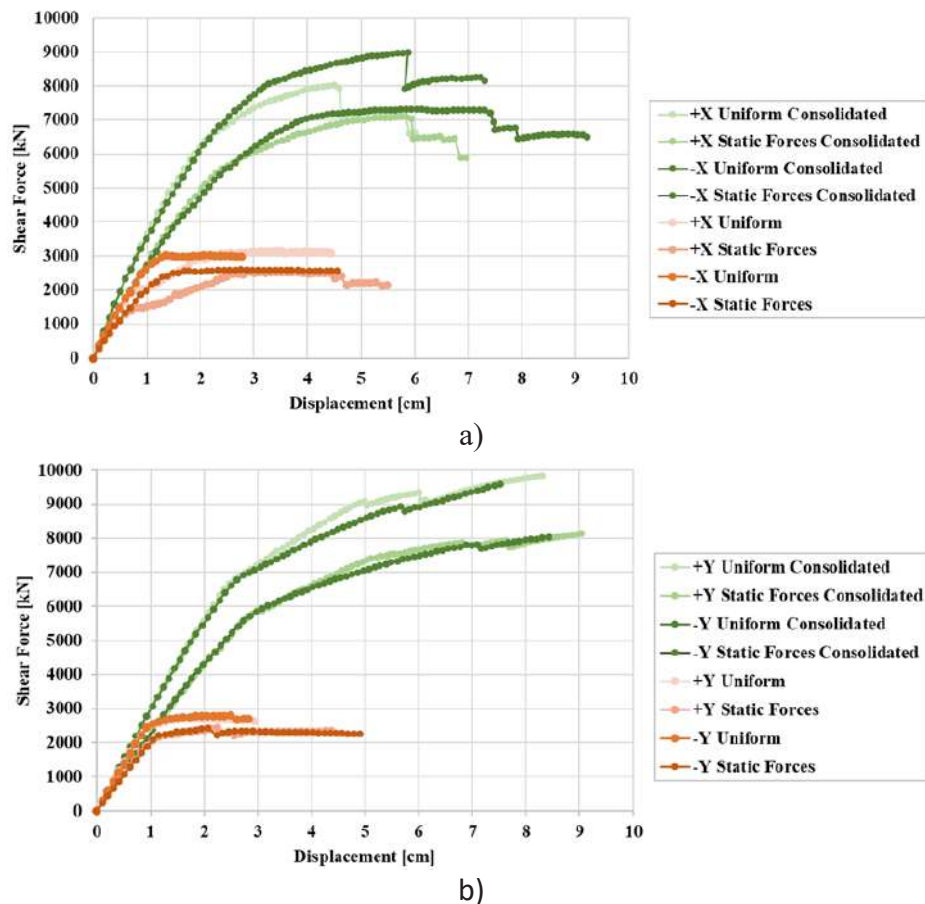
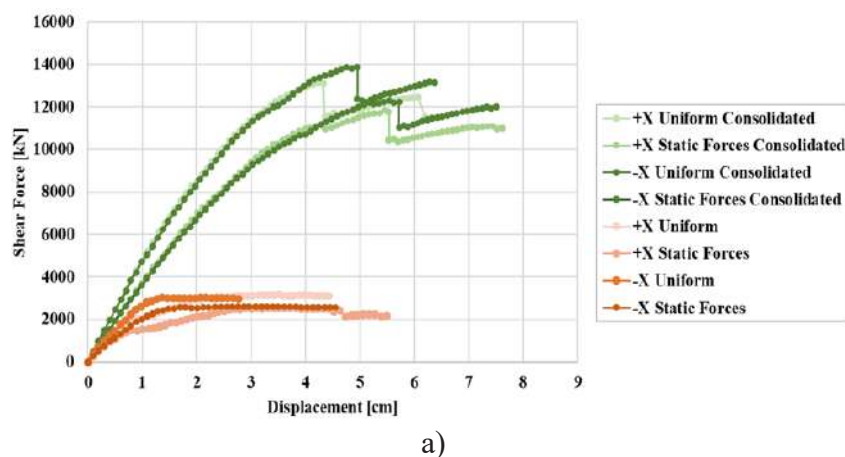
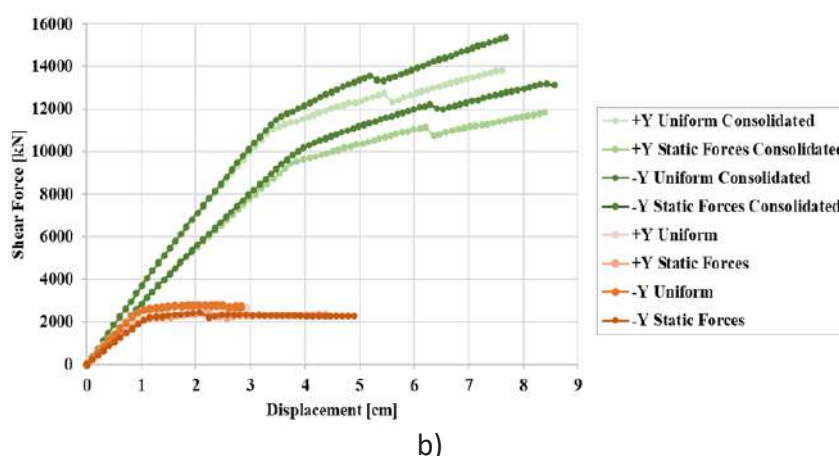


**Figure 5.** Comparison between the building behaviours before and after intervention using FRP in X (a) and Y (b) directions.







b)

**Figure 6.** Comparison between the building behaviours before and after intervention using reinforced plaster in X (a) and Y (b) directions.

## 6. Conclusions

The present paper examined the seismic vulnerability of a heritage structure in the city centre of Sofia. A mechanical procedure, through the use of the software “3muri”, has been applied to the case study. From the results obtained, it was observed an unsatisfactory global seismic behaviour of the building, with risk indexes in the two main analysis directions less than one. Therefore, two efficient retrofitting systems, FRP and reinforced cement plaster, were adopted to reinforce all masonry walls in order to improve the capacity of the structure in terms of strength, stiffness and ductility. The analyses on the retrofitted building demonstrated the effectiveness of the proposed retrofitting system since, for each reference limit state, the risk indexes were always less than the unit.

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