

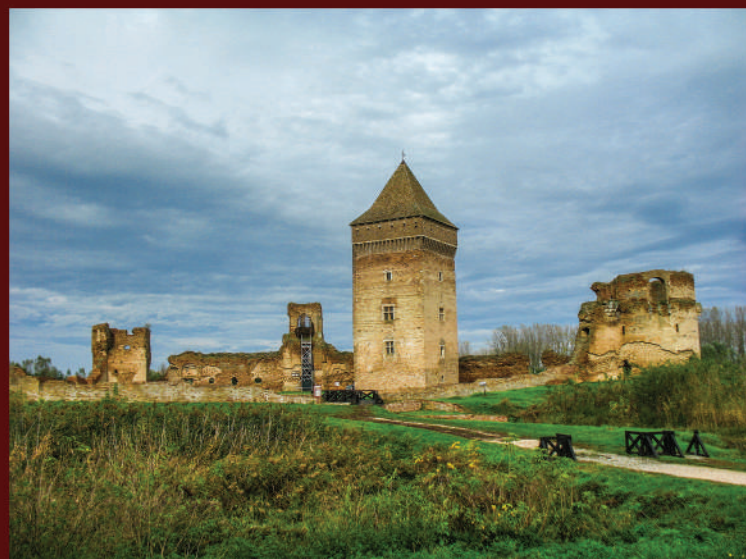


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КОНФЕРЕНЦИЯ
ПО ОПАЗВАНЕ
НА
НЕДВИЖИМОТО КУЛТУРНО
НАСЛЕДСТВО
БАНИ' 2022**

21-23 ноември 2022

**PROCEEDINGS
INTERNATIONAL SCIENTIFIC
CONFERENCE
PRESERVATION
OF CULTURAL HERITAGE
BASA' 2022**

21-23 november 2022



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BUILDINGS, DEMOLISHED DURING THE CONSTRUCTION OF THE SLOVAK NATIONAL UPRISING (SNP) BRIDGE

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Abstract: *Collection of rare photographs of old buildings located around Bratislava Castle which were destroyed during construction of SNP bridge over the Danube river in Bratislava.*

Key words: *collection of rare photographs, cultural heritage, buildings around Bratislava Castle, demolition of old Bratislava due to SNP bridge construction*

1. Introduction

SNP Bridge is the second permanent bridge over Danube River in Bratislava, the capital of the Slovak Republic. It is a steel cable-stayed bridge with a single backward inclined steel tower and a single cable plane. The asymmetrical position of the inclined 84.6 m high A-shaped tower, crowned by a circular restaurant with 32 m in diameter, creates a natural balance to the famous Bratislava Castle and the St. Martin's Cathedral. When it was constructed in 1969, the bridge, with main span 303 m, was the world record holder in the category of cable-stayed bridges. The bridge was the fourth-longest cable-stayed bridge span when it opened to traffic on August 29, 1972. It was the winner of the competition Structure of the Twentieth Century in Slovakia, organized in 2001. The original name of the bridge was Slovak National Uprising Bridge (in Slovak: Most Slovenského Národného Povstania). It was renamed, like other Bratislava Danube bridges, after the collapse of the communist regime in the beginning of the 1990s. The name "New Bridge" was valid only in the period 1993–2012. Before and after this period the name is SNP Bridge. The bridge was designed by a team from Slovak University of Technology (STU) in Bratislava led by Professors Arpád Tesár, structural engineer for steel structures,

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Ján Lacko, architect and Associate Professor Jozef Zvara structural engineer for concrete structures (Fig.1).

Just after winning the competition Structure of the Twentieth Century in Slovakia, organized in 2001 a lot of people attacked SNP bridge in media. The reason was his location intersecting historical old part of Bratislava around the Castle Hill.

The paper shows collection of several photographs of demolished buildings and their state in the time just before their demolition.

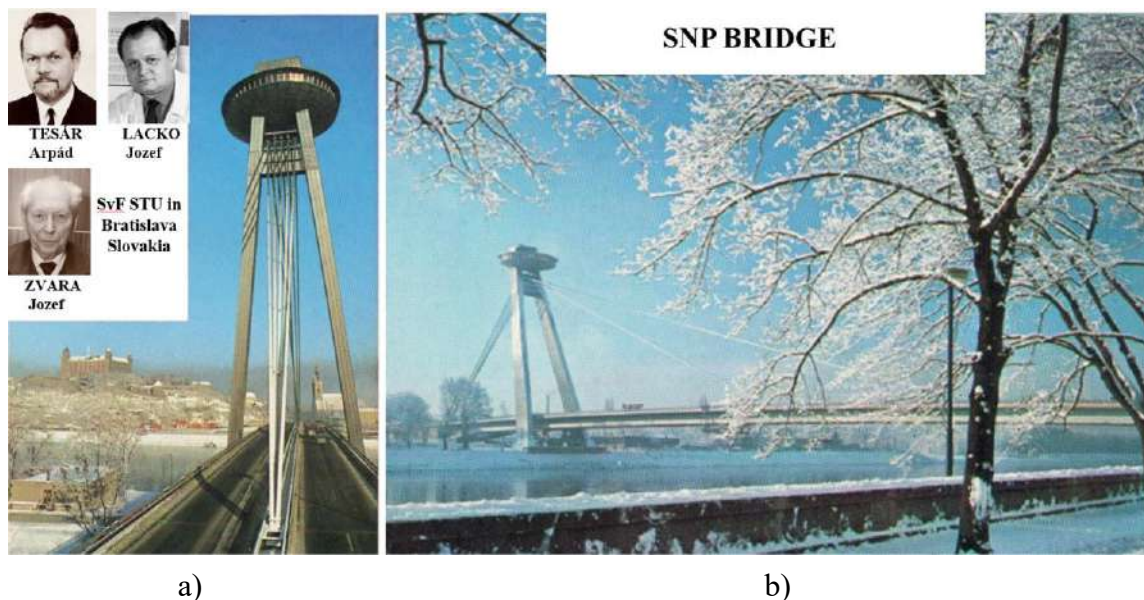


Fig. 1. SNP Bridge opened to the traffic in 29th August, 1972:

a) Photo: J. Bartoš, B. Schreiber, J. Sekal; b) Photo: J. Čeněk. PRESS FOTO ČSTK.

2. Collection of photographs of destroyed buildings

Figure 2 shows an aerial view of Bratislava in 1919-1921. The purpose of the three maps is: a) topography of Bratislava in 1958 (Figure 3), b) names and location of streets, squares and embankments in 1958 before SNP Bridge (Figure 4) and c) the changes in 1980 after building of SNP bridge (Figure 5).

The maps are followed by 150 photographs of buildings which were demolished during construction of SNP bridge in the sixties and the seventies. The buildings were photographed mostly in the sixties, some of them in the fifties. Most of the photographs were taken by Juraj Bončo [2] and Igor Scholtz [3]. Some of them are taken from the book [2] and from the portal PamMap [4] (courtesy of Professor Juraj Šedivý, founder of PamMap and owner of the big collection of photographs).

The following system is applied in the arrangement of photographs sequence:

- the photographs on a given street or embankment are arranged from the west to the east. In many cases this was possible to do it thanks to known house numbers;
- photographs of buildings on embankments next to the Danube river are treated as first. They are then followed by photographs of building in streets and squares arranged systematically from the south to the north. The beginning of the streets and embankments is always in their west end.
- the above system facilitates fining the positions of the buildings on the map in Figure 4;
- the names of the embankments, streets and squares are in English and in brackets (...) also in Slovak language, as they are written on the map in Figure 4;
- former names of streets are written in brackets [...];

- some embankments, streets and squares are named after important persons. The names of these persons are provided at the end of the paper.
- The older historical photographs may be found in the portal PamMap [4].

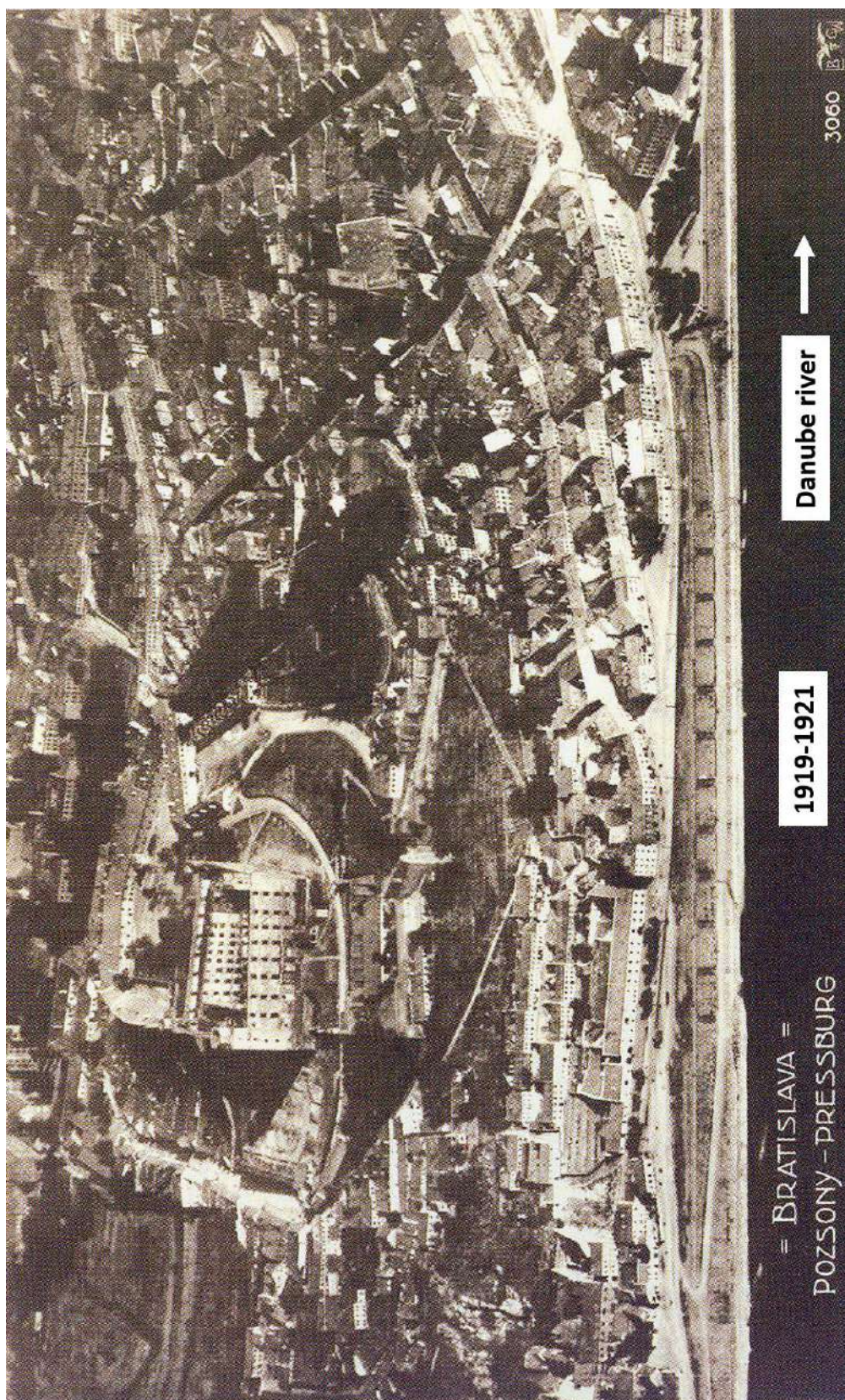


Fig. 2. Bratislava. Aerial view from 1919-1921.

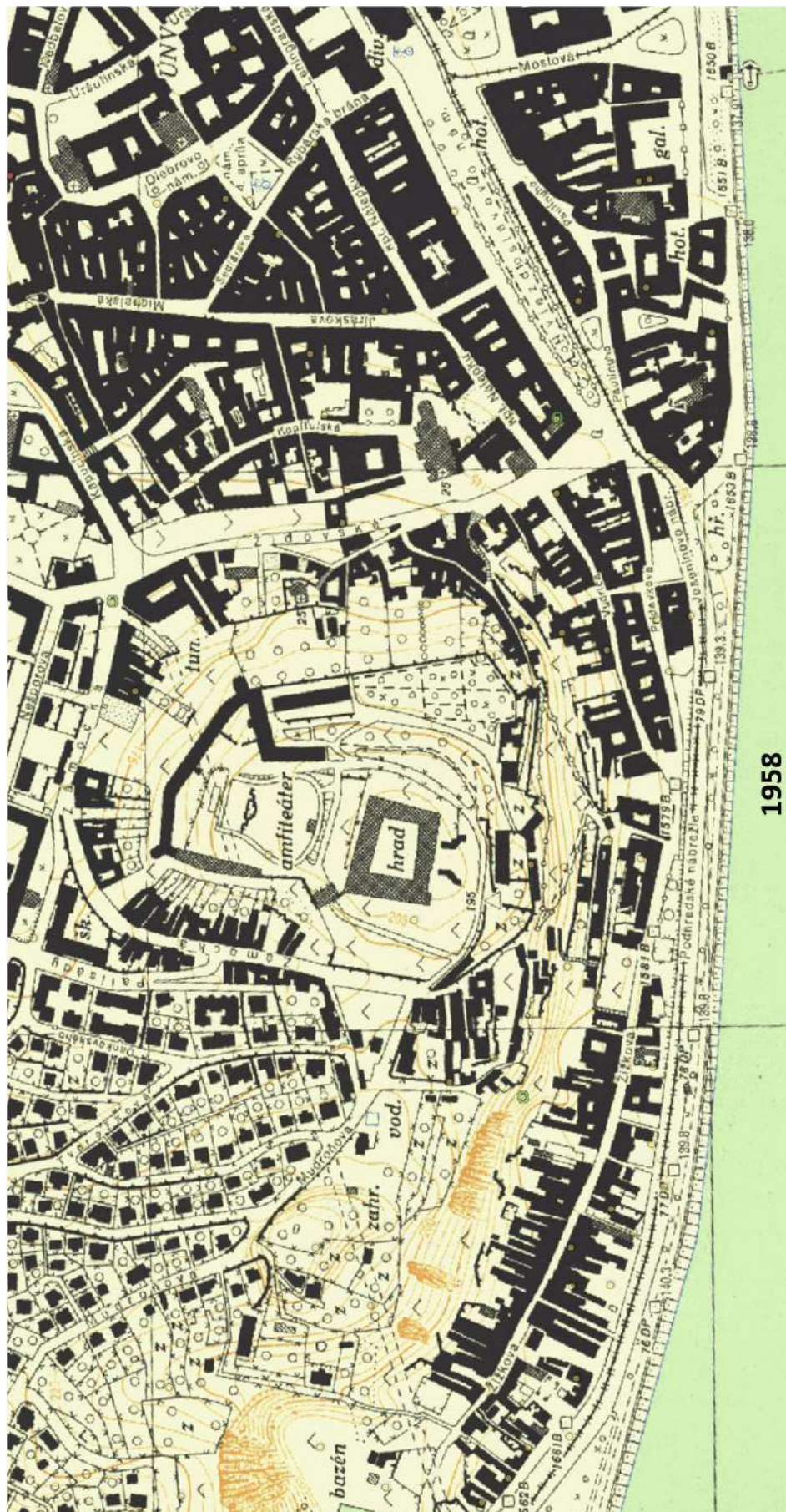


Fig. 3. Bratislava. Topographic map 1958. [1].

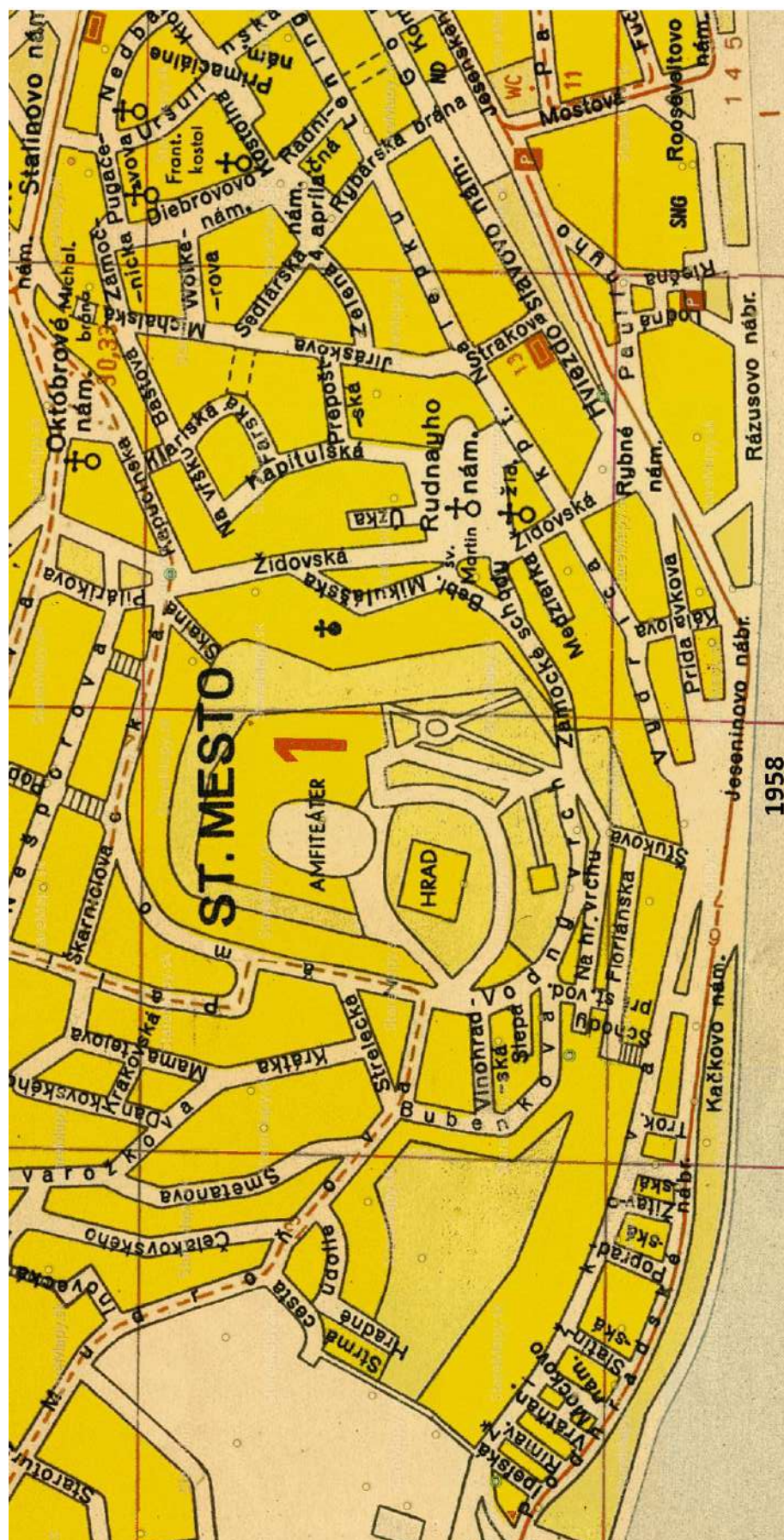


Fig. 4. Bratislava. Map 1958. [1].

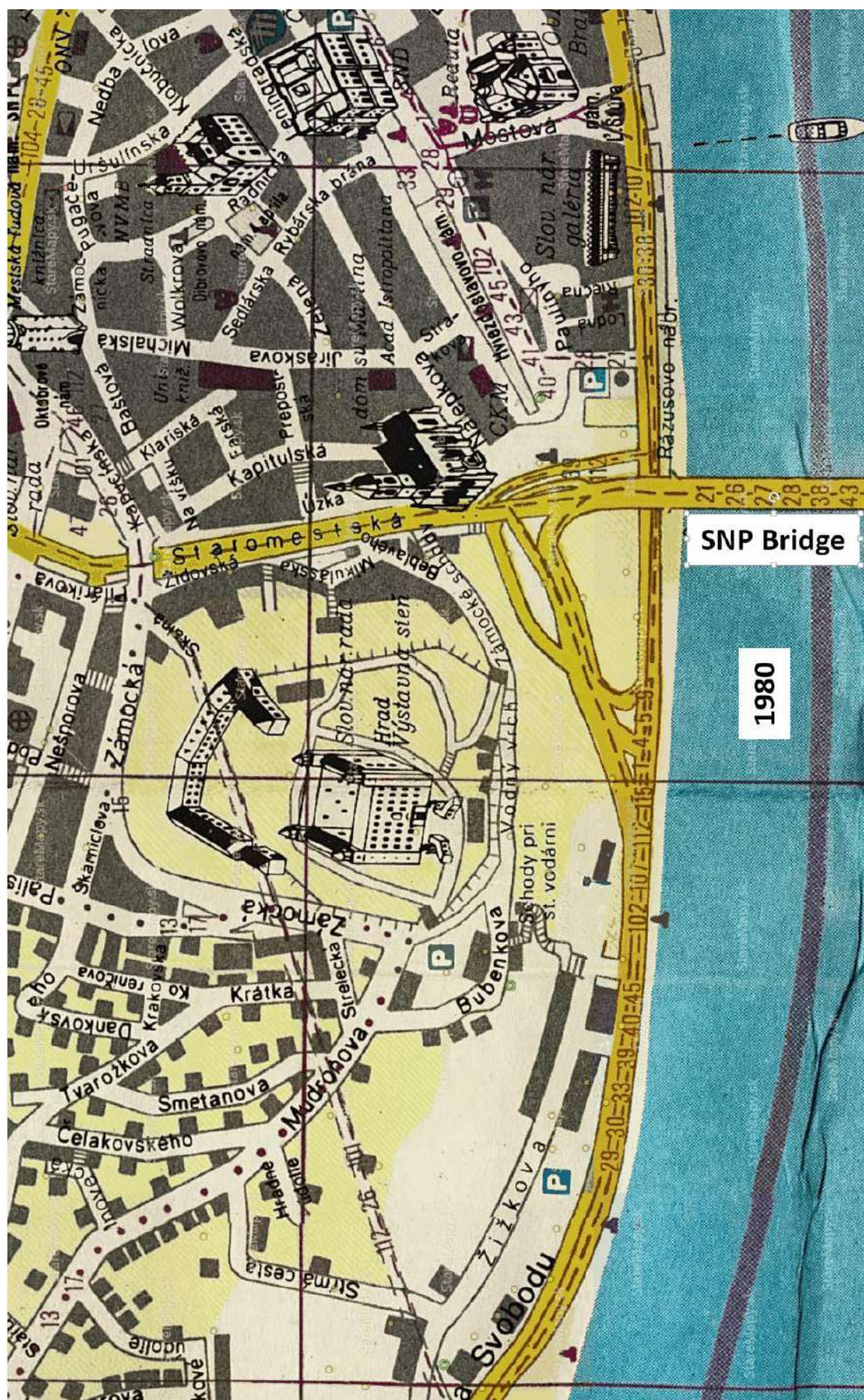


Fig. 5. Bratislava. Map 1958. [1].



a)



b)

Fig. 6. Embankment under the Castle (Podhradské nábrežie). About 1960: a) Münzhaus (monetarium) in the background; b) Münzhaus. [2].



a)



b)

Fig. 7. Embankment under the Castle (Podhradské nábrežie): a) Church of the Holy Trinity (Kostolík Sv. Trojice) built in 1734 by bishop Jozef Berényi. Trokanova [former Vitézova, Korabinského] street is on the left. About 1960; b) Water tower (Vodná veža) was found here on Pike (Šťuková) street. Sixties. [2].



a)



b)

Fig. 8. a) Embankment under the Castle (Podhradské nábrežie). Former inn At Black Bear. About 1970. [2]. b) Jesenin embankment (Jeseninovo nábrežie) No.2. View from the Danube river. (Photo Igor Scholtz).



a)



b)

Fig. 9. Jesenin embankment (Jeseninovo nábrežie) No.16, 14 and 12. View from the Danube river. Kálal street on the right. 1965. [2]; b) No.12. (Photo Igor Scholtz).



a)



b)

Fig. 10. Jesenin embankment (Jeseninovo nábrežie). View from the Danube river:
a) No.16; b) No.16. (Photo Igor Scholtz).



a)



b)

Fig. 11. Rázus embankment (Rázusovo nábrežie). Buildings prepared for demolition:
a) 31.3.1968; b) 1.4.1968. (Photo Juraj Bončo).



a)



b)

Fig. 12. West end of Žiškova street (Žiškova): a) House of F. X. Messerschmidt in the middle. After 1969; b) House of F. X. Messerschmidt No.49. Between houses there is Ipeľská [former Messerschmidt] street. The street on the right is Mlynárska street, where there were only two houses [2]. F. X. Messerschmidt bought building of the former inn At Jeleň (At Deer) in 1780 and lived in this house. A gravedigger lived in Mlynárska street.

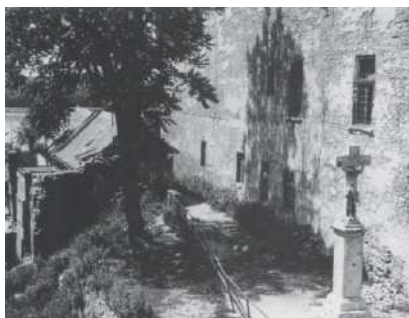


a)



b)

Fig. 13. a) Ipeľská [former Messerschmidt] street. View from the Danube river. About 1960; b) Žiška (Žiškova) street No.43. 1965. [2].



a)



b)

Fig. 14. a) Braunecker street. View from Žiška (Žiškova) street. About 1960;
b) Žiška street. The entrance to Braunecker street on the left. Fifties. [2].



a)



b)

Fig. 15. Žiška (Žiškova) street: a) The entrance to Braunecker street on the left. 1955;
b) The Kamper mansion (Žiška street No.12) in the foreground, the Brämmer mansion (Žiška street No.18) in the background. About 1970. [2].



a)



b)

Fig. 16. Žiška street (Žiškova): a) The Kamper mansion (Žiška street No.12) in the foreground, the Brämmer mansion (Žiška street No.18) in the background. 1961; b)
Former Zuckermandel City Hall. The Brämmer mansion in the background. [2].



a)



b)

Fig. 17. Žiška (Žiškova) street: a) Kamper mansion No.12. 1965;
b) Inn on Žiška street No.1. About 1960. [2].



a)



b)

Fig. 18. a) Žiška street No.60. 1930. Destroyed in 1961. [2];
b) West end of Prídavok (Prídavkova) street. Around 1962. [2].



a)



b)

Fig. 19. a) East end of Žiška (Žiškova) street. Beginning of Zuckermandel. King's arsenal from 16th century. 1969. Demolished in seventies. [2]; b) Zuckermandel. Arsenal. 1968. (Archive of Peter Horanský).



a)



b)

Fig. 20. Prídavok (Prídavkova) street No.38: a) View from Vydrlica street;
b) View from the Danube river. (Photo Igor Scholtz).



a)



b)

Fig. 21. Prídavok (Prídavkova) street prepared for demolition. From the left: a) No.38, 36, 34, 32, 30, 28, 26. About 1970; b) No.34, 32, 30. 31.3.1968. (Photo Juraj Bončo).



a)



b)

Fig. 22. Prídavok (Prídavkova) street. View from the Danube river:
a) From the left No.34; b) No.32. (Photo Igor Scholtz).



a)



b)

Fig. 23. Prídavok (Prídavkova) street. View from the Danube:
a) From the left No.30, 28 and 26; b) From the left No.28 and 26. (Photo Igor Scholtz).



a)



b)

Fig. 24. Prídavok (Prídavkova) street. View from the Danube:
a) From the left No.20 and 18; b) No.18. (Photo Igor Scholtz).



a)



b)

Fig. 25. Prídavok (Prídavkova) street. No.16: a) View from the Danube river; b) View to Kálal (Kálalova) street. (Photo Igor Scholtz).



a)



b)

Fig. 26. Prídavok (Prídavkova) street No.16.: a) View to Kálal (Kálalova) street. 1965; b) The corner of Prídavok and Kálal street. 1964. [2].



a)



b)

Fig. 27. Prídavok (Prídavkova) street. View from the Danube. From the left: a) No.18, 16 and 14. After 1965; b) No.16, 14 and 12. About 1960. [2].



a)



b)

Fig. 28. Prídavok (Prídavkova) street: a) From the left No.14, 12 and 10. View from the Danube river; b) No.15. (Photo Igor Scholtz).



a)



b)

Fig. 29. a) and b). Prídavok (Prídavkova) street. No.11. (Photo Igor Scholtz).



a)



b)

Fig. 30. a) Prídavok (Prídavkova) street; b) Stairs at old water station (Schody pri starej vodárni). About 1965. [2].



a)



b)

Fig. 31. Florian (Floriánska) street. 1955: a) No.14. Stairs at old water station (Schody pri starej vodárni) on the right; b) View from Pike (Šťuková street). [2]. Floriánska street was famous in the past for its inns and whorehouses.



a)



b)

Fig. 32. a) Florian (Floriánska) street. In the background there are houses on Vydrica street. 1955; b) View from Oeser (Oeserova) street to Pike (Šťuková) street. Archive of Peter Horanský. 1961. [2].



a)



b)

Fig. 33. Pike (Šťuková) street: a) Sixties; b) No.2. 1961. [2].



a)



b)

Fig. 34. a) Pike (Šťuková) street. About 1960; b) Water tower (Vodná veža) was found here on Pike (Šťuková) street. Sixties. [2].



a)



b)

Fig. 35. a) West end of Vydrlica street. View from the Danube river. Sixties; b) Braunecker mansion on Vydrlica street No.42, so called Kettenhaus (Chainhouse). About 1960. [2].



a)



b)

Fig. 36. Vydrlica street: a) View from the Danube river. About 1968; b) View in direction to the Danube river. Sixties. [2].



a)



b)

Fig. 37. Vydrlica street: a) End of Vydrlica street. Last buildings prepared for demolition. [2]; b) No.50. (Photo Igor Scholtz).



a)



b)

Fig. 38. a) Prídavok (Prídavkova) street No.36 and Vydrlica street No.37;
b) Vydrlica street No.25. View from Prídavok street. (Photo Igor Scholtz).



a)



b)

Fig. 39. Vydrlica street. From the left: a) No.19, 21 and 23. 1962. [2]; b) No.17, 19 and 21.
View from the Castle. (Photo Igor Scholtz).



a)



b)

Fig. 40. Vydrlica street. From the left:
a) No.15, 17 and 19. [2]; b) No.11 and 13. (Photo Igor Scholtz).



a)



b)

Fig. 41. View from the Castle. Vydrlica street. From the left:
a) No.11 and 13; b) No.5 and 7. (Photo Igor Scholtz).



a)



b)

Fig. 42. View from Synagogue on Fish (Rybné square). Vydrlica street. From the left: a) No.1, 3, 5, etc. About 1960. [2]; b) No.5, 7, etc. 11.10.1967. (Photo Juraj Bončo).



a)



b)

Fig. 43. a) Vydrlica street. From the left: No.1, 3 and 5. View from Vydrlica street to Synagogue. [2]; b) View on Rybné square and Vydrlica street. From the left: No.5 and 7. All buildings except two next to Synagogue were demolished. 1.4.1968. (Photo Juraj Bončo).



a)



b)

Fig. 44. View from the Castle: a) Rybné square. In the background the brewery just before demolition. 6.4.1969; b) The same view in 3.4.2005. (Photo Juraj Bončo).



a)

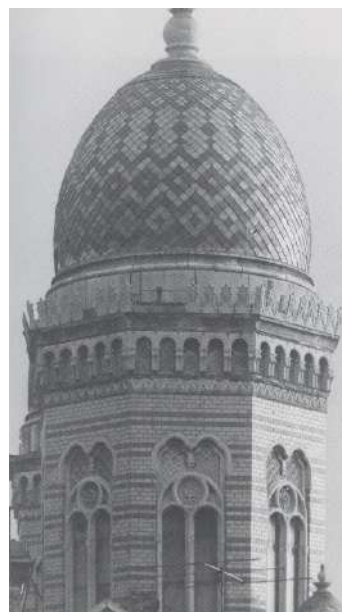


b)

Fig. 45. a) Demolition of Fish (Rybné square). Reconstruction of St. Martin's Cathedral. [2]; b) Fish (Rybné square) No.6 before demolition. (Photo Igor Scholtz).



a)



b)

Fig. 46. Synagogue on Fish (Rybné square): a) Beginning of demolition. 6.4.1969. (Photo Juraj Bončo); b) Detail of Synagogue cupola. [2].



a)



b)

Fig. 47. View from the Danube river to Fish (Rybné) square: a) Synagogue served as TV studio. In the background the House at Good Sheperd. 25.11.1968; b) Synagogue demolition. 13.4.1969. (Photo Juraj Bončo).



a)



b)

Fig. 48. Fish (Rybné) square: a) No.9 after demolition. 1.11.1968; b) There is the House at Good Shepherd far in the left. (Photo Juraj Bončo).



a)



b)

Fig. 49. a) and b): Hviezdoslav square No.23. View from the yard. (Photo Igor Scholtz).



a)



b)

Fig. 50. a) and b): Hviezdoslav square No.23. View from the yard. (Photo Igor Scholtz).



a)



b)

Fig. 51. Hviezdoslav square: a) No.23. Near Hotel Devín after demolition; b) No. ? (Photo Igor Scholtz).



a)



b)

Fig. 52. Fish (Rybné) square: a) From the left: No.8 and 9; b) No.9. View from Hviezdoslav square. (Photo Igor Scholtz).



a)



b)

Fig. 53. a) Conservatory. View from Hviezdoslav square; b) Conservatory. (Photo Igor Scholtz).



a)



b)

Fig. 54. a) Fish (Rybné square). View in direction to the Danube river. Conservatory on the left. Rybné square No.9. on the right. 1965; b) Zuckermandel. Sixties. [2].



a)



b)

Fig. 55. a) Nálepka (Nálepkova) street No.30. Synagogue in the background. (Photo Igor Scholtz); b) Mikulášska street (on the left) and Židovská street (on the right). The House at Good Shepherd is in the middle. View from St. Martin's Cathedral. The end of the sixties.



a)



b)

Fig. 56 a) Rudnay square (Rudnayovo nábrežie) No.7; b) Rudnay square No.7. View from the yard. (Photo Igor Scholtz).



a)



b)

Fig. 57. Mikuláš (Mikulášska) street: a) No.15 and 17. Sixties;
b) Church of St. Nicholas (Kostolík Sv. Mikuláša). Seventies.
(Photo Mikuláš Gažo. Private collection of Prof. Juraj Šedivý on PamMap.sk [4]).



a)



b)

Fig. 58. The building on the left, the Church of St. Nicholas (Kostolík Sv. Mikuláša) and the House at Good Shepherd were not demolished. Jewish (Židovská) street was on the right. It completely disappeared. 6.4.1969; b) The same view. Mikulášska street No.3 and 5 on the left. 22.05.2002. (Photo Juraj Bončo).



a)



b)

Fig. 59. Jewish (Židovská street): a) No.20; b) On the left. (Photo Igor Scholtz).



a)



b)

Fig. 60. Rest of Jewish (Židovská street): a) On the left; b) On the right. (Photo Igor Scholtz).



a)



b)

Fig. 61. a) Houses on Pilárik (Pilárikova) street prepared for demolition;
b) Petržalka. North (Severná) street No.1. (Photo Igor Scholtz).



a)



b)

Fig. 62. Petržalka. a) and b) Demolition of the shipyard Kablo. (Photo Igor Scholtz).



a)



b)

Fig. 63. Petržalka. Makarenko (Makarenkova) street: a) No.19; b) No.28.
(Photo Igor Scholtz).



a)



b)

Fig. 64. Petržalka. Railway track of youth street (Ulica Trate mládeže). From the left:
a) No.1, 2 and 3; b) No.6 and 8. (Photo Igor Scholtz).



a)



b)

Fig. 65. Petržalka. Platan (Platanova) street: a) No.38; b) No.40. (Photo Igor Scholtz).



a)



b)

Fig. 66. Petržalka. Platan (Platanova) street: a) No.42; b) No.46. (Photo Igor Scholtz).

List of the persons according to which embankments, streets and squares are named:

Braunecker von, Anton, baron, imperial-royal court councillor. He built his palace according to architect Mühler in 1775.

Brämmer František. Castle officer at Pálffy family from 17th century. Brämmer mansion (Brämmerova kúria).

Bubenko Tomáš (17th century). Painter and wood engraver. Bubenko street (Bubenkova ulica).

Floriánska street. Florian street (Floriánska ulica). Named according to statue of St. Florian.

Hviezdoslav Pavol Országh (1849-1921). Slovak poet, dramatist, translator. Hviezdoslav square (Hviezdoslavovo námestie).

Ipel'ská street. Ipeľ street. (Ipeľská ulica). Named according to Ipeľ river. Left tributary of the Danube river.

Jesenin Sergej Alexandrovič (1895-1925). Russian poet. Jesenin embankment (Jeseninovo nábrežie).

Jiringer Joseph (1809-1862). Bratislava businessman and philanthropist. Jiringer street (Jiringerova ulica).

Kálal Karel (1860-1930). Czech teacher and writer. Kálal street (Kálalova ulica [Kunigsfelderova]).

Kamper František. Austrian nobleman from 16th century. Kamper mansion (Kamperova kúria).

Korabinský Johann Matthias (1740-1811). Native of Bratislava, teacher and writer. Korabinský street (Korabinského ulica).

Makarenko Anton Semjonovič (1888-1939). Ukrainian, social worker and writer. He is often reckoned among the world's great educators, and his books have appeared in many countries. Makarenko street (Makarenkova ulica).

Messerschmidt Franz Xaver (1736-1783). German-Austrian sculptor most famous for his "character heads" (Fig. 67). Messerschmidt street (Messerschmidtova ulica).

Mikuláš street (Mikulášska ulica). Name according to extinct settlement named according to St. Nicholas (Sv. Mikuláš).

Mlynárska street. Mill street (Mlynárska ulica). Named according to water mills on the Danube river.

Nálepka Ján (1912-1945). Slovak captain who organized and led an anti-fascist Slovak partisan detachment in the Soviet Union during World War II. Nálepka street (Nálepkova ulica).

Oeser Adam Friedrich (1717-1799). German etcher, painter and sculptor. Oeser street (Oeserova ulica).

Pilárik Štefan 1616-1678). Slovak writer. Pilárikova street. Pilarik street (Piláriková ulica).

Platan street (Platanova ulica). Street named according to plane (platan) tree.

Prídavok Anton (1904-1945). Slovak poet. Prídavok street (Prídavkova [Jiringerova] ulica).

Rázus Martin (1888-1937). Slovak poet, dramatist, writer, politician and Lutheran priest. Rázus embankment (Rázusovo nábrežie).

Rudnay Alexander Štefan (1760-1831). Hungarian–Slovak Roman Catholic prelate. He started as a parish priest, but later he became the Archbishop of Esztergom, the Primate of Hungary and a Cardinal. Rudnay square (Rudnayovo námestie).

Rybné square. Fish square (Rybné námestie). Named according to fish market (Forum Piscium).

St. Martin's Cathedral. It is the largest and one of the oldest churches in Bratislava, known especially for being the coronation church of the Kingdom of Hungary between 1563 and 1830. In 1760, the top of the Gothic tower was struck by lightning and later replaced by a Baroque one, which was subsequently destroyed by fire in 1835 and reconstructed in 1847 (with some modifications), and topped by the crown of St Stephen. The church attained its present-day appearance during the period 1869 to 1877, when it was re-Gothicised after suffering damage by fire, war, earthquake and other disasters.

Synagogue. The synagogue of the Neolog Jewish community was demolished in 1969 to make way for construction of the SNP Bridge. The two-story Moorish building with two octagonal onion-domed towers was built in 1893 based on the designs of Dezső Milch. The Neolog community was established in 1871 by a secessionist group unhappy with the strong Orthodox leadership of the (mother) Jewish community. The synagogue was constructed on a square adjacent to St. Martin's Cathedral, and was a major landmark often depicted on postcards. The building served as a TV studio during the post-war period.

Šťuková street. Pike street (Šťuková ulica). Named according to inn Blue Pike.

Railway track of youth street (Ulica Trate mládeže [former Masarykova]).

Trokan Ján (1857-1894). Slovak writer. Trokan street (Trokanova [former Korabinský] street).

Vydrlica. Street named according to animal otter.

Water Tower (Vodná veža). The Water Tower is part of Vydrlica, an ancient settlement that used to sprawl below the Castle Hill for centuries. The Water Tower built at the ford

served as a toll station on the international trade route and as a western fortification, important for the defence of the city and the Castle.

Zuckermandel (Podhradie). Podhradie (Latin: Suburbium, German: Schlossgrund and/or Zuckermantel/Zuckermandel, Hungarian: Pozsony-Várallya) has been a district of Bratislava since 1848, previously a separate residence, which around 1800 spread east of the Castle (between the Castle and the city), south from the Castle (between the Castle and the Danube) and partly west of the Castle (to Mlynská dolina).

Židovská street. Jewish street (Židovská ulica).

Žiška Ján z Trocnova (1360-1424). Bohemian military leader during the Hussite Wars. Žiška street (Žiškova ulica). The former names: [1860 Zuckermantl Hauptgasse, 1879 Maria Theresa street, 1921 Podhradská street, in period the 1939-1945 Hermann Göring street].

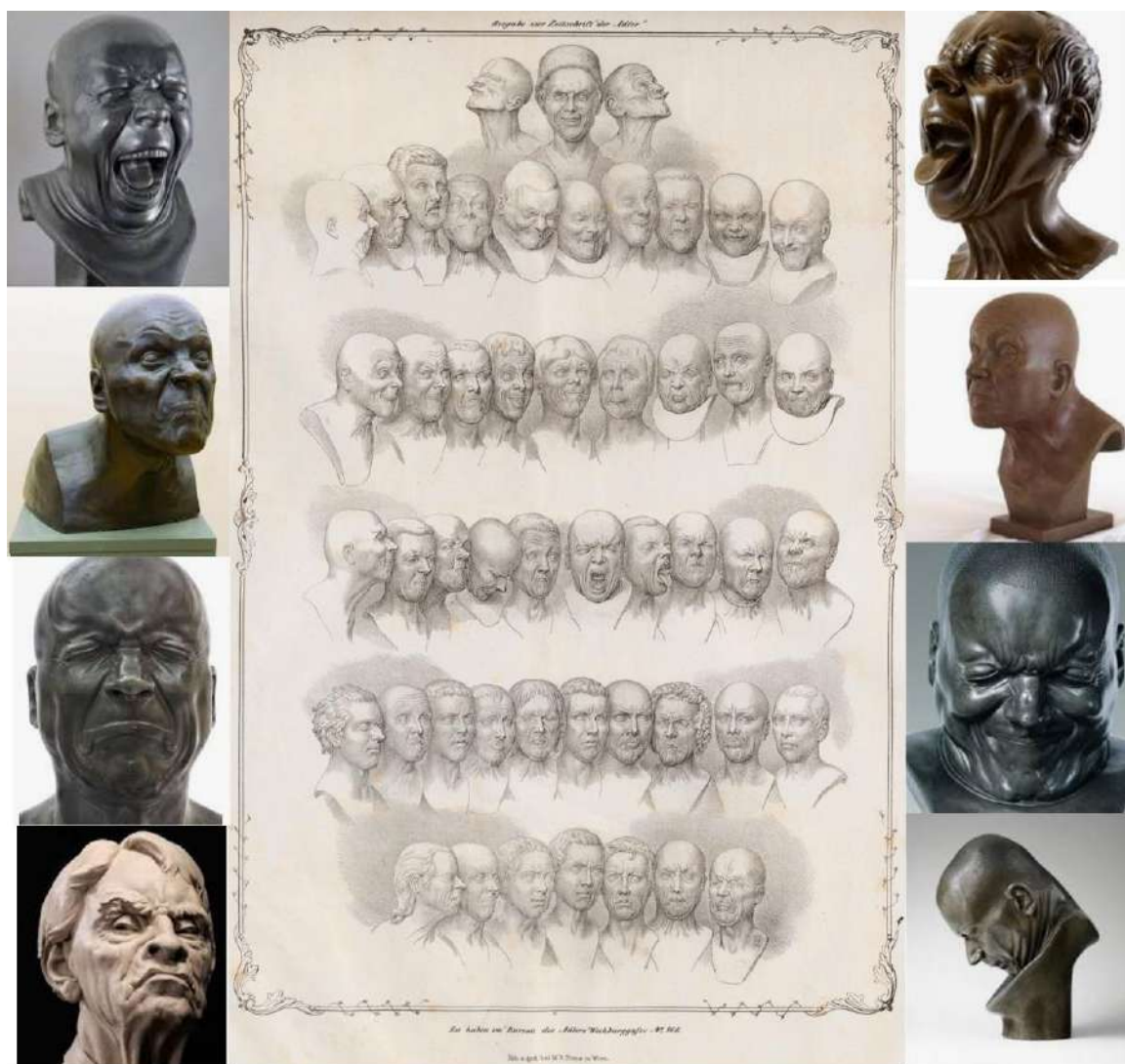


Fig. 67. Some of the “character heads” created by sculptor Franz Xaver Messerschmidt. Expressive intensity of distorted faces places them amongst masterpieces of 18th century sculptures.

3. Conclusions

The paper presents collection of photographs taken mostly in the sixties, some of them also in the fifties. They show buildings demolished during the construction of the SNP Bridge. The location of the SNP Bridge, crossing the old part of Bratislava, and the consequent demolition of old historical buildings around Castle Hill is considered by some as controversial. In fact, most of demolished buildings were in a poor state due to lack of money for their reconstruction.

Table 1 Timetable of buildings removal according to plan published in SMK 28.2.1967.
The list of buildings was later continuously updated.

Month, year	Locality (street, square, embankment, house number)	Number of houses	Number of flats
1966	Židovská street 20	1	5
Till March 1967	Lodná street 12	1	9
	Hviezdoslavovo square 23	1	50
	Rázusovo embankment 12, 18	2	45
	Rázusovo embankment 20	1	8
	Rybné square 4, 5, 6	3	42
	Petržalka, shipyard, drugstore, ZARES	3	8
Till May 1967	Jeseninovo embankment 2 (conservatory)	1	-
	Jeseninovo embankment 4, 6	2	22
	Prídavkova street 4, 10	2	10
Till August 1967	Prídavkova street 16	1	10
	Vydrica 5, 6, 7, 11, 12, 13, 15, 17	8	58
	Rybné square 2 (Synagogue)	1	-
	Rybné square 8, 9, 10, [6]	3	18
	Nálepková street [30]	1	10
	Rudnayho street 7	1	5
	Petržalka, Platanova 9, 33, 38, 40, 42, 46	6	21
Till X. 1967	Rázusovo embankment 14, 16	2	29
Till February 1968	Jeseninovo embankment 12, 16, [2, 14]	2	10
	Prídavkova street 11, 15, 18, 20, 26, 28, 30, 32, 34, 37 [12, 14, 38]	10	60
	Vydrica 18, 21, 25, 27, 29, 31, 33, 35 [1, 3, 19, 21, 23, 36, 37, 42, 50]	8	50
	Petržalka, Trať mládeže street 1, 2, 3, 4, 6, 8, 10	7	43
Till March 1968	Petržalka, Sofijská street 11, 16, 26, 30, 30, 32	5	12
	Petržalka, Severná street 1	1	2
	Petržalka, Zväzácka street 11, 17	2	6
	Petržalka, Makarenkova street 19, 25, 28	3	6
	Petržalka, Lysenkova street 1, 3	2	2
	Petržalka, Uralská 6	1	2
Till June 1968	Proboštská street 6	1	1
	Kapucínska street 2, 4	2	11
	Októbrové square 10	1	5
	Pilárikova street 1, 4, 6	3	25
	Šmeralova street 7	1	3
Till Dezember 1968	Veterná street 13, 15, 18	3	18
	Konventná street 8, 10, 13	3	22
	Suché Mýto square 9, 11, 13, 15	4	55
	Palisády street 64	1	4
	Rybné square 3, 10	1	1
	Hviezdoslavovo square 23, 25, 27	3	28
Till June 1969	Vysoká street 5, 7, 9, 13	4	33
	Suché Mýto square	2	11
Total	Bratislava and Petržalka side	112	770

Note: **Bold** face means that there is a photo of the building in this paper. *[Italic]* indicates that the building on the photo was not in the list of buildings published in SMK 28.2.1967.

The paper contains photos of buildings in: **Žižka street [1, 12, 18, 43, 60]** and **Mikuláš street [15, 17]**.

Altogether 112 buildings (1 in 1967, 40 in 1967, 65 in 1986, 6 in 1969), containing 770 flats including 100 flats in Petržalka (5 in 1967, 345 in 1967, 376 in 1968, 44 in 1969) with approximately 2 500 inhabitants were in the list of buildings published in SMK 28.2.1967. The inhabitants from demolished buildings were rehoused to the new modern flats.

The authors attempted to reconstruct the sequence of buildings in streets, squares and embankments that no longer exists according to photographs taken from [2, 3, 4]. This detective work worthy of Sherlock Holmes was possible in many cases thanks to known house numbers and to comparing details in photographs.

The maps on Figures 3, 4 and 5 enable to locate the demolished buildings in the relevant streets, squares and embankments.

4. Authors

As teachers of Slovak and foreign students at Faculty of Civil Engineering STU in Bratislava inform students about important historical structures and their designers. Students are interested also in other actual topics like those treated in [5 - 8].

Acknowledgement

This research Project No. 1/0453/20 is supported by the Slovak Grant Agency VEGA. Authors would like to thank to the authors of photographs: Mr. Juraj Bončo [2], Ing. Igor Scholtz [3] and to Prof. Juraj Šedivý from Comenius University in Bratislava, who provided two photographs (Fig. 57a, and b) from his collection, which may be seen also on the PamMap portal [4].

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STATE OF ART OF TECHNOLOGIES FOR SAFEGUARDING HISTORIC STRUCTURES IN BULGARIA

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Abstract: *In this paper new and traditional technologies for analysis, restoration, repair, maintenance and strengthening of historic structures in Bulgaria are presented. It is emphasized on the masonry structures as churches, mosques, synagogues, monasteries, tombs, towers, belfries, minarets, etc. A short review of the valuable Bulgarian architectural, historic and cultural is shown. The general peculiarities of these structures are discussed. The models and methods for their analysis are regarded and compared. The seismic response and analysis are drawn. Basic methods and techniques applied for safeguarding and renovation of such structures are described. Important examples of some typical representative historic monuments from different historic epochs are given. Experience, state and problems in this domain are outlined. Conclusions, recommendations and proposals for study, repair, exhibiting, maintenance and strengthening of historic structures are derived.*

Key words: *restoration , repair, maintenance, masonry structures*

1. INTRODUCTION

The cultural heritage of Bulgaria is exclusively rich and diverse. It reflects our national history. Some of these monuments have world meaning and they are under the aegis of UNESCO. Our duty is to safeguarding and show this valuable richness.

Basic definitions and classifications: in this domain should be taken into account. Following the World Heritage Committee in conformity with the World Heritage Convention it has developed a system of operational guidelines which can be maintained and revised as necessary. The following three categories are specified as world cultural heritage as well:

Monuments: Architectural works and works of monumental sculptures are included in this category. Also, structures that have archeological nature and cave dwellings should be considered in this class. Works that have outstanding value from the viewpoint of the history, art and science are specified to be considered as monuments too. Particularly for Bulgaria, this class is represented by a number of old churches in Sofia, Svishtov, Veliko Turnovo, Nessebar, Roman theater in Plovdiv, etc.

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Groups of buildings: These are groups of separate or connected buildings which, because of their architecture, homogeneity or place in the landscape, are of outstanding universal value from the point of view of history, art or science. This class is represented by a number of houses typical for the period of the Bulgarian Renaissance in the town in Plovdiv, Koprivshtitza, Veliko Turnovo, Melnik, etc. These houses are related to the Bulgarian traditions and culture.

Sites: These are works of man or the combined works of nature and of man, and areas including archeological sites which are of outstanding universal value from the historical, aesthetic, ethnological or anthropological point of view. From the viewpoint of the civil engineering the first two categories are the most essential for repairing, strengthening and rehabilitation. For the sake of simplicity in the further considerations this two categories will be called shortly “ancient structures” or “historic structures”.

The historic epochs were made their influences by the characteristic religious structures. Peoples of different ethnic were lived at the Bulgarian territory. They were left their effect on the temples and other monuments connected with the religious rituals. The historic processes effect also strong to the church construction. There are magnificent paragons of this imposing group of historic structures dated from different historic periods. Some of these masterpieces were built by genius past masters and they are real sacred monuments. They bring distinguishing marks of their times. First of all they depend on the canonical function of these structures and the state of the nation (scientific knowledge, technological level, financial possibilities, state management, etc.) during the period of their construction. The religious canonical requirements are very strong and therefore all human interventions to the historic structures should be conformable to them

The engineering interventions should be realized only when all reserves of the historic structures are exhausted. These interventions should be minimal and appropriate. Usually these activities must keep the external and internal architecture of the visible parts of the unique historic structures. Strong control on the human activities in this domain is need to safeguarding the valuable cultural heritage. In view of these numerous and strong limitations and requirements the restoration and preservation works including engineering works meet many difficulties. The multi and inter disciplinary team is need for restoration and preservation works. It should includes investigators, specialists, designers, executors, investors, users, etc. Architects, geologists, civil, heating, electrical, hydraulic engineers, painters, chemists, etc. are members of these group. The activity of this body should be subordinated to the following factors: information for the state of the cultural monuments, methods for the management of information, the process of intervention in the monument, the human resources and the standard base. The second factor is especially important. The responsibility of these professionals is enormous. Their works of high quality and their professionalism are compulsory in the name of all human society. Violations, mistakes and lapses are inadmissible. Three main subsequent steps should be carried out in the process of safeguarding of the cultural monuments: monitoring, analysis and restoration and preservation. The monitoring is an expensive activity and it is applied rare in Bulgaria. But experts make inspections on these monuments. Each monument should be provided with a “passport” in which all changes, interventions and other data should be recorded. It is established National Institute for the Monuments of the Culture. Its specialists study the state of these structures, manage and inspect their restoration and preservation. There are some national licensed laboratories for testing of construction and soil materials.

2. GENERAL FEATURES OF HISTORIC STRUCTURES AS GUIDELINES FOR REHABILITATION

Stones, bricks, wood and mortar were typical construction materials in old times. Stones and bricks were ordinary made by special technologies. For example the ancient Roman masters were used special clay for producing bricks. The clay were washed out by a large quantity of water. Then the clay were matured for some time. After that the bricks were fired into special furnaces in a high-temperature regime. This production process was continued about one year. The stone blocks were left in natural conditions for a long time and the cracked blocks were not used. The stones were cut out or knocking out very carefully. Often travertine and limestone are used. The mortar was mixed with river felt and brick small pieces. Therefore the joints in ancient and old masonry are about 4-5 cm thick. In the cases where the masonry is not wear-out the up-to-date tests sometimes determine significant higher strengths of these construction materials in comparison with the present-day ones. Usually the historic structures have shallow embedded stone masonry strip foundations or column footings, large span and high premises, complex geometrical forms, underground basements, massive masonry walls minimum 60 cm thick, internal rows of masonry columns, vaults, domes, arches without or with wooden or wrought iron tension members in one or two rows and in one or two directions, roofs of different levels, wooden balconies, frescos, etc. Often the structure above the cornice level is with equal ceiling on trim joists at one or different levels. Usually one vertical longitudinal plane of symmetry exists. The stone masonry is mostly implemented. The cultural heritage as churches, historical buildings and monuments have specific features and deserves special attention when their civil engineering parameters are to be improved. In a number of cases conventional methods for repairing and strengthening can not be applied in ancient structures. It is worth mentioning the following features that are common for large classes of ancient structures:

1. The construction technologies used in ancient structures are quite different from the up-to-date technologies.
2. Considering stone and brick masonry structures, their lateral stiffness is mainly influenced by the gravity effects and adhesion between separate blocks/bricks.
3. The loads that are considered within the analysis are mainly vertical (dead and live loads). However for seismic active areas like Bulgaria where seismic risk is potentially large, seismic type of loading appears to be quite unfavorable.
4. Dynamic parameters of ancient structures are not favorable from the viewpoint of the aseismic design. For example the fundamental period of such type of structures ranges between 0.10 – 0.30 s, i.e. they are very rigid. The consequence is that such structures are much sensitive to pulse type of loading. Stone and brick masonry both have large specific mass.
5. Ancient structures are not ductile and their failure modes show tendencies of brittle fracture. Current design philosophy based on the reduction of seismic forces as because of inelastic behavior can not be applied.
6. Repairing and strengthening of the ancient structures should be so performed as to keep architectural style and some other requirements that are to be satisfied from canonical point of view.
7. Traditional methods for modeling, calculation and design are not directly applicable to the analysis. For example, the well known finite element method needs additional upgrade in order to obtain more adequate solutions. The idea for development of problem-oriented software is very fruitful for the purposes of ancient structures analysis.

8. In contrast with the conventional buildings in ancient structures no damages are allowed. The prevention from earthquakes implies lack of damages.

These structures were well predicted to sustain only or predominantly basic vertical loads as self weight, useful and snow loads. Possible damages caused by these loads are the following [14]: wear-out of the stones and mortar, putrefaction and decrease the bearing capacity of the wooden material. As a result deflections, geometrical changes, shear of the cross section, crease of the supporting parts occur and can be attended with carrying away of the above structure. But the historic structures are also subjected to horizontal reversal loads caused first of all by earthquakes and wind loads too. The wind loads influence slightly on religious buildings but strong on some high-rise structures as minarets, towers, belfries. In almost all cases the historic religious structures are not ensured to withstand the special horizontal loads originated from strong earthquakes.

Almost all Bulgarian territory fall into a seismic active zone. A large part of the rich Bulgarian culture and historic heritage was effected by the numerous past strong earthquakes. Many historic monuments were endured significant damages and failures caused first of all by catastrophic earthquakes and climatic effects. They have no half-storey plates to redistributed the horizontal loads. Their horizontal strength and the spatial interaction of the structural elements are insufficient or absent. They resist mainly by their self weight and work by shear predominantly. (Fig. 1). Their shear base coefficient at some level, i.e. above structure weight to a horizontal force ratio is up to 0,10, while for the usual buildings this ratio is between 0,20-0,40. That is mean the ancient masonry structures withstand horizontal loads up to 10 % large versus vertical loads. The masonry tensile strength is very little. They can not dissipate energy. Therefore they are quite sensitive to horizontal loads, tensile stresses in structural elements and non-uniform settlements of the subsoil. They are in danger to a brittle demolition, if their bearing capacity is exceeded (Fig. 2).

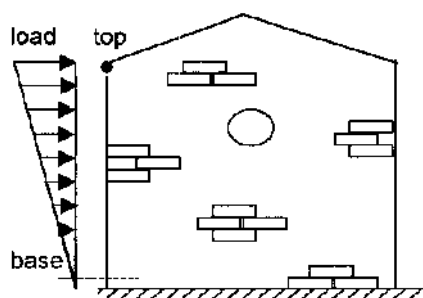
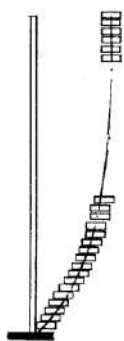


Fig. 1. Simplest model for church seismic response

Fig. 2. Base shear - top displacement relationship for ductile and brittle structures subjected to lateral monotonically increased static load

In addition during earthquakes specific crack meshes propagate (Figs. 3, 4), the available tension members get loose or they are pulled out, the internal columns are inclined, the supporting nodes of the trim joists and beams of the roof structure are damaged. Strains, crack propagation, deplanation and disembowelling of the masonry, failures, non-uniform subsoil settlements are the most spread damages in these structures. This insufficient horizontal loading capacity of the historic religious structures is always limiting for their state. Significant construction interventions are need to safeguarding them

according to current design codes and standards. During the catastrophic earthquake in 1858 in Sofia there were 24 mosques and 7 churches. But the minarets of 19 mosques were destroyed and large cracks were arisen. Only 2 churches were stayed fitted for use too.

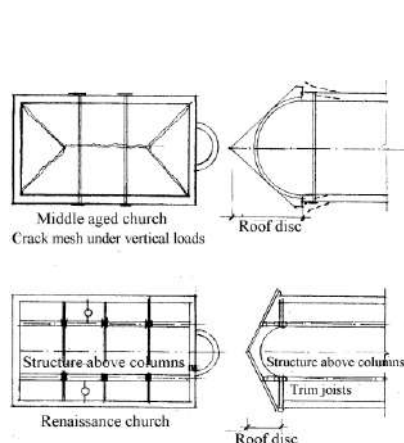


Fig. 3. Simplified schemes of churches and typical crack mesh under vertical loads

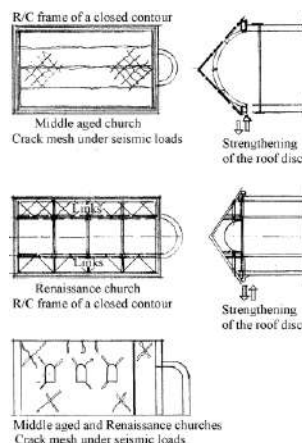


Fig. 4. Simplified schemes of churches, typical crack mesh under seismic actions and roof strengthening

Capillary humidity, aggressive effect of the candle smoke, the human temperature and humidity arising by the congregation and visitors are other external actions on these structures. These actions and especially the humidity effect also on such structures.

3. IMPROVING THE SEISMIC RESISTANCE OF HISTORIC STRUCTURES

As it is mentioned above, ancient structures are not ductile and are unable to dissipate energy. In contrast with the newly designed buildings their failure mechanisms are in a large extent related to brittle failure. The amount of inelastic strains that is expected in ancient structures is considerably small to ensure seismic loading reduction and seismic protection. Thus, it is implied that linear elastic behavior is still valid. After exceeding the elastic limits the capacity of the structure is soon reached and then the structure is collapsed. Figure 2 illustrates the principle of strengthening that is used as basic philosophy. A single wall structure is considered being subjected to monotonically increased lateral load. This type of analysis is often used to predict some of the properties of the system. The relationship base shear Q_{base} – top displacement, v_{top} , is selected as representative for structural behavior. Current design philosophy enables the structure to dissipate energy by making it ductile (see the plot of the line denoted by "ductile"). The elastic limit is slightly exceeded and inelastic behavior is initiated.

The plot of the next two curves in Fig. 2 is given for better understanding the difference that exists between ductile and brittle structures. The line denoted by "brittle 1" is representative for structure, whose capacity in shear is $Q_{base,1}$. Before reaching this limit structure will remain elastic. For a value of $Q_{base,2}$ slightly greater than $Q_{base,1}$ structure will be fractured. If this structure is to be improved, its strength should be increased rather its stiffness (see the curve, denoted by "brittle 2"). It means that the strength is increased reaching the value $Q_{base,2}$. The idea of strengthening is to increase structure strength using structural engineering approaches. The reasons for important structural defects in the historic structures can be summarized and generalized in three groups as follows:

- Mistakes during the construction, operation and repair because of insufficient knowledge of the construction science;
- Deformations, crack propagation and damages caused by the basic loads;
- Deformations, cracks and damages due to seismic actions;
- Environmental conditions as sun radiation, water, rain, humidity, frost, temperature changes, air pollution, etc.

4. ANALYSIS OF HISTORIC STRUCTURES

The antiseismic insurance of the historic religious structures is more difficult. Many problems arise. First of all the real available carrying capacity of the structure is to be carefully studied and evaluated. The physical-mechanical characteristics of the original construction materials (stones, bricks, mortar, wood) of the bearing elements, the soil and the properties of the whole examined structure should be studied carefully by field and laboratory tests using the experimental and analytical methods of the structural physics, soil mechanics and construction materials. Especially shear test of the masonry is need.

The tangential (shear) strength in horizontal direction versus the normal tensile and compressive stresses in vertical direction in conditions of different normal and tangential horizontal stresses are the most important characteristic diagrams of the masonry. Non-destructive or destructive experimental methods can be implemented. Full-scale dynamic and static experiments of the structure are need to determine the free vibration periods, distortions, displacements, inclinations, settlements. This test can be accomplished by special dynamic equipment. The displacements are measured by transducers located at appropriate places of the structure in plan and in height. The crack mesh propagation should be studied too. The current seismic design codes are not treated this type of special and complex structures. Adequate mechanical-mathematical models of the structure should be used in research. The effective up-to-date methods should be applied in dynamic and seismic analysis. First of all the finite element method (FEM) is most appropriate for structural analysis. Thick shell or 3D brick elements of higher order are adequate for modeling of walls, vaults and domes. Thick beam-column or 3D brick elements of higher order are suitable for idealization of the columns, foundations and arches. Special masonry models and elements can be utilized also. The shear are to be taken into account.

The cracks in masonry can be modelled in some ways: by finite elements with double unconnected nodes at the two sides of the crack, with orthotropic elements of different properties in parallel and orthogonal directions to the crack, using special finite elements provided with special properties as gap elements, etc. In the first case large stress concentration is obtained. In the second case the value of Young's modulus in the crack orthogonal direction can be assumed 5-10 times less than in parallel direction. In this case stress concentration is slightly expressed. All finite elements should be orthotropic since the value of Young's modulus in horizontal direction, i.e. parallel to the horizontal joints between the masonry rows is less than in vertical direction, i.e. orthogonal to these joints.

The old thin shell elements of lower order and plane initial geometry are too inaccurate. They are provided with 5 DOF - 3 displacements and 2 rotations - in each corner node. These elements lead to non-smooth diagrams of the internal moments at the nodes since the curvatures expressed by the second derivatives of the displacements are not included as basic unknowns (DOF) and there are no internal nodes at the element sides. Therefore fine meshes should be implemented. The new thick and thin shell finite elements of higher order with curved initial geometry and including shear are significantly more precise. They are assumed to provide with 6-9 DOF - 3 displacements, 2-3 rotations and 2-

3 curvatures - in each node and have internal nodes on the element sides and in the shell thickness. As a result smooth diagrams of the internal moments at the nodes are obtained.

Then rough meshes can be used. Results obtained by some models and methods should be compared. During strong earthquakes the historic structures should be kept in view of their importance. The modal superposition method and the response spectrum method can be used in linear seismic analysis. Two equal or correlated horizontal simultaneous spectra in parallel and orthogonal directions to the church main entrance can be used. Then the results will be on the safety side. A preliminary solution to a vertical seismic spectrum usually establish that for this type of structures the internal forces caused by a vertical seismic spectrum are usually less than 30 % to the internal forces originated by dead vertical loads. In these cases a vertical spectrum is not need according the design codes for masonry structures. Non-linear time history seismic analysis is recommended and rational for such unique and important structures. In such cases real characteristics of the construction materials including non-homogeneous or orthotropic masonry and adequate real or artificial accelerograms of the region and the site should be taken into account. As a rule two criteria for masonry failure can be applied. They cover displacement and force requirements respectively.

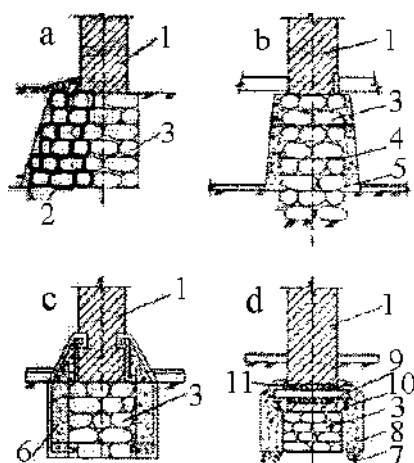
Powerful and multi-purpose computer programs as ANSYS, COSMOS, NASTRAN, ABACUS, SAP, TOWER, ALGOR, STAAD, etc. are desirable (need). The complex geometry can be previously idealized by the AUTOCAD, SOLID EDGE, COREL DRAW, PRO-ENGINEER, etc. programs. The structure are to be investigated numerically and/or experimentally before and after the strengthening. Comparisons and assessments are need. Moreover the structure must be ensured in all periods of its structural renovation.

5. METHODS AND TECHNOLOGIES FOR STRENGTHENING OF HISTORIC STRUCTURES

The restoration and preservation of historic structures is difficult, important and responsible problem. New effective and adequate methods, materials and technologies should be used to ensure their reliability. Experienced experts should be engaged. Restoration works are expensive too. They need to be executed of high quality. The strengthening can not increase the ductility of these structures but it can enlarge their bearing capacity (limit for brittle failure). Seismic isolation or vibration dampers are almost impossible in such structures. The research results and estimation serve to make a precise diagnostics and to accept right and optimal approach for the strengthening. Some methods for strengthening of the masonry foundations are shown in Fig. 5 [11].

The aim of this approach is to reach strong soil at deeper level and to decrease the stresses in the soil-foundation contact surface by enlargement of the foundations. In the case a) the dimensions of the foundation basic plane are spread by new stone masonry built-in in the old masonry making bond between them. To improve this connection steel anchors of diameters 12 to 16 mm and class A-III can be anchored in the horizontal joints at a distance 50-60 cm and cement-lime mortar can be used as a connection. In the case b) the strengthening of the existing masonry foundations is realized by execution of the concrete enclosures. Steel anchors can be driven in the horizontal joints of the masonry to withstand the horizontal forces arising in the vertical joints between the masonry and the concrete enclosure. In the case c) the strengthening is analogous to the case b) but the concrete enclosure is reinforced by transverse and longitudinal reinforcement. In the case d) for stronger overloading of the existing masonry foundations the strengthening is carried out using the concrete enclosures united by horizontal bearing steel beams of spacing 1,5-2,0 m. They distribute the concentrated loads in longitudinal direction between

longitudinal beams built-in into the concrete enclosure. The transverse beams cross via holes bored through the existing foundations. The holes are about 50 cm in dimensions and after the mounting of the cross beams they are filled up by concrete. The simplified diagram of the stresses at the subsoil before and after application of the additional loading is drawn in Fig. 6.

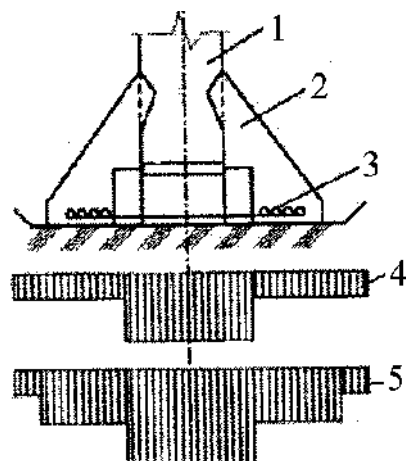


a - Enlarging of the masonry base by stone masonry
b, d - Enlarging by a concrete enclosure
c - Enlarging by a reinforced concrete enclosure

- 1 - Masonry
- 2 - New masonry, connected with the existing ones
- 3 - Old masonry
- 4 - Anchors
- 5 - Concrete enclosure
- 6 - Reinforced concrete enclosure
- 7 - Sand bed
- 8 - Concrete add
- 9 - Bearing beam
- 10 - Distributing beam
- 11 - Thickening concrete

Fig. 5. Methods for strengthening the bases

In the case of the cohesionless soils, If the masonry foundations are in a good state, their load capacity can be increased by their silicatization, i.e. by injection of cement mortar and water glass. If the soils are cohesive, it can be implemented an electrical-chemical method to enhance their carrying capacity (Fig. 7).



1 - Existing foundation
2 - Enlargement of the base
3 - Reinforcement
4 - Simple diagram of the compression loads before the enlargement
5 - Simple diagram of the compression loads after the enlargement and additional loading on the foundation

Fig. 6. Simple scheme of the increased loading under the foundations

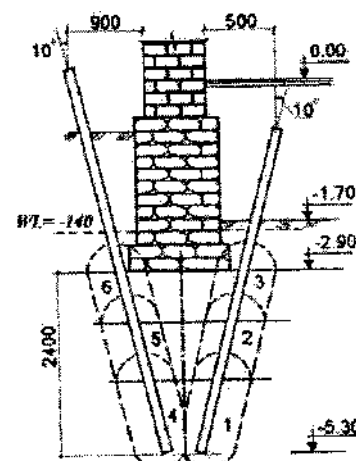


Fig.7. Electrical-chemical strengthening of the soil under the foundations

Some of these historic structures were built at sloping land. The rain and surface water were caused common wetting and erosion of the masonry. Therefore on the side of water moistening dry channels were built to collect and lead away water and the walls to remain dry. There are not serious problems about the strengthening of the historic structure to vertical loads. The main ideas and principles are presented here. The old and rotten bearing elements are replaced or strengthened. The damaged masonry can be rebuilt. The damaged

tension members are also substituted by the new steel ones including the anchor places. Different types of braces are used: steel (bars, shaped iron), wooden (circular, rectangular), steel with wooden cover, etc. The braces can be anchored by epoxide resin directly in the holes bored in the masonry. They can be anchored outside of the reinforced concrete legs (footings) hidden into the masonry wall. The holes can be drilled through the whole wall thickness and the anchorage can be made outside of the wall. Sometimes the tension members are provided with pipe thread to be tightened, if it is need. The damaged internal columns are replaced or strengthening in their lower footing. The cracks are filled up by appropriate structural mortar neutral to the masonry. All these activities require permanent and continuous contact between the structural engineer, architect, painter hydroengineer, heating engineer and engineer-geologist to control the results from the intervention. The capillary humidity is removed by adequate in disposition and in type isolations, air channels and floors, electrical heating, injection of masonry and subsoil for their hydrophobization. The insufficient foundation in depth, type and dimensions can be improved by spreading and tamping. In special cases the appropriate foundation pile system can be applied. The seismic vulnerability of these structures can be decreased by hardening of the roof disk (Figs. 4, 8) [14], i.e. by maximal diminishing of its strains and the seismic internal forces to be transfered onto the existing outer masonry wall. To achieve this aim at the top of the walls at the cornice level a hidden horizontal closed reinforced concrete frame (ring) can be built and connected by dowels with the underlain existing masonry (Fig. 9). It is very important to ensure the reliable anchorage of the hardened roof disc into the r/c frame. This frame can be provided with vertical r/c legs with or without footings hidden in the masonry walls too (Fig. 10).

In this way better connection between the frame and the walls is achieved and the braces can be anchored more reliable through the r/c legs or footings. If the roof is located at different levels, this frame can be built at the top of the largest and higher main structural body. Besides, if the limitations admit, vertical r/c columns hidden in masonry or as external wall pilasters can be also built and connected with the roof frame forming a space r/c system. But usually the canons of the East Orthodox Church do not permit the pilasters. Another way to strengthening the historic religious structure is to build a new r/c roof (vaults, domes, etc.) located above the existing damaged masonry roof (Figs. 8-10).

The new roof is structurally connected with the r/c roof supporting frame (ring) mentioned above. This roof can cover directly the old one or by an air interlayer to ensure the "breathing" of the existing roof. This is need in order to keep the climatic conditions in the temple. In this way the frescos can be preserved more successfully. The connection between the two roofs is realized by steel anchors (dowels) built-in them. In this way the old roof is hanged to the new one. This method is especially suitable in the case of very damaged existing roof. The new roof can be cast-in-situ or be built by light concrete blocks. In the second case thin steel meshes are used in the joints between the adjacent blocks. Ordinary it is assumed the new roof will sustain the old one. Besides the new roof prevents the old one from outside to the surrounding environment effects and can improve its isolations too. In another version of this idea the new roof can be built directly on the old one as a thin cover constructed by structural polymer mortar reinforced by thin steel meshes and strips of geotextile (geo-nets). The holes were remained in the cover to breath the old roof. This approach is implemented in the church "St. Iliia" in Iliamtzi [14].

Such idea can be extended if the old damaged masonry roof is hanged on steel roof frames anchored in the r/c roof horizontal frame hidden in the walls (Fig. 11). To ease the hanging a central r/c beam built-in the old roof can be used. This technique was applied to the church "St. George" in Kremikovtzi [14]. In other cases the pretension of cracked walls

in one or in two directions (horizontal and/or vertical) will increase the load capacity of the structure.

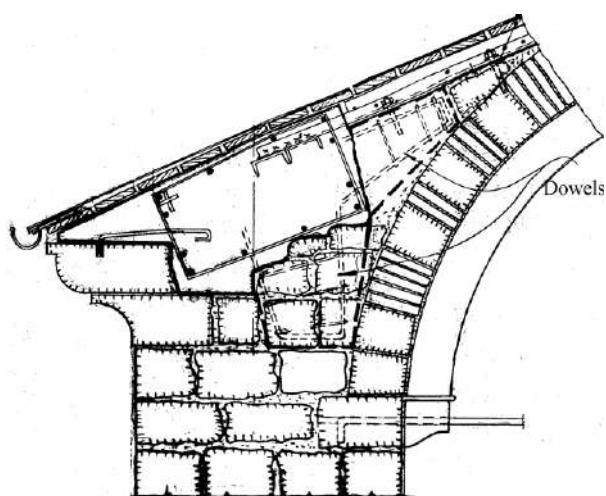


Fig. 8. Strengthening of the damaged masonry vault (dome) by hanging it on the new r/c one located above

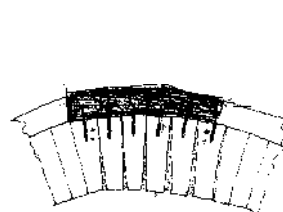


Fig.9. Dowels between the new and old roofs

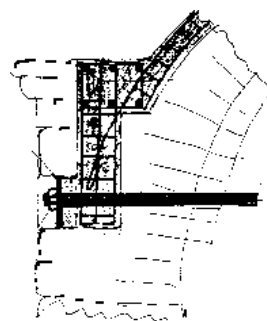


Fig.10. Legs, footings and brace anchorage

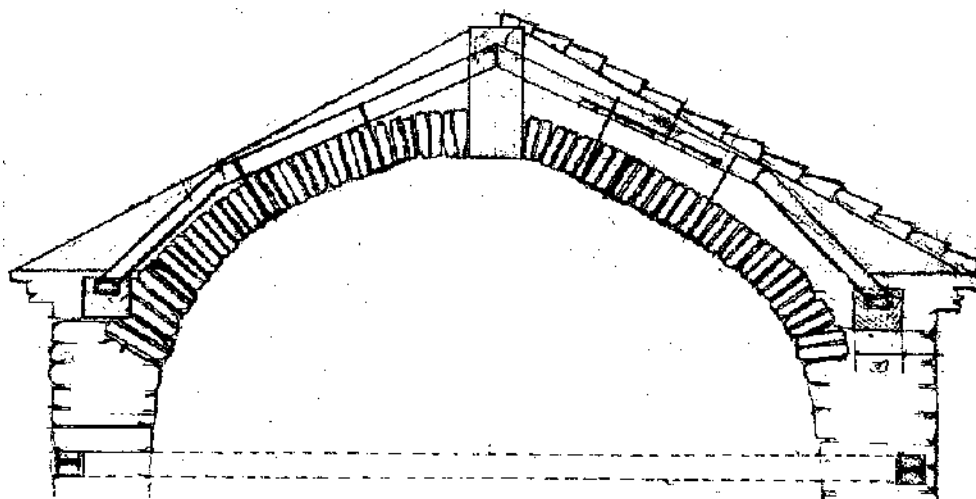


Fig. 11. Strengthening of the damaged masonry vault by hanging it on the steel frame above

If there are no other possibilities or they are insufficient, some monuments can be visibly strengthened from inside and/or outside. For example the structure can be girded with external horizontal closed steel or r/c frames which tighten the masonry walls. Horizontal ray-type beams can connect these frames with steel or r/c columns erected at a certain distance to the old building. All monument can be put on by transparent plastic to keep the visibility. The polymer can contain built-in special fibers for heating and thaw the snow by electricity. It is admissible for some monuments as minarets, towers, belfries, etc. to change them from inside. In these cases the strengthening can be executed by construction of an internal thin r/c enclosure connected with the old masonry walls by steel dowels. The reinforcement consists of thin straight and curved bars. The role of dowels can be performed also preliminary cleaned joints of the existing masonry removing their visible mortar and then gunite is injected. If the limitations admit, r/c columns and beams can be constructed outside and structurally connected directly with the external walls. This new r/c space structure can be hidden by stone facing or brickwork casing from outside.

This technology was applied in the church in Eleshnitsa. Besides in these easy cases the cracked walls can be restored as the following (Fig. 12) [11]. The old plaster is removed. On the two sides of the masonry steel meshes are mounted. They are united by steel anchors, crossing through the entire wall thickness at spacing 50-60 cm. The mesh covers the cracked surface and exceeds its bounds of 50-60 cm. Then the masonry and meshes are injected by the gunite 2,5-3,0 cm thick. If the crack reaches to the wall corner, the strengthening meshes continue minimum 100 cm in the opposite side (Fig. 13).

If the wall masonry cracks are strong, an more essential intervention is need (Fig. 14). On the two ends of the crack vertical columns passing through the whole masonry thickness are executed. The crack is crossed by intermediate horizontal and vertical belt courses located chess on the external and internal masonry sides. The pilaster thickness is from 10 to 15 cm. The brick masonry between them is reinforced with a mesh and then it is injected by gunite 1,5-3,0 cm thick. The masonry top end should be finished with a r/c belt course which to catch the roof wooden structure (Fig. 15).

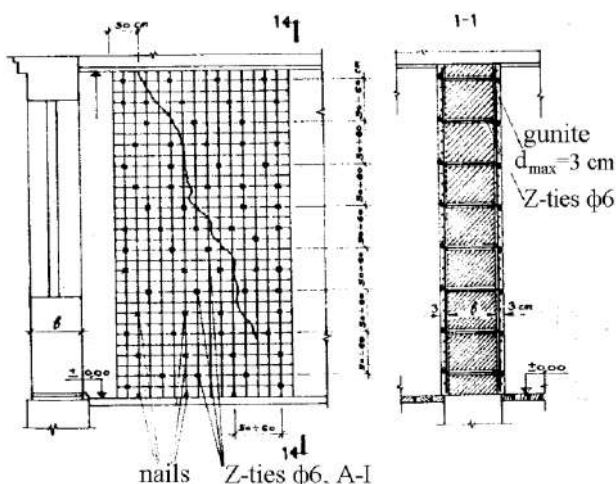


Fig. 12. Strengthening of the cracked masonry

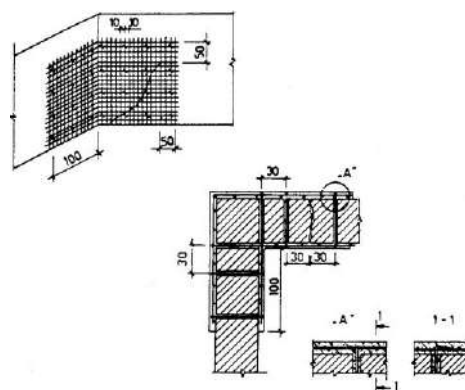


Fig. 13. Strengthening of the cracked masonry in the corner zone

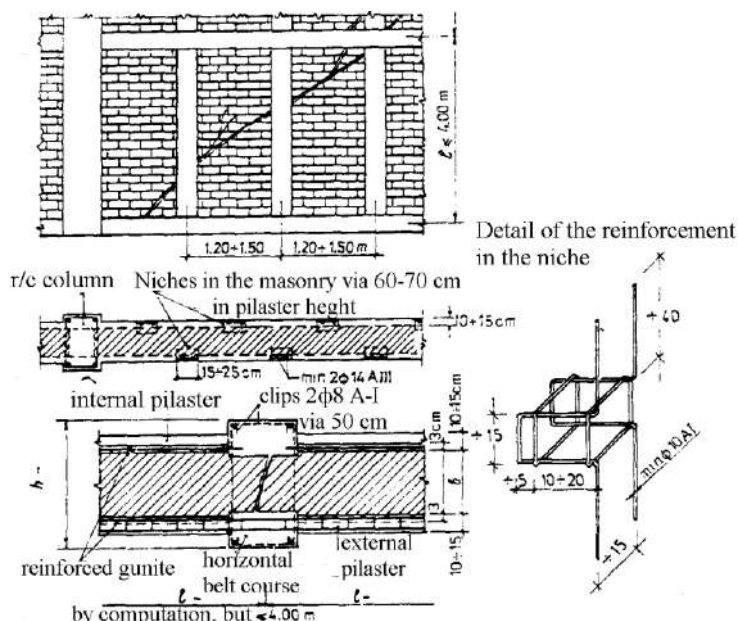


Fig. 14. Strengthening of the masonry with strong cracks

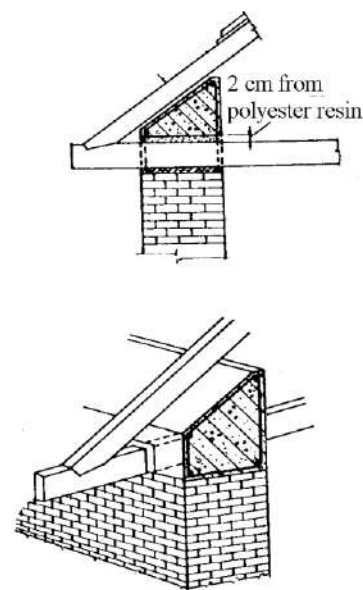


Fig. 15. Connection between the masonry and the wooden roof

If openings in the brick masonry are need, the two-side of the masonry can be covered with meshes anchored at intervals of 50-60 cm (Fig. 16) [11]. Moreover strengthening by the vertical and horizontal bars (above the opening) minimum 4N14 anchored at least 50-60 cm after and on the two masonry sides is need. To prevent the shear strains in the masonry around the opening, it is reinforced by horizontal and vertical stirrups on the opening contour. In the case of wall up the existing openings, they are reinforced by horizontal meshes via 3-4 brick rows (Fig.17). The strengthening of the strip between two walls located in one plane, can be made by cold formed shaped steel in the corners of the strip and its reinforcing by transverse and longitudinal bars continuing out of the strip minimum 50-60 cm still.

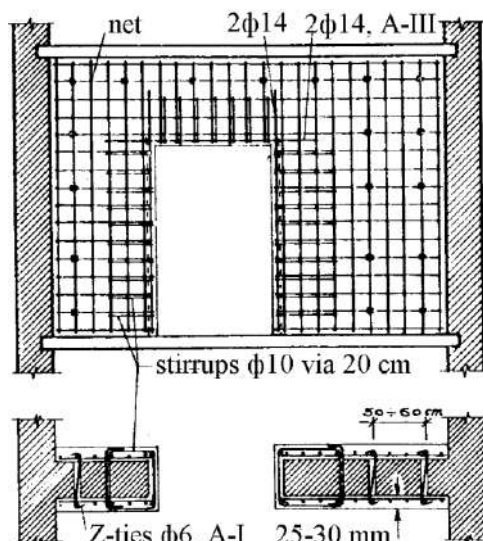


Fig. 16. Strengthening of masonry with an opening

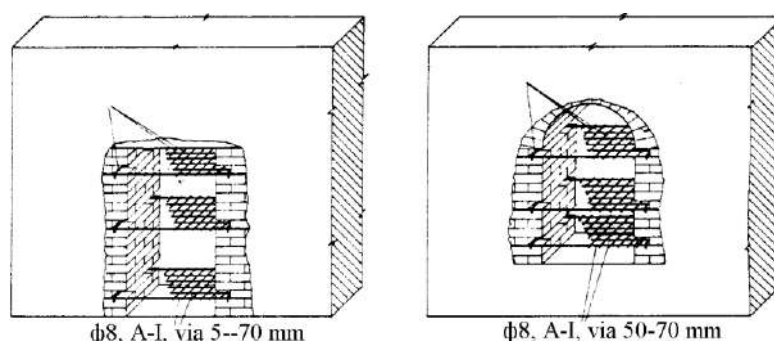


Fig. 17. Block up the existing openings

The deformed vault or arch can be contemporary supported by wooden (from the oak core) or steel curve-pieces. If steel pipe will be used, preliminary a sample is made by a steel bar 6,5 mm in diameter, bent on the real curvature. Then the steel pipe is bent using the thin sample bar. The cracks can be filled up by mix prepared by epoxide resin, stone, marble sand and brick powder and hardener. Large cracks can be enlarged and excavated to 8-10 cm in depth and then to be injected. Another way is to plaster up slightly the cracks by a thin layer of mix prepared by gypsum and stone or brick powder and then the internal cavities of the cracks to be filled up gravitationally by epoxide resin poured out carefully from above. In such cases good thickening of the cracks can be achieved. First of all in the beginning of the restoration and preservation process the monument is covered by contemporary steel cover till the necessary amount for safeguarding works is available.

This cover is used also during the construction works. It is made by steel frames (columns and beams) and corrugated iron for roof and walls. The engineering systems (heating, ventilation and electrical systems, a water supply and sewerage system) should be put in order also. In view of the features of the liturgy the heating system should be appropriate also to keep the interior and micro climate and to be economic. The heating systems used are by electrical heaters under the floor, by channels in which hot air circulates, by radiators, etc. In some cases special computer programs are implemented to find the optimal decision. Ventilation systems are only utilized in limited cases due to the interior, heating and climate.

6. PRESERVED AND RESTORED HISTORIC STRUCTURES

6.1. Strengthening of the tomb in Sveshtary

This tomb (Figs. 18-20) was built in 4th-3rd century B. C. for a Thracian eminent ruler [2]. It is situated at an archeological reserve "Sborianovo" which comprises the territory of one of the most remarkable centers of the religious and political life in ancient Trace. The tomb illustrates the perfect construction technology, impressiveness, rich sculptor and painting decoration, high creation and talent of the genius ancient master. Therefore the tomb is included in the list of the world cultural heritage. It was discovered in 1982 excavating a tumulus. The tomb consists of a corridor (4,06 m long, 1,85 m wide and 2,15 m high) and 3 almost square cameras of different heights (4,55 m of the tomb camera and 3,25 m of the side camera). They were built by well trimmed stone blocks from white limestone and they were covered by semi-cylindrical vaults. The total dimensions of the tomb are 7,52 m long and 6,23 m wide at the facade. This unique monument of exclusive significance is a part of an unified architectural complex.



Fig. 18. Thracian tomb

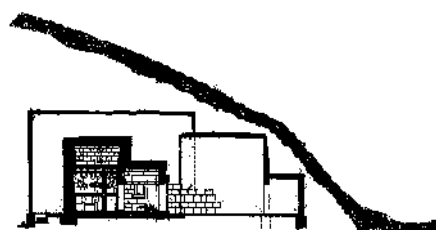


Fig. 19. Vertical section

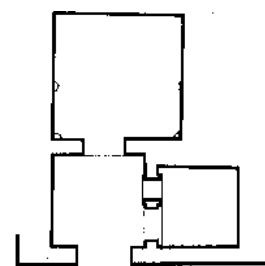


Fig. 20. Plan of the tomb



Fig. 21. Covering shell structure at the site

The structural and technological preservation, restoration and presentation of the tomb was a complex problem due to the following severe limitations: prohibition to the wet processes near the tomb, to keep the climatic conditions in the tomb, highest biological sterility of the used materials, maximal consideration and utilization of the soil properties (collapsible loess of second group), minimal risk, etc. Two temporary protection metal covering were mounted (Figs. 18, 19, 21) [17].

First one was also climatic and nearest to the tomb. It was assembled by the steel sheets simultaneously with the careful excavation around the tomb. A soil layer 60-70 cm thick was remained on the tomb to keep the internal climate. The second one was erected later on the tumulus part above the tomb and it was a tubular scaffold of large dimensions (about 18 m span) and far from the tomb. The constant covering is composed from two r/c cylindrical shells of equal span and 20 cm thick but of different radii of curvature (Fig. 21)

[17]. The higher shell was designed with a variable radius in two steps but the shallower shell has a constant radius. They were cast-in-situ at a site situated about of 35 m to the external temporary covering. The shells were provided with two external longitudinal supporting ribs near the ends. The vaults were lifted by 10 hydraulic jacks of 50 t every one and jacked down on 6 special transport double rail cars supporting the shell ribs. The back r/c retaining wall was constructed behind the tomb against the tumulus embankment. Then the higher shell was transported by pushing the rail cars between the two temporary coverings and to the back wall. Then the shell was slightly lifted by the jacks and jacked down on the lead thrust bearings placed at the preliminary constructed r/c strip foundations. The shallower shell was transported by thrusting the cars to the first shell, jacked down on its bases and the two shells were connected. The fore r/c wall was built. The internal temporary covering was disassembled. The soil layer on the tomb was removed. The external temporary covering was dismantled and the shell vaults were mutually and uniformly filled back. All technological process were realized by a strong time control. The shells were computed by the FEM applying the STRUDL program. This unique technology is also very effective. The technology and the transporting cars were technical innovations. The tomb was presented in 1985.

6.2. Research and strengthening of the basilica “St. Sofia” in Sofia

This church is a unique monument from the early Byzantine construction. It dates from the second half of the 5th century [1], [2], [3]. It is a three auditorium construction with total length 46,50 m and width 23,00 m (Figs. 22, 23, 24). The central part of the structure is divided into three auditoriums by massive masonry columns with a cross-type cross-section. They are connected one another by arches without capitals. The main auditorium after crossing with the transverse auditorium continues with a same height in the altar space and forms a Latin cross. On the crossing place of the two auditorium a square is formed marked by 4 columns that sustains the main cupola by means of 4 triangular spherical surfaces. The cupola is hidden from outside by tetrahedron. In 1878 the main architect of Sofia the Czech L. Bayer and engineer Prosech pay attention to the church keeping and restoration. The basilica ‘Saint Sofia’ has almost 15 century history. It had been constructed for a long time. The structure has endured some strong earthquakes. The last reconstruction of the church has been made in 1930. Two investigations were made to evaluate the seismic vulnerability of the church. The first one is accomplished for the unstrengthened structure in 1989 by the computer programs COSMOS [8] and IES using 3D brick finite elements [5]. The second analysis is performed in 1998. In this study the spatial model is composed from existing structural elements from one side and some new strengthening elements (two reinforced concrete shells above the existing masonry domes and six tension bars of the roof arches and vaults) on the other hand [6]. The analysis is made by the finite element method (FEM) using also the COSMOS computer program [8]. The model contains 2475 finite elements and 2283 nodes with 6 DOF (3 displacements and 3 rotations) in each one (Fig. 24). The total number of equation is 12666. The following types of finite elements are used:

- type 1: 3-node linear triangular thick shell element;
- type 2: 4-node linear quadrilateral thick shell element;
- type 3: 2-node linear 3-D elastic beam element.

The shell elements have bending and membrane stiffnesses. The shear deformations are taken into account. Elastic and isotropic material is assumed. The particular structural elements are defines as ‘regions’. The finite element mesh of each ‘region’ has a maximal dimensions up to 1,5 m (Fig. 24). The adjacent ‘regions’ are connected each other by

common nodes which guarantee displacement and rotation compatibility. Supporting nodes are entirely fixed. The geometrical characteristics of the structure are the following: the wall thickness varies from 1,10 to 2,00 m, the masonry vault thickness is 0,30 m, the new r/c shell is 0,10 m thick. The material properties of the brick masonry are: the Young's modulus is $E=1350$ MPa, the shear modulus is $G=0,4E=540$ MPa, the mass density is $=1800$ kg/m³, but for the concrete they are: $E=25000$ MPa, $G=0,425E=10625$ MPa, $=2500$ kg/m³. The basilica structure is analyzed under vertical load and seismic actions of 9th degree according to MSK-64 and Bulgarian design codes [7]. The first 6 periods of the undamped free vibration are obtained as $T_1=0,287$ sec, $T_2=0,216$ sec, $T_3=0,197$ sec, $T_4=0,178$ sec, $T_5=0,154$ sec, $T_6=0,153$ sec. The internal forces and stresses are computed considering the first 50 natural mode shapes. The seismic analysis is accomplished by two simultaneous horizontal seismic signals given by two correlated horizontal spectra. As a result the stresses in the dangerous points of the structure are obtained for the following two combinations: 1st combination - max stress = stress caused by the vertical load + stress from the seismic action; 2nd combination - min stress = stress caused by the vertical load - stress from the seismic action. The characteristic stresses of the wall masonry are: compression stress is 0,9 MPa, tension stress is 0,03 MPa, tension stress under bending is 0,04 MPa, shear stress is 0,05 MPa but for the roof masonry they are 0,5 MPa, 0,005 MPa, 0,01 MPa, 0,01 MPa respectively. The vertical stresses σ_z and the tangential stresses in walls and columns caused by the vertical and seismic loads exceed the material strengths. As a result brick elements are in danger. The stress concentration around the tension members is observed. Full scale dynamic experiment is realized too [4]. The numerical and experimental results have good agreement.



Fig. 22. St. Sofia basilika

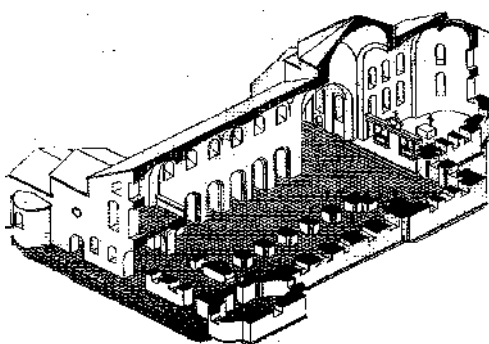


Fig. 23. Axonometric plan-section

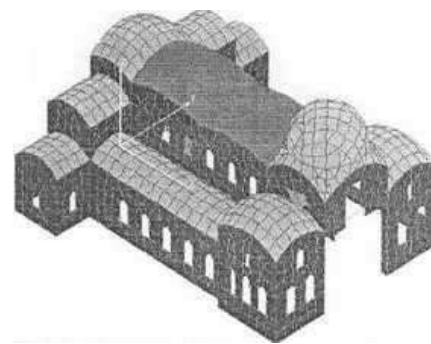


Fig. 24. FE model of the St. Sofia basilika

Many structural damages are detected [13]. Large cracks in vaults, domes and walls, soil settlements, deformations, decrease of the material strengths by the surrounding environmental effects, etc. are available. The main reason for damages are the past strong earthquakes. Some structural interventions are made during the last years to safeguarding of this unique and important monument in the center of Sofia. The investigations discovered that the structure responses as it is comprised of particular (isolated) fragments due to the deep and large cracks between them and insufficient contacts between the interventions. The structural spatial unity is slightly expressed. The basilica was very vulnerable to seismic actions. Therefore the effective structural measures were accomplished to restore the entire response of all structure. The cracks are filled up by injection of mix prepared by epoxide resin, stone powder and marble sand (Figs. 25, 26) [13]. A SIKA injection technology for restoration of the masonry continuity is applied. The large cracks are excavated 10 cm deep and enlarged to ease their injection. Some of cracks

are filled up gravitationally discharging carefully the mix mortar from the roof after the cracks were preliminary closed by a thin surface coating. About 2 tones epoxide resin are used. Some masonry and masonry joints are rebuilt. The r/c floor plate and beams are strengthened. The new r/c vaults and domes with their supporting beams and rings are cast-in-situ on the old cracked brick masonry ones. The cracked masonry roof elements are hanged on the new ones by steel dowels (Fig. 26). The tension members of high strength steel bars are mounted in the arches and vaults. Their anchorage is in holes 50-60 cm in a diameter and 120 cm deep drilled in the walls or outside of the walls using the epoxide resin or cement mortar. They were tensioned by thread studs or nuts. Soil injection ate executed also. Some stone masonry foundations are enlarged and made deeper. Some temporary openings were blocked. An external r/c belt course is built at a level 0,00. Some tension members are assembled under the floor plate too.

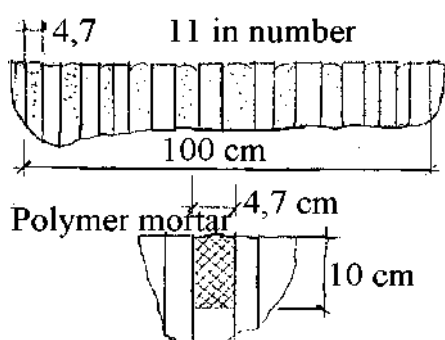


Fig. 25. Filling of cracks

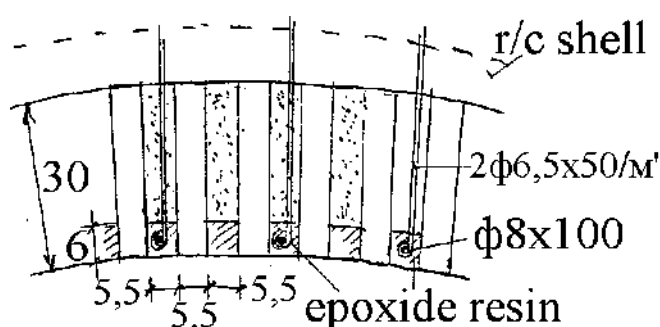


Fig. 26. Anchorage of anchors in the domes (vaults)

6.3. Strengthening of the church “St. Iliia” in Iliantzi

The vault of this church has numerous severe cracks (Fig. 27) [14]. The large cracks cross through the whole thickness. As result the vault consists of 4 particular parts and it is strong deformed. The new roof is built directly on the old one as a thin cover constructed by structural polymer mortar reinforced by thin steel meshes and strips of geotextile (geonets) (Figs. 27, 28). The holes were remained in the cover to breath the old roof. The old roof and its covering are suspended on the new transverse steel frames structurally connected by vertical legs to the r/c beam hidden in the masonry walls. The suspension members are anchored through the joints in the old roof connecting with longitudinal reinforcement mounted in the longitudinal joints. In this way the braces are united at two levels. The large cracks are filled up by the epoxide resin from inside. A trial fragment was prepared and tested. The deflections during the disassembly of the scaffold after the strengthening are controlled by transducers. The total deflections were up to 1-2 mm only.

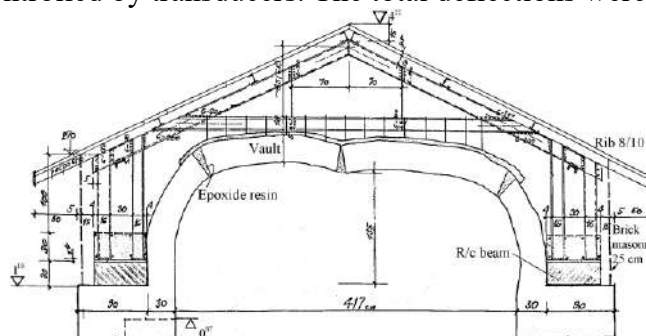


Fig. 27. Strengthening of the vault by polymer mortar and suspending on the new steel frames

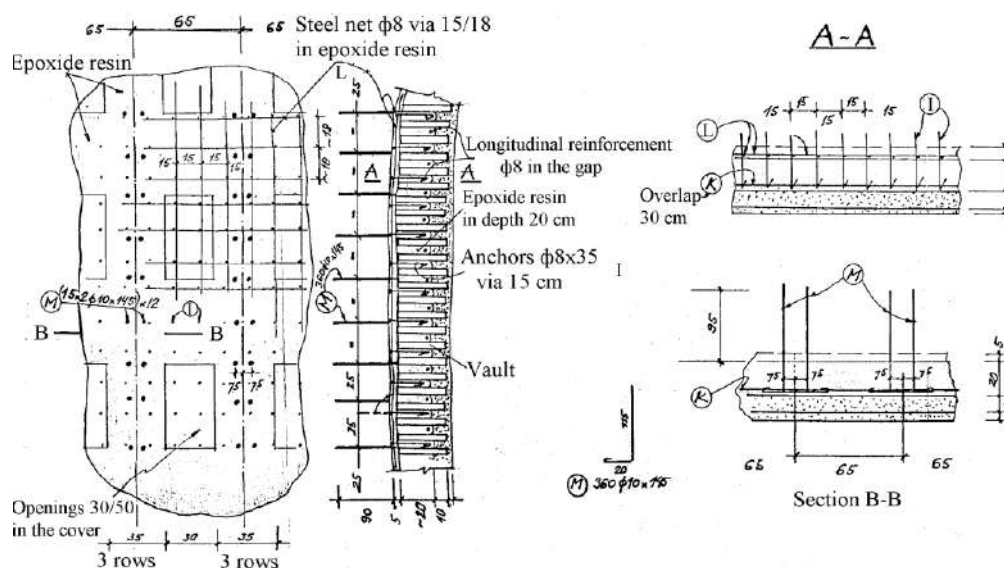


Fig. 28. Anchorage of the suspension braces and filled up the cracks by epoxide resin

6.4. Analysis and strengthening of the minaret of Tombul mosque in Shumen

The Tombul mosque (Figs. 29, 30) has been constructed in about 1743. The mosque minaret is 38,74 m high [21]. Till the +8,65 m level the minaret is joined to the mosque lower body but this connection is structural up to the +5,57 m level only. The minaret main high prismatic part has a circumcircle diameter of 1,70 1,53 m and wall thickness of 23,0 18,5 cm. The aboveground minaret structure up to the +30,55 m level is constructed by stone masonry. The principal damage of the minaret masonry is series of particular almost vertical cracks at the foot of the main prismatic part (Fig. 30 a). The cracked sector is 2 3 m high. It is need to check the minaret safety.



Fig. 29. Minaret in Shumen

The full-scale dynamic tests of the minaret part between the 0,00 m and +25,41 m levels have been done (Fig. 30) [24]. The masonry part of the minaret structure between the levels of +5,57 m and +30,55 m is studied numerically also (Fig. 30) [21]. It is modelled as a vertical plane cantilever beam-column rigidly fixed at its bottom to the minaret low body (Fig. 31). In order to consider all structural features the beam is divided into 20 finite elements. The linear dynamic, static and stability analyses of the model are accomplished by the FEM and ROKU program [22]. The well known uniform beam-column finite element of six DOF is used. The seismic loads, the dynamic and resonant wind loads are determined by the response spectrum method according to [7] and [25]. The structure static stability is verified and then the higher-order bending moments caused by the vertical loads on the deformed beam axis in small displacements are computed by the iterative Vianello-Dischinger's method. The cross-section bearing capacity is checked by the stresses.

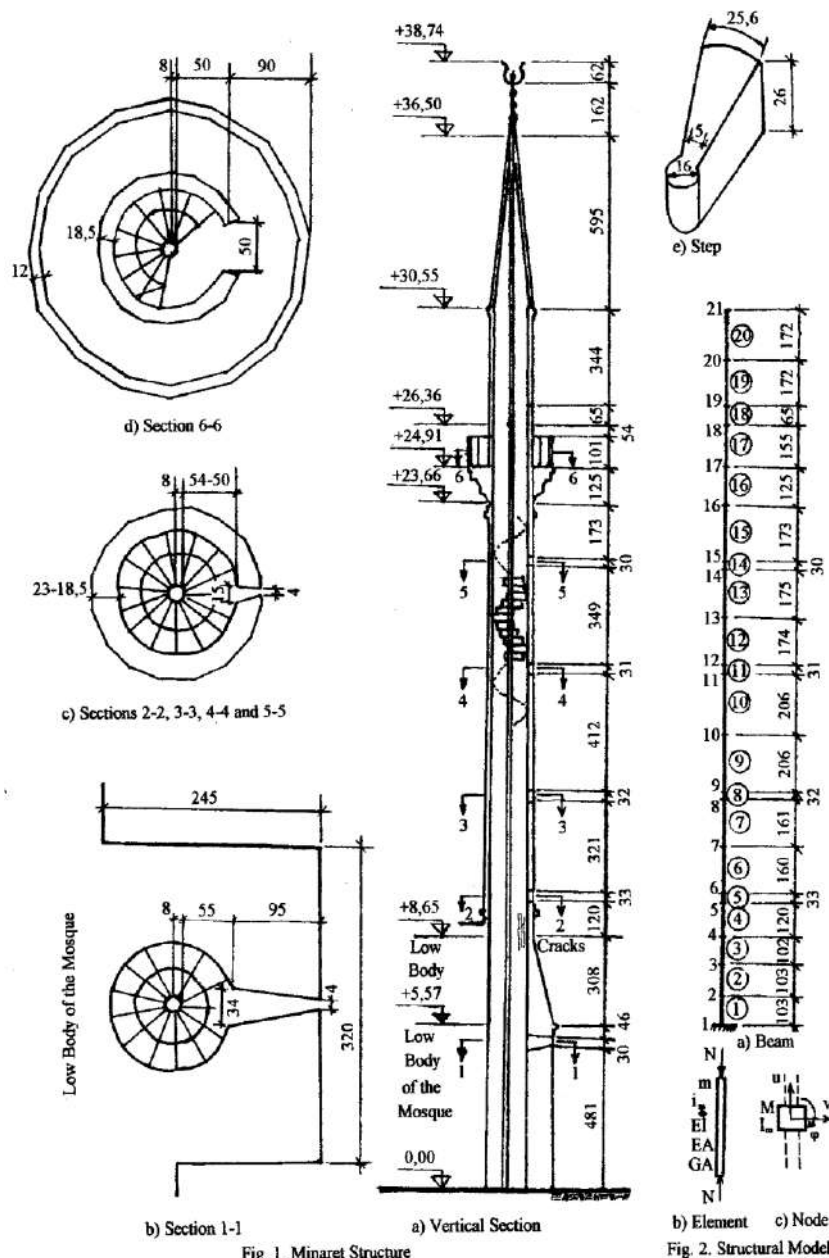


Fig. 30. Minaret in Shumen

A good agreement between experimental and numerical results is obtained. The structure safety is not ensured due to the horizontal loads. Therefore an urgent strengthening of the stone masonry is need to be made only from inside by an appropriate way to restore the minaret normal safety in accordance with the current Bulgarian design codes. The strengthening should cover in height the whole internal surface of the masonry wall or at least the unsafe lower portion of the main prismatic part. Two alternative ways for repair and strengthening are discussed in [21], [23]: prestressing by internal vertical tendons and construction of an additional internal reinforced concrete skin. The first seems to be a simple and elegant solution which would leave no visible traces. However it was considered as impractical because the masonry cross-sections will be too heavily loaded to withstand the additional compression from the necessary prestressing force of about 1000 kN. Nevertheless the first way was adopted but it was not been applied due to financial shortage. Moreover several cracked stones can be replaced by new hard ones. Probably the

second technology will be more rational. It has been applied in Hungary [23]. A thin reinforced concrete skin 5-8 cm thick can be constructed over the whole internal surface of

the masonry wall. Concrete grade C28 has been used and injected in place. It can be reinforced by vertical steel bars N 20-25 mm and 8 mm thin helical wire following the minaret ascending internal stairs. The vertical reinforcement can be made continuous spliced by thread-bars through 30 mm holes drilled in the stairs close to the wall and by ensuring adequate laps of the bar successive lengths. To ensure the full interaction of the existing masonry and the new internal skin, the mortar internal joints have been raked out and the stones have been washed before concreting. Thus penetration of the raked-out joints by the concrete has provided continuous shear practical dowels between the two materials (stone and concrete) for their work together.

7. RECOMMENDATIONS AND PROPOSALS

Now the basic difficulties for safeguarding of the cultural monuments are the financial shortage in the last years and the activity of the treasure-hunters. The numerical reanalysis and redesign as well the full-scale dynamic and static experiments of particular phases during the structure repair are necessary. The use of high performance materials and techniques as carbon fibers, geo-nets, fiber reinforced plastics and grids, steel nets and bars for reinforcing of historic, old and new masonry should be enlarged. But the limitations should be given in applying the modern materials and technologies to keep the original architecture of the monuments and to escape unfavourable effects to the environment. The cultural heritage, restoration and preservation works of historic and old structures should be studied by students and specialists in the corresponding universities and colleges. Specializing post-graduate courses for professionals will be useful. Moreover scientific problems for research and renovation of such structures are actual and important for diploma works, post-graduate diploma and Ph.D. theses. International research and educational projects are need to be elaborated in this domain. Scientific workshops, conferences and other activities should be held to discuss and exchange experience, achievements and knowledge on these problems between the professionals. More data should be collected in this area. Closer relations between the universities, institutes, companies and specialists are need. Larger amounts should be given for safeguarding of the cultural monument all over the world and especially in Europe. The national governments and the authority of EU are responsible to enlarge and support these activities and to make them reality. The cultural heritage of the world must be kept for the next generations by all means[26].

8. CONCLUSION

Special attention should be paid to prevention measures for safeguarding of the historic objects. As a rule the historic structures are not ensured to horizontal loads and especially to strong seismic actions. The human intervention on the historic structure should be minimal, careful and adequate. New and effective models, methods and technologies should be used to study, preservation and restoration of these unique and valuable cultural monuments. The safeguarding of the cultural heritage becomes one of the main problems of each civilization. The relation to the cultural heritage is a mark for the civilization of every nation. The European countries with their long and rich cultural history take a special responsibility to keep the roots of life at the Earth. In the future united Europe every country will keep your national cultural identity. The richness of the Europe is in its cultural and historic variety.

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ABOUT THE HISTORY OF CONSTRUCTION, PROJECT DESIGN AND ERRECTION OF THE PALAZZO DEL LAVORO IN TORINO

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Abstract: *Viewed as a symbol of integration between structural and architectural invention, the Palazzo del Lavoro has fascinated entire generations by emphasizing, with a certain mannerism, the overly exposed role of the structure. The competitive tender for the construction of the 47,000.m² pavilion that, for the Centenary of Italian Unification, was to have hosted an important exhibition on labour, was won by Nervi & Bartoli with Pier Luigi Nervi, his son Antonio and Gino Covre in 1959. The project revolved around the subdivision of the square roof into 16 independent “umbrellas”, each 40.meters per side, separated by continuous strip skylights and made from a sunburst pattern of steel beams fixed to a central column with a variable geometry, a recurring characteristic in Nervi’s work. The perimeter gallery is instead constituted of Nervi’s typical isostatic ribbed slabs, realized using moveable ferroconcrete formwork, based on a process widely tested by Nervi in various buildings. The proposal was deemed convincing for its simplicity and structural legibility and, thanks to the modular solution and differentiation in materials, it was the only submission capable of guaranteeing completion within the limited time available. Beyond the technical data, nonetheless impressive –.158.meters per side, 26.meters in height, and a total volume of 650,000m³ – the most innovative aspect of the building was, in reality, the organization of the building site. Work began in February 1960 and by late December the building was already complete. This huge architectural space, listed by the Ministry of Cultural Heritage in 2011, has continued to pose problems of reuse since the ending of the “Italia ’61” celebrations. After years of abandon and neglect, a variant of the Master Plan, approved by the City Council in 2008, provided for a new use of the structure: the building, now privately owned, should be transformed into a shopping mall housing also boutiques, restaurants and public establishments.*

“If one considers the near future of construction, I think that we can glimpse a progressive separation between residential architecture and the great works of a collective nature which, driven by general progress, will increase their dimensions and thus progressively reduce the freedom of choice of their structural and constructive schemes. Indeed it is evident that, when the dimensions of a building are very large or, for any reason, important issues of statics are at play, the liberty of the schemes, shapes and dimensions of the resistant structures is diminishing until once certain limits have been reached they become virtually nil”[1].

Pier Luigi Nervi

Key words: *Palazzo del Lavoro, 16 independent “umbrellas”, huge architectural space*

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1. Introduction

The engineer and constructor Pier Luigi Nervi [2], spoke these words during a lecture at Oxford University in 1960. Perhaps he was referring to the “huge structure” of the Palazzo del Lavoro of Turin where the building work –which had begun in February 1960– was coming to an end at the time of his Oxford lecture. Besides “static problems,” numerous other factors had come into play while determining the project solution which enabled the Nervi & Bartoli Company to win the tender published by the Committee for the celebrations of the Centenary of Italian Unification (Italia.'61). The entire question of Palazzo del Lavoro –from its design project to its construction – reflects the complexity of Nervi's design undertaking. It reveals the determination with which Nervi combined the roles of designer and entrepreneur throughout his career, thus maintaining a continuity between the moments of invention, calculation and then of execution so as to control all the economic and quality aspects of the work [3]. Seen as a symbol of the integration of structural and architectural invention and publicized by major national and above all international journals [4], the Palazzo del Lavoro has fascinated entire generations [5]. By emphasizing, with a sort of mannerism, the way in which the structure is almost flaunted, it marks the transition to the third phase of Nervi's design activities, that of the great international commissions in which the “Nervi system” became a repertoire of solutions to be adopted around the world.

2. The tender

The call for tenders for the construction of the Palazzo del Lavoro, the exhibition pavilion in Turin built for the 1961 celebrations of the Centenary of Italian Unification to host the great exhibition on labour directed by Giovanni Agnelli and staged by Gio Ponti, was published on 4 July 1959 [6]. Based on a preliminary plan by Ludovico Quaroni, it required the construction of a symmetrical pavilion along two principal axes at right angles to each other. The building was to provide 47,000°m2 of usable surface area for exhibitions and a range of services, and was to be used subsequently as a National Centre for Professional Education [7]. The call for tenders [8], which put the emphasis on low construction costs and the “expressive” use of construction techniques and materials, allowed three months for the development of a working plan, including structural calculations, estimated bill of quantities and financial quotation. The panel of judges was chaired by Viorio Bonadè Boino, head engineer of the Construction and Systems division at Fiat, and the panel members were Lodovico Barbiano di Belgioioso, Luigi Carlo Daneri, Franco Albini, Adalberto Libera, Giovanni Michelucci and Roberto Pane. Six Italian companies, with their associated architects and engineers, presented tenders: Borini and Padana with Roberto Gabei, Aimaro Isola and Riccardo Morandi; Dalmine with Sergio Nicola and Aldo Rizzoi; Guerrini with Carlo Mollino, Carlo Bordogna and Sergio Musmeci; Guffanti with Piero Locatelli; Nervi and Bartoli with Pier Luigi Nervi, his sons Antonio and Mario, and Gino Covre; and Recchi with Gino Levi Montalcini, Aristide Antoldi and Angelo Frisa [9]. On 20 October 1959 the jury judged that only Nervi and Bartoli's proposal was entirely suitable (Fig.1).

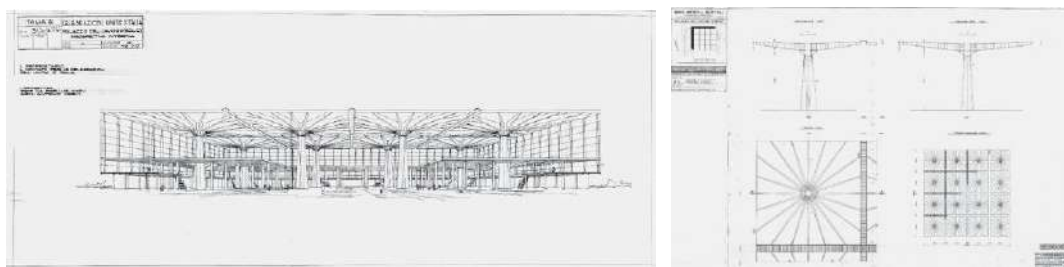


Fig.1. The project presented by Pier Luigi Nervi, 1959.

Fig.2. The square based umbrella at the base of the project, 1959.

Although it lacked a number of the tender requirements, for example, having the unified surface broken up by pillars – which Turin-born architect Carlo Mollino [10], attacked in an eventually unsuccessful appeal – Nervi's proposal was convincing in its simplicity and structural legibility. The Nervi design [11], the only one capable of meeting the very tight deadlines, was centered on the subdivision of the square roof into 16 independent umbrella-style elements with sides of 40 meters, separated by continuous strips of skylights and consisting of a central pillar with a cross-section varying continuously from a cruciform shape at the base to a circular shape at the top, and radiating steel beams. The independence of the individual umbrellas, taken as the inspirational motif, has continued to be the symbol of the building. The choice of the square structure was taken to be a declaration of honesty over extravagance, but it was above all the result of precise economic conditions, and it was the only possible answer in view of the time constraints. Having a year at most for its construction dictated the need for a modular structure based on the reproduction of identical independent units that were self-supporting and able to be executed separately. The variable geometry of the support structures is a recurrent feature in Nervi's work (Palazzo dello Sport, Rome 1959; Corso Francia viaduct, Rome 1960; Savona railway station 1961). Besides its indubitable value in terms of form, this above all dispensed with stability and construction issues. The modular solution and the differentiation of the materials would allow the almost simultaneous process of creating the structure and the finishing, and managing the problem of the casting and hardening times for the reinforced concrete [12]. The consensus of opinion between the designer and the company also worked in favor of efficiency and speed of execution, as had been the case in previous designs that Nervi had completed Exhibition halls (1948-1950), the Fiat factory buildings (1955) and the public tram depot (1954) [13].

Nervi had initially intended to use reinforced concrete for the roof, re-utilizing the system of ribbed umbrella structures he had adopted in previous works. Instead in September 1959 he called in Gino Covre [14], one of the principal Italian engineers for steel structures. For the composition of the cantilevered beams, Covre had proposed two solutions for the tender: one with a solid I-beam section with vertical ribs to reinforce its core and another, more metallic version, composed of a lightened node framework and secondary points which had not been established beforehand. The committee judging the tender chose the first solution, even though it was heavier and more expensive. Until that time not many buildings had been constructed using steel structure: the most important of these were the ENI office block by Marcello Nizzoli and Giuseppe Mario Oliveri (1956-1958) in San Donato and the Turin SIP office building by Domenico Morelli (1957) [15], while the Rinascente designed by Franco Albini and Franca Helg and built by the Badoni Company with Covre was still under construction in Rome. However, the solution that was adopted in the end used solid I-beams with reinforcing ribs, visible, connected by a perimeter beam that served as a strengthening element for the entire system, with the effect

of harnessing and redistributing the loads between the various projection beams. With sheet-steel insulated waterproofed cladding panels, the 16 squares of the roof covering were joined together by a network of 2-meter-wide skylights composed of metallic up-sidedown V-frameworks with metal window pane holders without putty. From the functional organization standpoint the project envisaged the creation of two exhibition levels: one on the ground floor, below ground level, and the other about 10 meters above, supported on a mass of pillars and crossed by perimeter pillars with a square opening at the centre; on the intermediate floor there was a continuous perimeter gallery, the outside entrance and the services.

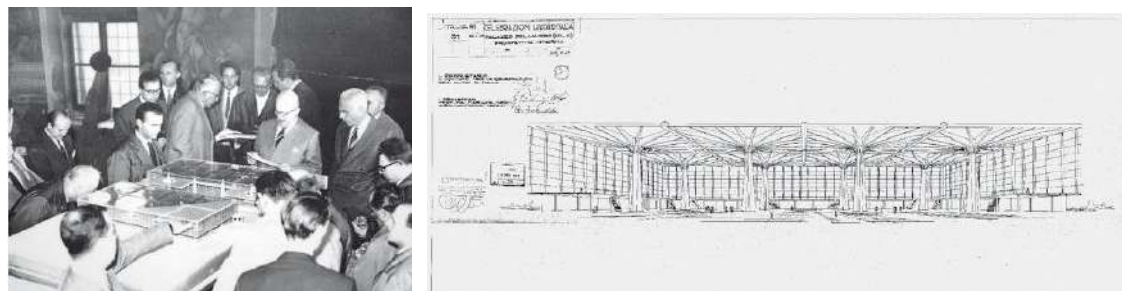


Fig.3. The presentation of the winning project by Nervi (on the right), October 1959.

Fig. 4. The new project approved, 22 January 1960, perspective cross section

3. From tender to project execution

After various changes, the site of the Palazzo del Lavoro was defined on 23 November 1959 as being on the southernmost edge of the Italia '61 exposition complex with a parallel axis to the south radial road Corso Polonia which had been built in 1956. The competition project had to be finalized and executed with the supervision of the Fiat Constructions and Systems division [16]. The project was immediately subjected to a series of changes to ensure smoother execution -and to satisfy the exhibition requirements which were defined in parallel with the start of construction. After various meetings with Gio Ponti, Ludovico Quaroni, and members of the executive committee of the Labour Exposition, in May 1960 the Nervi studio drew up the definitive project [17]. The basement floor was raised to ground level as requested by the adjudicating committee [18], while the second level was deferred until after the celebrations. The 10-meter-perimeter gallery remained intact and was built with isostatic ribbed floors inspired by the isostatic lines of the bending moments, a variation on the orthogonal crossed ribbing patented by Nervi in 1949 which he had already used in 1953 for the Gazi Wool Mill in Rome. A basement floor was added for service areas, along with a day hotel and two projection rooms. Vertical access routes between the hall and the gallery, which varied in parallel with the exhibition solutions proposed by Ponti, were finalized as eight traditional staircases and seven escalators placed in correspondence with the perimeter pillars. Heating and ventilation systems were projected [19], whose realization was postponed until after the celebrations were over [20]. Ponti urged the elimination of the second level supported by what he did not hesitate to call "a jungle of 182 little pillars that left, visible only the four central columns and obscured the view of the others." Ponti frequently laid claim to his role in the transformation of Nervi's project [21], a contribution that his friend Nervi never failed to acknowledge [22]. Also at an advanced stage of the works, Ponti suggested a series of changes to the project. In particular he wanted the lighting of the exhibition space to come only from below so as to highlight the columns and the umbrella

effect of the beams, but Nervi disagreed [23]. In the competition project there was a series of details that had remained unsolved, including the glass walls [24]. "Of this excellent project," as Bruno Zevi wrote on the day after the results of the competition were announced, "the only worrying aspect is the inconsistency of the envelope. Each modular scheme presents the problem of figuratively concluding the repetition of the modules, of connecting its rhythms to the outer walls, designing the latter in conformity with the structural language. From what one can make out from the scale model, Nervi has not yet tackled the issue, the glass walls that provide the envelope are vague and anonymous, while the reinforcing corner pillar is weak and out of context. Identifying the construction method of a building of this type is to travel 99% of the path to art; but this will be achieved or not by solving the last 1% which means exactly the moulding of the envelope in its two-dimensional rhythm" [25]. The elaborate external wall was worked out later by Covre. It is 19 meters high and divided in two by an opaque strip of the same thickness as the floor to be completed later, which was completely independent from the structure and was left undefined for the tender competition. The external facade is composed of a curtain wall constructed from the inside, with brisesoleil [26], and sheet-metal shades fixed at the bottom on cardan joint supports and at the top on piston rods, absorbing horizontal movement and expansion [27]. The windows of the facade are in aluminium while the doors are in iron [28]. The independence of the outside envelope compared to the internal structure is further highlighted by the removal of the four corners, a solution that intends to underline the leading role played by the structure in architecturally connecting the building. In reality only night views – when the envelope virtually disappears – or indoor views make it possible to grasp the majestic movement Nervi imprinted on the structural arrangement. During the day the envelope alone is absolutely impenetrable: cutting off the rhythm of the interior pillars, it does not express the umbrella structure.

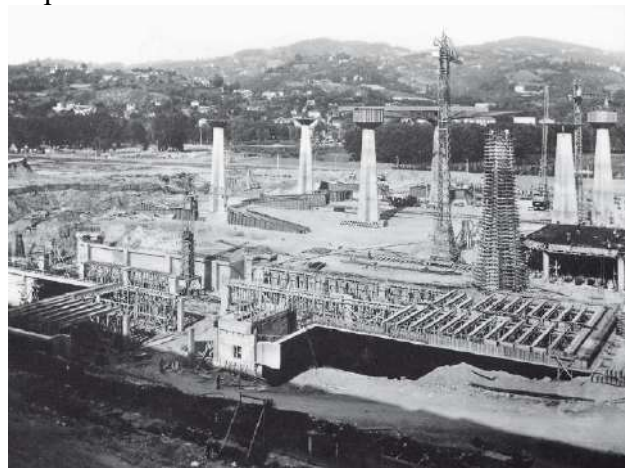
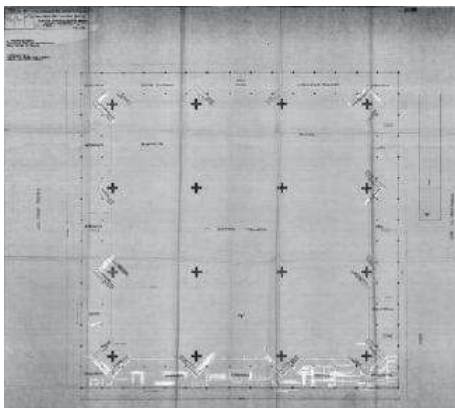


Fig.5.The new project approved, 27 January 1960, plan.

Fig.6. General view of the construction site

4. The building site

Apart from the technical or purely quantitative data – 158 meters long by 26 meters high with 650,000 m³ of volume – the construction site and its organization constituted the most innovative aspect. The building works started with outlining the foundations on 1 February 1960 [29] and the construction was finished on 31 December of the same year [30]. The excavation and backfilling began in mid January and went on until April, while the palification of the foundations was concluded in June [31]. The foundations were outlined at the same time as the plinths and load-bearing walls: work on continued into August as they were built step-by-step the pillars were while being completed. About 300 people

worked on the Nervi Company building site. To these should be added the workers putting up the metal structures, those involved with the doors, windows and systems, technicians and draughtsmen, for a total of about 1,000 people [32]. The site data [33], reveal that the pillars and roof were constructed over a period of a month: each pillar corresponded to construction of one unit of roofing. The building of the pillars began on 1 April at the rhythm of a pillar and a half per month: by 30 July ten of them had already been completed. The assembly plan for the roof and perimeter floor casting [34] reveals that the metal roof started from one side of the perimeter, then the roofing of an adjacent perimeter pillar and a central one was laid, and from this point work proceeded with the roofing of the first and then the second row of pillars (so as to have two complete rows); in the third row two non-adjacent roof coverings were laid and then the fourth row was dealt with, concluding with the central column. Nearly 21 meters high, the 16 reinforced concrete pillars rose from a cruciform base (5 x 5 m), and they were progressively tapered and moulded to close at the top with a circular supporting area with a 2.5-meter outside diameter. The need to have a self-centring, freestanding formwork (without internal tie-rods) whose weight could be lifted by a crane and with a volume that allowed casting to be concluded in a day led to the subdivision of the formwork into six shafts, marked with a 2x2 channel to separate concrete pouring and absorb the inevitable differences in color of the cement mixture. The work plan set the pace of completion at ten days for each pillar: once the formwork had been mounted, the cement was laid in 24 hours with the night shift, and then the next shaft (1 day) was done and the two areas of concrete were poured in the 24 hours and so on for the three shafts for a total of seven days which became ten with the cleaning of the formwork. Produced externally with an iron beam framework, on the inside the individual type moulds were fully lined with a double layer of wooden planks. The planks on the inside, which were no higher than 12 centimeters, were cleaned and planed after each pour [35]. To facilitate the organization of the building site and to aid work progress also in bad weather, scaffolding was built with waterproof sheeting spread over the top to cover the entire pillar, and heating was provided inside this provisional worksite structure to enable complete drying of the concrete [36]. At the same time, the roof beams were being constructed in the workshops and transported to the site by the Badoni Company from Lecco. At the top of the pillars was a truncated cone capital, anchored to the concrete structure by means of fastening screws, and a polyhedral corona joined to the fixed-joint drum of the 20 cantilevered I-beams connected by a perimeter beam that served as a strengthening element for the entire system. The total weight of each square was about 125,000 kg. The ground level floor was made up of the main beams poured on site onto which were laid 10-meter-prefabricated beams at a distance of 1.66 meters. On these rested prefabricated slabs reinforced with poured concrete joints above the secondary beams between one floor slab and another. The isostatic ribbed slab on a mobile deck, which is the real synthesis of Nervi's work and of his concepts of economy with a constant quest for the aesthetic appearance through the application of the laws of statics, was realized using formworks in ferro-cement. These were composed of individual assembled modules which were rolled along rails and lifted again by jacks where they were set ready for the next span of the slab to be poured, according to a procedure Nervi had already widely experimented with in various buildings including the Gatti Wool Mill (1951-1953) in Rome, the Manifattura Tabacchi in Bologna (1951-1953) and the Mirafiori workshops in Turin (1954). During the executive phase the passage from concrete pillars to steel roofing looked incongruous. As speed of execution was paramount, the mixture of heterogeneous materials was not let to its authentic look. The steel was painted utilizing a rather cement-like light grey gloss paint, which defeated any intent of contrast and tended to conform it

with the concrete, revealing Nervi's propensity for the original idea of a building all in ferro-concrete, including the roofing.



Fig.7.The construction of the “umbrellas”

Fig.8.The construction site

Fig.9. The construction site: welding roofing sheets

5. Inside setting

The setting and layout of the exhibition, coordinated by Ponti but with the involvement of the most advanced architectural and artistic culture of the day, from Ettore Sottsass to Marco Zanuso, and from Lucio Fontana to Bruno Munari, was strongly influenced by the imposing structure of the building, “rejecting any solution that did not leave all the columns visible” [37], it consisted of extremely light, ephemeral partitions in aluminium, and panels clad in translucent, reflecting stainless steel which enclosed the Italian section. These slender translucent or mirrored elements represent a deliberately transitory construction arranged alongside the monumental expression of Nervi's columns. In the choice of setting it is possible to glimpse Ponti's difficulties in structuring an exhibition layout in a gigantic hall without proportions. Nervi's building is imposing with its monumental columns but it lacks spatial rhythm, it is a static space that is not designed, and it needs another substructure to be inserted for it to be shaped and modulated. Before this immense space, which excludes views and stands empty and inert, Ponti built crooked 12-meter high walls that broke up the rigid geometry of the container. The main criticism leveled at the Palazzo del Lavoro after its inauguration was that “it is an enormous umbrella that needs to be rebuilt inside for it to be utilized” [38].

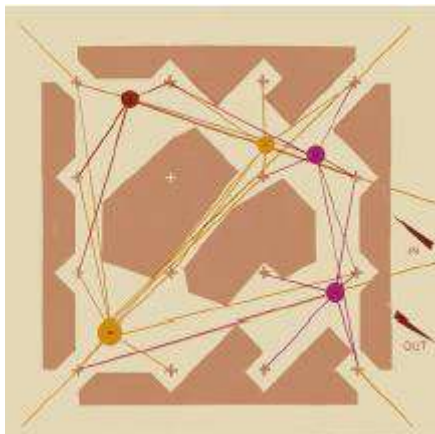


Fig. 10. The design by Gio Ponti, perspective views on the columns, 29 April 1960.



Fig.11. Interior view of the exhibition setting.

6. From 1961 until today: abandoned and reutilized

In Ugo Gregoretti's 1963 film *Omicron*, the buildings of Italia '61 depict the backdrop of a science fiction future metropolis [39], worthy of the best scenarios of the Mickey Mouse universe. The future of that imaginary city instead became a chronicle of demolitions, increasing neglect and missed opportunities. Bruno Zevi had already identified the difficulties that would be encountered in utilizing those structures: "Architects plan buildings independently from their contents, then architects arrive to set up the exhibition and they have difficulty filling the general predetermined spaces. At the root of all the aesthetic and technical defects of exhibition architecture is this procedural dissociation. The perimeter of the Colosseum is 527 meters, that of Palazzo del Lavoro reaches 640, the Basilica of St Peter's covers an area of 15,160m² and the Palazzo del Lavoro 25,000m². But who is interested in these data? The positive verdict for Italia '61 is restricted to the town plan. The rest is rhetoric, more or less mechanistic futility is folklore given that nobody has yet invented a possible rational utilization of Palazzo del Lavoro when the party is over" [40]. Despite the various solutions for a possible reutilization proposed by Nervi himself in the public tender of 1959, including for sports facilities [41], on the day after the exhibition closed there were already reports on the utilization of the buildings and works constructed for the centenary celebrations [42]. It was predicted that 3 billion lira would be required for the adaptation of the building in the absence of the initial project's requirements. Initially it was thought that the Italia '61 structures could become the headquarters of one or more education centers of high, middle and low-level professional training. In 1962 the Nervi office was entrusted with the project of transforming the building into a professional center. The cost of this operation turned out to be unsustainable, so the only option was to propose that the Palazzo del Lavoro should become the headquarters of the United Nations International Labour Organization so as to attract foreign capital into the city.

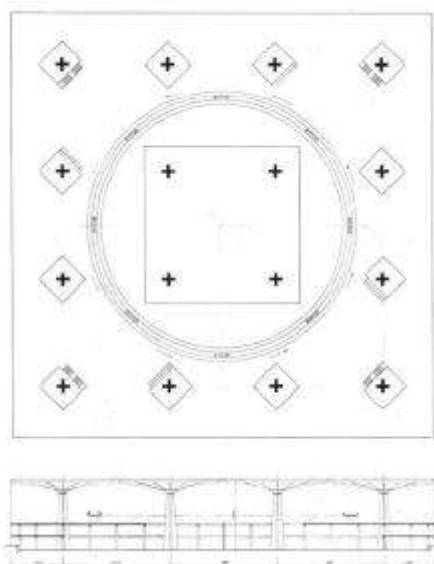


Fig.12. The proposal to reuse the Palazzo as a sports center designed by Nervi himself, 10 October 1959.

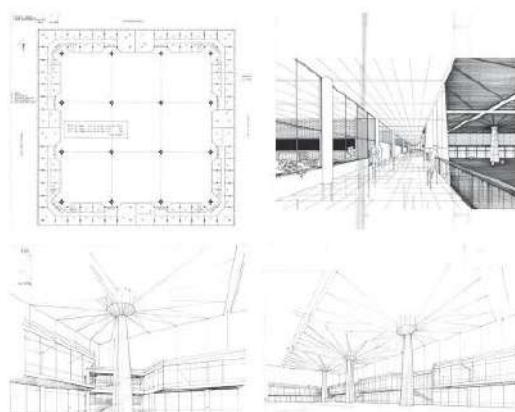


Fig.13. The proposal to reuse the Palazzo as a sports center designed by Nervi himself, 10 October 1959.

Thus in December 1963 the Ministry of Finance (owner of the building but not of the land on which it stood) entrusted the building to the Comune (municipality) with a convention of 19 years to make it the headquarters of the United Nations International Labour Organization which took up residence there in 1965 after important reconversion works [43]. The most important work was the construction of a second and third floor (partly based on the project by Studio Nervi with the executive project of the Fiat Constructions and Systems division). The staircases that led to the intermediate gallery were eliminated in favor of four groups of fixed staircases and escalators in the corners behind a wood panelling screen where the various systems were installed. Above all the roof panels in thin sheet steel had turned out to be unstable [44], and insufficiently insulated: these had to be substituted with prefabricated ferro-cement panels. Moreover the building required insulation work to be inhabitable (new doors and windows, double-glazing on the facade, wire mesh glass for the skylights, insulation and waterproofing of the roofing), with modular divisions (new porches at the ground floor entrances and interior movable partition walls), new technical equipments (false ceilings for cables along the isostatic ribbed ceilings of the first level, plumbing improvements, drink-ing water, fire security devices, hot water), and new air conditioning and fan coil units along the outside walls, convection heaters for the rooms facing inwards onto the hall, electric fans on the ceilings, underfloor radiant heating panels for the central hall covering the whole floor in Montecatini marble, and a new heating and cooling station. All this cost about 2 billion lira. At the beginning of the 1970s the City of Turin tried to utilize unsuccessfully a part of the Palazzo del Lavoro for the installation of a public sports complex including a swimming pool, sports fields and an athletics track. The United Nations International Labour Organization abandoned the building in the mid 1980s after constant increases in maintenance costs with problems of infiltration from the roof and leaks from the radiant panel systems. The building continued to be partially used by the Regional Administration Authority which installed the regional cartographic center, a vocational training center and the Giorgio Quazza Institute for Information Technology and Electronics there. During this period there was also the idea to convert the Palazzo into a conference centre [45], still using a Studio Nervi project for a cost of about 10°billion lira, but this was never realized.



Fig.14. The opening ceremony of the ILO, October 1965



Fig.15. The roof before (left.) and after (right) the addition of the cooling/heating pipes, 1961-1965.

The Region also abandoned the building, and in the mid 1980s the Palazzo returned to the State Property Office. Between the 1990s and the early 2000s it was utilized only partially by the Faculty of Economics and Business, and by the Central Police Headquarters. The years went by without there being a proper strategy for the building's use. In 2000 a feasibility study conducted by the Turin Polytechnic and coordinated by Professor Aimaro Isola was commissioned by the Province of Turin. The study proposed its conversion into a Science Center; then there was the hypothesis to make it the headquarters of the prestigious Egyptian Museum; and after that the idea was put forward to

make the Palazzo the headquarters of Fiat Engineering. However, none of the projects tackled the crucial problem, which was also the original defect: that it should be reintegrated into the urban fabric with a real collective use. The sheer volume of the building was a great stumbling block. Any conversion project had to take into account the problem of heating a volume of this size or whether to use the complex as an enormous covered square in which to build a series of independent volumes.

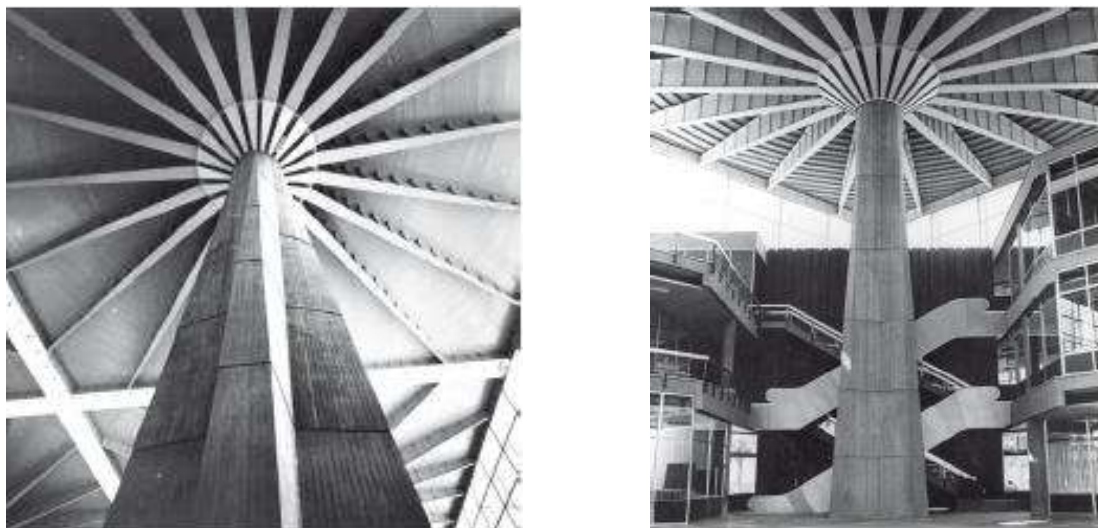


Fig.16. The roof before (left) and after (bottom) the substitution of the steel panes into prefabricated ferro-cement ones.

In 2004 the Turin Municipality decided to purchase it: the asking price was 20 million euros. However, the procedure was long and complex. Despite the Mayor's provocative proposal to have it wrapped up –literally – so as to hide it from sight during the 2006 Olympic Winter Games, the Palazzo del Lavoro continued its wretched existence as a sporadic exhibition hall. In October 2005 a variation of the zoning use clauses of the Town Planning Regulations was approved (which at that time specified its use as an exhibition space) into “new zoning use for tourism, culture, exhibition and the service sector, conferences, research, university, services for people and for enterprises”. This might be a way out, although there may be concerns about the broad spectrum of the uses permitted and proposals advanced. There is again the idea of using the building as an umbrella for smaller structures which, however, could still be six storeys high. In 2007 Fintecna, the estate agent of the Ministry of Finance, chose the ideal partners for a real estate portfolio, and the Turin estate agent Gefim was picked. The final variation of the Town Planning Regulations was approved in July 2009. It allows for a change in the zoning use of the Palazzo for commercial uses and for a considerable increase in its construction volume.



Fig.18. The central exhibition space.

Fig.19. The interiors with false ceiling covering the isostatic ribbed slabs

Moreover, it should bring into the Municipal coffers at least 3.7°million euros for development of the building and 8°million euros for the planning fees. It was only in 2010 that Pentagramma, a company composed of 50.% Fintecna and 50.% Gefim, was able to identify the final user: the Dutch group Corio which builds shopping malls using a pension fund. The project was presented in 2011 and for the urban planning aspect it was signed by the Turin architect Alberto Rolla who maintains that he intends to faithfully pick up the hypothesis developed by Nervi for the conversion of the building during the tender phase in 1959. In reality, however, his project proposes the construction of bulky internal volumes.



Fig.20. The original ceiling with isostatic ribbed slabs

In July 2011, when the building celebrated its 50th anniversary, it finally came under the constraints of the cultural heritage code. The constraint that was imposed referred, however, to the relational value of a commemorative event, which is heteronomous as regards the values of the work. It is not clear why it was decided that the Palazzo shouldn't be protected for its intrinsic value [46]. In the report the salient features of the internal space and the architectural concepts in relation to the structural aspects are not indicated. This in effect prevents any specific limitation by the Soprintendenza (government department responsible for the environment and historical buildings). Hypothetical limitations would include, among other aspects: the height of any volumes, which should not exceed that of the perimeter gallery; their arrangement and the amount of space they occupy; and their distance from the pillars so as to respect the internal perspective space defined by the succession of pillars and by the interplay of light. As the building is now private property, the Soprintendenza can give its approval on the basis of a preliminary project without acting as tutor throughout the project process. The public administration is the only authority remaining that can impose the protection guidance that would result from this work. At the beginning of 2012 the city of Turin set up a specific committee for the refurbishment but this was done more to respond to the inhabitants' perplexity over the building of yet another shopping mall than to protect an architectural monument of the 20th century, to the extent that only local politicians sit on this committee, with the owners, and no architects.

In June 2012 the regional administrative court blocked the project following a complaint [47], by the company that manages the shopping mall in the nearby ex-Fiat factory Lingo³o, an icon of modern movement built between 1915 and 1922 and transformed by Renzo Piano Building Workshop into a multipurpose center in the 1990s. The project for the Palazzo was therefore suspended. Being a celebration of Italian technique and know-how admired around the world the building was likely to become a symbol of immobility. Suddenly in August 2015 the southern part of the building was devastated by fire. From this fire, the municipal authority has made a commitment to accelerate and conclude within one year the approval process. In December 2015 the planning agreement was finished, and it was approved by the Turin City Council in mid-January 2016: the transformation of the Palazzo was then scheduled for completion by fall 2018, but then always postponed for regulatory and economic reasons. No transformation is documented until the beginning of 2019. The failure to fully utilize the Palazzo due to its high maintenance costs is a reminder of how policies relating to major events may fail if they are not connected to the real economic and social requirements of the city, despite in this case the innovative formal and technological qualities of its architecture.



Fig.21.The Palazzo in 2017

Reference

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- [2] On Pier Luigi Nervi, cf. Olmo, Carlo, Chiorino, Cristiana (Eds.), *Pier Luigi Nervi. Architecture as Challenge*, Silvana Editoriale, Brussels 2010; Gargiani, Roberto, Bologna, Alberto, *The Rhetoric of Pier Luigi Nervi: Concrete and Ferrocement Forms*, EPFL Press, Lausanne 2016; Bianchino, Gloria, Costi, Dario (Eds.), *Cantiere Nervi. La costruzione di una identità. Storie, geografie, paralleli*, Skira, Geneva-Milan 2012; Iori, Tullia, *Pier Luigi Nervi*, Mo.a Archite.ura, Milan 2009; Greco, Claudio, *Pier Luigi Nervi. Dai primi breve_i al Palazzo delle Esposizioni di Torino 1917-1948*, Quart Verlag, Luzern 2008.
- [3] Mario, Nervi's son, remembers how his father loved to compare each building process to a three-legged table whose weight has to be evenly distributed over the three legs (client, architect, builder), in the sense that if one of the legs gives way, the equilibrium of the whole table is compromised; cf. *Pier Luigi Nervi e la sua opera: incontro di studio organizzato dal Comitato del Premio Ingersoll Rand Italia*, Circolo della Stampa, Milan 1980.
- [4] Italia '61, in "L'Archite.ura, cronache e storia", 70, 1961; Il palazzo del Lavoro, in "Architectural Record", 3, 1961; Il palazzo del Lavoro, in "Progressive Architecture", 11, 1960; Il palazzo del Lavoro, in "Architectural Review", 765, 1960; Nervi, P. L., *Dentro l'immane stru_ura*, in "Domus", art. cit.
- [5] *Le Corbusier to Nervi*, postcard of 25 May 1961 and answer of 1_June 1961, MAXXI, Archivio Pier Luigi Nervi.
- [6] On Italia '61, Chiorino, Critiana, Pace, Sergio, Rosso, Michela, *Italia . '61. Identità e miti nelle celebrazioni per il centenario dell'Unità d'Italia*, Allemandi, Turin 2005.
- [7] The proposals for future uses ranged from the Egyptian Museum, exhibition headquarters for products produced by Turin industries to a hall for shows and sports displays, and to a supermarket, Archivio Storico della Ci.à di Torino (ASCT), Fondo Comitato Torino 61 (FCTO61).
- [8] Call for tenders and related documents, ASCT, FCTO61.
- [9] *Idem* and *I proge_i vincitori del concorso per il Palazzo del Lavoro a Torino*, in "Casabella continuità", 235, 1960, pp. 33-34.

[10]“[...] I want to see you in that red-hot glass cage baked by the summer sun even if you put all the brise-soleil you want and which Nervi has not provided... I want to see whether that monumental Assyrian (structure) pleases Agnelli! And I want to see the final price, and how they will sort out the schools at the end of the exhibition, which Nervi has not even dreamt of solving in the teeth of the tender requirements! [...] It's up to you to make do in the midst of those four great columns that Nervi has dumped on you!”, *Letter from Carlo Mollino to Gio Ponti*, 29/1/1960 and *Notes for the appeal, December 1959*, in Archivio Mollino, Biblioteca Centrale di Architettura (BCA), Politecnico di Torino.

[11] All the drawings are at the Centro Studi Arte Contemporanea (CSAC), Parma, Fondo Pier Luigi Nervi.

[12] Cf. Nervi, Pier Luigi, *Architettura strutturale con riferimento al Palazzo del Lavoro* in “A.i e rassegna tecnica della Società degli ingegneri e degli architetti in Torino”, 6, 1961, pp. 165-190.

[13] Cf. Pace, Sergio (Ed.), *Pier Luigi Nervi. Torino, la committenza industriale, le culture architettoniche e politecniche italiane*, Silvana Editoriale, Milan 2011.

[14] Born in 1892, Gino Covre graduated in Milan in 1916. He moved to Rome and was already specializing in steel structures before the war. In 1937 he had taken part in the EUR arch project, which was not realized, proposing a solution in aluminium alloy as an alternative to Libera's project, on which instead Nervi & Bartoli worked with a solution in ferroconcrete and another in concrete segments without iron reinforcement, according to a procedure conceived by Pier Luigi Nervi himself. Together with Eduardo Viora, Covre had also designed the Stabilimento della Officina Meccanica Olivetti at San Bernardo in the 1950s. Covre worked with Nervi on the suspended structure of the Burgo paper mill in Mantova between 1961 and 1963; cf. Covre, Gino, *Il Palazzo del lavoro all'Esposizione Italia. '61 di Torino*, in “Costruzioni metalliche”, 2-3, 1961, pp. 141-144.

[15] Cf. Notebook published in 1961 by Ufficio Italiano per lo sviluppo delle applicazioni dell'Acciaio (Italian office for the development of applications in steel).

[16] The role of the Fiat Works Management, which had already worked with Nervi for the halls of Torino Esposizioni in 1948-1950, was fundamental, as can be deduced from the number of tables and charts present in the Archivio Servizio Costruzioni e Impianti Fiat which later became Fiat Engineering (AFE, rag. 0084, opera 004, *Palazzo del Lavoro*).

[17] The final drawings by Nervi are dated 22 January 1960. The request for building permission is dated 29 January 1960, Archivio Edilizio della Città di Torino (AECT).

[18] The basement floor planned by Nervi would not have provided sufficient illumination for the subsequent school that was planned: “The examining committee recommends that the possibility of considerably raising the floor of the building should be studied, both for the building and for the green area”, BCA and AFE.

[19] Nervi's project was particularly detailed in the structural part but completely lacking as regards the systems, which were developed by the Fiat Constructions and Systems division. After various solutions, in February 1960 an air conditioning system was chosen with a network of large air supply channels on the roof, a floor return air system and a radiant floor, a perimeter channel outside the building containing the air pipes and the pipes to supply the air vents and convector heaters of the future Professional School. The main heating system was set outside the building, AFE.

[20] Nervi had won with his bid of 1,64 billion lire, excluding heating and systems, which would then reach 1,88 billion lire. The total cost amounted to 2,72 billion plus 648 million for the work subsequent to the celebrations. To this sum should be added one billion for the design of the exhibition according to Ponti's project. Cf. AFE, Archivio di Stato di Torino (AST), Fondo Comitato Italia 61 (FCIT61) and ASCT, FCTO61.

[21] "It is thanks to Mr Ponti, Nervi's devoted friend, that in Turin the structure by our Master has been able to appear in all its beauty and in the isolation of its elements. It is thanks to myself that this gallery has been eliminated, taking advantage of the time constraints Nervi was allowed for the execution, and namely that the 16 columns have been brought to the fore, which is the pure beauty of this building", *Le_er from Gio Ponti to Cesare Merzagora with copy to Pier Luigi Nervi*, 7/6/1961 MAXXI, Archivio Pier Luigi Nervi. See also, *Le_er from Gio Ponti to Giovanni Agnelli with copy to Filiberto Guala, Pier Luigi Nervi and Vi_orio Bonadé Bo_ino*, 2/11/1959, Milan, Archivio Gio Ponti (AGP) and *le_er from Gio Ponti to Arrigo Castellani*, 15/5/1961, Milan, AGP.

[22] Cf. *Le_era di Pier Luigi Nervi al presidente del Senato On. Cesare Merzagora*, 26/6/1961, AGP.

[23] "Everything in your pavilion is aesthetically coherent and also the glass joints overhead between one element and another is a delight in its purity. What it is necessary to avoid in the first appearance of your work is that row of large hanging lamps, which jar with the rest. As I shall not be using them you really must have them removed, for the purity of your creation, then if necessary they can put them back later on", *Le_er from Ponti to Nervi*, 2/12/1960, MAXXI; "With this aberrant light and with that (equally aberrant) perimeter lighting the capitals and the entire ceiling will look black and flat against the light, losing all their plastic value", *Le_er from Gio Ponti to Pier Luigi, Antonio and Mario Nervi*, 11/4/1960, AGP.

[24] Among the causes for the increase in price compared to the solution provided in response to the call for tenders was the increase in the glass surface and the choice of the moulded uprights solution instead of that of beam uprights.

[25] Zevi, Bruno, *Degni seguaci di Guarini e Antonelli*, in "Cronache di archite.ura", 288, 1978.

[26] Antonio Nervi initially opted for brise-soleil in aluminium, then he fell back on steel for reasons of costs. The executive drawings were realized by the Fiat Constructions and Systems division which dealt with the inclination studies, AFE.

[27] Maintenance of the glass wall was provided for with crowbars running along a monorail situated in the cavity between the brisesoleil and the glass wall. Window and door frames, uprights and brise-soleil were painted with gloss finish martinorint of a color called London smoke grey, AFE.

[28] Semi-double glazing was used for the steel laminate door frames, with 5_+_6_ millimeter plate glass for the gallery frames in light alloy, excluding the mirror effect below which uses an insulation panel and tempered glass, 8-9 millimeter plate glass for the main glass walls, with wire mesh glass for the skylights. Opaque rolled figured plate glass was used for the cleaning service areas; cf. *Elenco di_e Palazzo del Lavoro*, in "L'Archite.ura, cronache e storia", 70, 1961.

[29] Cf. *Riunione del 12 gennaio 1960 presso la Divisione Costruzioni e Impianti della Fiat*, AFE.

[30] Four prefabricated wooden shacks were placed on the worksite for the Works Management and Site Management; a varying number of prefabricated shacks were on site to recover various materials and for site requirements; there was a small experimental laboratory for testing the concrete and for granulometric analysis of the grades (of sand and gravel); an area for metalwork and one for carpentry: *Pianta degli scavi e sistemazione cantiere*, 31_December 1959, CSAC and *Palazzo del Lavoro, Planimetria sistemazione di cantiere*, 4 March 1960, AFE.

[31] The foundations stand on 500-millimeter-diameter Franki piles. The ground has a 5-meter-layer of landfill, a 5-meter-layer of sand and loose silt and a 20-meter-bank of sand

and gravel. The pile heads are about 30_meters deep, *Esecutivi Palazzo del Lavoro, Programma dei lavori* in c.a., AFE.

[32] The total time was about 50_thousand working days. On the site were three pile drivers used for the Franki piles, four tower cranes, five mobile cranes, one bulldozer, two dumpers, and one concrete mixing centre with a daily production of 100_m3 of concrete with compressed air systems to transport the concrete.

[33] The work site is documented with pictures conserved at the MAXXI and at AST.

34 Cf. *Esecutivi Palazzo del Lavoro*, Production phases of the pillars and assembly plan of the metal roof and mobile deck pouring of concrete floor, July-October 1960, AFE.

[35] Just the formwork cost over 10 million lire: the cost was written off as it was reutilized for all 16_pillars.

[36] Cf. Bossi, W., *Nel cantiere del Palazzo del Lavoro*, in "Notiziario Italia 61", 1, 1960, pp. 11-12; Pavia, N., *Centinaia di operai, mezzi potentissimi. Nell'enorme cantiere si lavora giorno e notte*, in "Notiziario Italia 61", 4, 1961, pp. 4-9.

[37] Gio Ponti to Nervi, 12 November 1960 and 28 November 1960.

[38] Buzzati, Dino, in "Corriere d'informazione", 7-8 June 1961.

[39] Cf. Berto, Paolo, *Torino nel cinema: l'identità imperfetta*, in Luigi Mazza, Carlo Olmo, (Eds.), *Architettura e urbanistica a Torino 1945-1990*, Allemandi, Turin 1991, p. 170; Portis, Benjamin, *Images de villes idéales.: les expositions universelles*, Centre canadien d'architecture, Montreal 1993.

[40] Zevi, Bruno, *La dissociazione architettonica, tara delle Esposizioni*, in "L'Architettura, cronache e storia", 70, 1961, p. 219.

[41] In the drawings deposited at the CSAC there is a drawing for a possible use for education with classrooms and a series of hypotheses for a sports center.

[42] Cf. Speech by Engineer Giacomo Bosso in the *Comune*, 2_December 1961, in "L'Unità", Sunday 3 December 1961, p. 4.

[43] Nervi also built the headquarters of the United Nations International Labour Organization in Geneva (1969-1973).

[44] Cf. Minutes of the Technical Consultancy Commission for the Committee for the International Centre for Professional and Technical Development, ASCT.

[45] Project by *Studio Nervi* which contains interesting notes on the degree of flexibility of the structure for different uses, ASCT.

[46] Using art. 10, comma 3, letter a, of the Codice dei beni culturali (D. Legge n.42/2004).

[47] The complaint, confirmed in May 2013 by the Council of State, focused on the change to the Town Planning Regulation that was approved as a simple variation and not as a structural one with strategic environmental impact assessment and on the evident violation of the standards (from 6,000m² of commercial area to 13,000m²).

**110 YEARS SINCE THE BIRTH OF THE GREAT BULGARIAN ENGINEER
ASSOC. PROF. DR. ENG. BORISLAV B. BANKOV AND HIS STEEL BRIDGES**

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Abstract: *In this paper the author present a short autobiography of the great Bulgarian engineer and lecture assoc. prof. Dr. Eng. Borislav B. Bankov. It is given short information about his education and scientific works in Brno and Darmstadt. Attention is drawn to six realized steel bridges projected himself.*

Key words: *education, 110 years anniversary, steel bridges, autobiography*

1. Introduction

Assoc. Prof. Dr. Eng. Borislav B. Bankov(Fig.1) was born on 26 September 1912 in a family of teacher in Kneja, a small Bulgarian town. He finished school in Knezha-Bulgaria-1932. In 1940 he graduated civil engineering in Brno, Czech Republic. He was assistant of Prof. A. Hawranek at the department of "Steel bridges and constructions". Till march of 1942 he worked in engineering company of Prof. Kleinlogel in Darmstadt, Germany. Between April 1942 and August 1944 he was assistant of Prof. K. Klöppel at the department of "Static, Steel bridges and constructions". At the time he completed his promotion and published his book "Anwendung der β -Belastung bei der Berechnung von Einflußlinien". After that he returned to Bulgaria and taught in different universities in Sofia, Russe and Varna. In 1968 he began to construct bridges. Till 1972 he constructed various kinds of steel bridges near Varna. In 1968-69 he constructed the first welding railroad bridge in Bulgaria approximately 400 m. long. In 1975 he constructed and managed the building of a road and railroad bridge near Varna. Prof. Bankov was awarded a medal for his great projects in 1976. In 1977 he constructed the steel highway-bridge "Bebresh" - 740 m. long. In the same year he made a special steel construction-168 m, long for the building of a concrete road-bridge. In 1980 he was supervisor of a steel bridge in Elin Pelin. The elements of this bridge were made in formerly Yugoslavia. In 1981 he constructed a special steel system for the renovation of a road-bridge 160 m. long. 1983 he constructed and managed the building of 90 m.-long bridge near the airport of Sofia. For this project he was awarded a prize. In 1984/86 he constructed and managed the building of compositesteel-concrete bridge near Varna. In the period 1984-86 he made a project for a bridge near the village of Roglez with parts from an old steel bridge.

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Fig.1. Dr. ing. B. Bankov(1912-1992)

2. Structure, Design and Construction of a Steel Scaffold Bridge in Sofia

2.1. Structure

In the beginning of the 90's, in Sofia, the capital of Bulgaria arose the necessity for a long bridge connecting the center of the city to the airport[1]. For this purpose a steel-reinforced concrete bridge was designed and constructed with a total length of 2114 m, a width of 21.5 m and a height above the ground of about 10 m. The bridge is located in a densely populated urban environment. This important, effective and aesthetic bridge serves as a connection between the busiest input-output highway and the airport of Sofia. It ensures a convenient access to some highways, railways, streets and the airport. As a result of this requirement, the erection of the reinforced concrete bridge is not allowed to cross the railway between Sofia and Varna. So, a 90 m long middle steel part was designed to cover two spans of 45 m each in the multi-span bridge structure (Fig. 2). The steel superstructure in the form of an open steel cross-sections (or boxes, alternatively) are prefabricated in the shipbuilding plant “Stoyan Pavlov” in Varna. The parts of the steel structure were cut out by a CAD/CAM system according to the final detailed drawings. Full corrosion protection was provided at the construction site. The structure of the bridge was divided into separate sections (Fig. 3) with such dimensions and weight that they could be transported by trucks about 500 km from Varna on the Black Sea coast to Sofia[1].

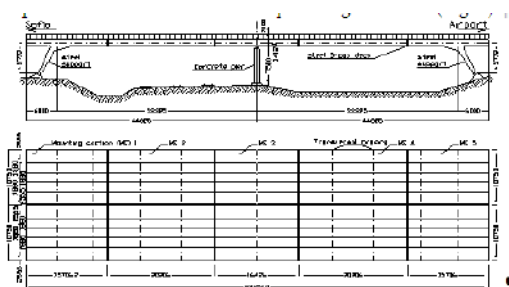


Fig. 2. Longitudinal cross-section



Fig.3. Triple-box section of the bridge deck

2.2. Construction technology

The r/c foundations are initially cast-in-situ. The steel superstructures are assembled by a crane using prefabricated elements. In longitudinal direction 8 mounting sections are formed. In transverse direction 5 mounting deck sections for each of two decks are produced. The supports and columns are initially mounted to bases. Then the deck elements are lifted and joined to each other. High-strength bulgarian steel and welded junctions are implemented. The girders were taken to the construction site where they were lifted into their final position with a mobile crane “Demag”, (Figures 4 and 5).



Fig. 4. Assembly of the box girders

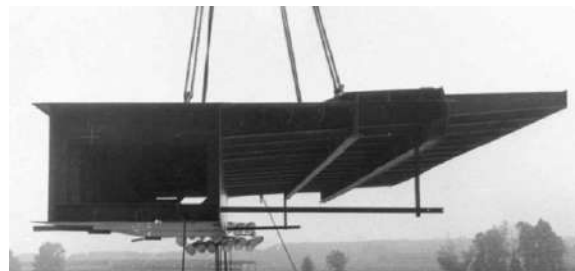


Fig. 5. Composition of a separate box girders

Each deck has a roadway of 7,5 m width with two one-way lanes. The steel decks are stiffened by longitudinal and transverse ribs. Transverse diaphragms over the supports and between them are also constructed. Some parts of the two steel decks have different and variable curvatures and crosswise slopes. A dividing longitudinal middle gap is available between adjacent decks. The V-type supports are rigidly connected to the decks. The r/c supports are also rigidly linked up the foundations but the steel supports are tied by cylindrical hinges to the bases. The two middle vertical r/c columns of the steel bridge part has a rectangular variable cross section and they are bound by elastomeric bearings to the steel decks but by rigid couplings to the foundations. The adjacent V-type supports as well as the columns of the two decks are connected to particular r/c foundations (Fig. 6 and Fig. 7).



Fig. 6. Connection of the “V”-support to the lower orthotropic steel deck



Fig. 7. Assembling of the steel “V”-supports

2.3. Analysis and design

For the final detailed design in the early 1982 it is computed by refined linear elastic spatial frame models and the finite element method using the computer program STRUDL. The structure is designed for all loads and models during its assemblage and operation. Additional manual computations of some elements are then accomplished taking into account several important peculiarities. The curvature effect on the strains and internal forces is considered. The restricted and pure twisting of the box-type thin-walled steel and r/c girder decks are then regarded. The inclined supports with variable cross sections are computed by their deformed schemes under eccentric compression. A parabolic constitutive law of the concrete but the Prandtl's law for the steel are assumed. The bearing capacity of the critical sections in all phases is ensured. The technical tests proved the high quality structure. The bridge was put into operation from 1983. The construction costs was restored. The traffic in this urban region is improved.

2.4. Summary

The steel orthotropic bridge over the railway Sofia-Varna is an example of an effective structural method for medium span bridges. It is applicable for spans of 20-60 m considering that the prefabricated steel components are transported by road. To achieve of design versatility with the use of steel as a building material, a visually attractive structure could also be created. Since the region of Sofia is an active earthquake area, seismic codes had to be considered. However, the structure was proven to be ideal for earthquake regions. The superstructure with small mass, together with the inclined supports and columns formed a structure with excellent stiffness. A high level of precision was required during the process of construction, since much of the work had to be carried out in advance under the factory conditions. The technical tests proved the high quality of the structure. This important and complex bridge has been operating successfully about 40 years (Figs. 8 and 9).



Fig. 8. A view of the concrete and steel bridges



Fig. 9. A view from beneath the steel orthotropic bridge

3. Large-Span Steel Travelling Spatial Trussed Girders for Assembly of Precast Reinforced Concrete Bridge Beams

3.1. Structure of the equipment

The installing structure consists of two particular main spatial longitudinal parallel identical frame or trussed girders axially spaced in 5,6 m. Each of them has a total length of 168 m, 4,25 m in height 1,2 m axially in width and its weight is of 210 t (Fig. 10a,b)[1].

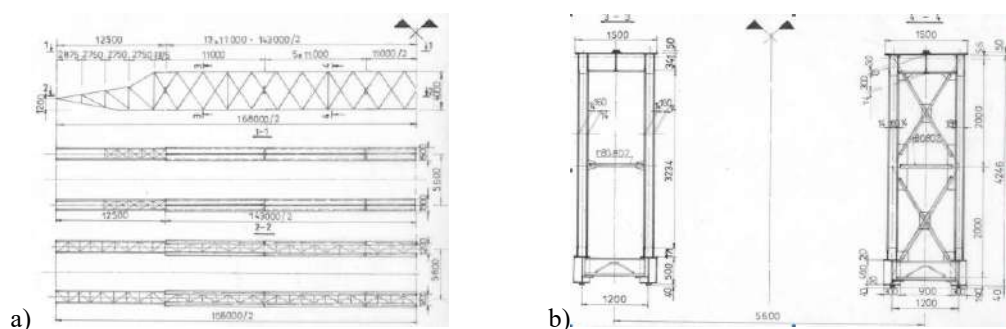


Fig. 10a,b. Overall view of the steel trusses – a); Cross-Section 3-3 and 4-4 of the two Trusses-b)

Every girder is composed from 15 separate prefabricated units (mounting blocks) in length - 13 inner uniform with a length of 11,0 m and a weight of 18 t and 2 outer non-uniform (the upper and the lower chords are inclined as a belaying cleat or as an fork arm) of 12,5 m long and of 21 t heavy. The units are made up in an automated factory for prefabricated steel structures. The whole equipment and each of its two girders have one longitudinal and one cross vertical plane of symmetry. Each lattice girder is made up from I-type steel rods. It is composed from 4 longitudinal horizontal parallel beams (excepting the outer blocks where the top and the bottom beams are inclined to the horizon) and discretely located vertical beams (left and right), cross horizontal bars (upper, middle and lower) and stiffening diagonal members (bracing struts) in all four side surfaces and within some cross sections also. A continuous upper plate of 1,5 m width and a lower plate of 1,2 m width are constructed also. They have a considerable stiffness in their horizontal planes necessary to bear the vertical and horizontal loads including lateral impacts during the movement of the upper installing and lower adjusting rolling trucks. A lower narrow-gauge double-track railway is welded on the bottom side of the longitudinal beams of each of the two girders. It serves to roll the equipment on four special steel underlying mechanical trucks (two by two under each trussed girder). These moving adjusting trucks can roll by 8 lower wheels on the two parallel cross double-track underlying railways situated at the side or on the r/c abutment in the initial adjustment process but attached on the cross cast-in-situ reinforced concrete bearer frames between and over the bearings on the bridge column heads(Fig. 10c,d) in the following operation or adjustment phases. The frames are equally high to the precast deck beams. These rolling trucks enable the equipment to move in the bridge length by rolling on 8 upper wheels and across the bridge as well to rotate in horizontal and in vertical planes also.

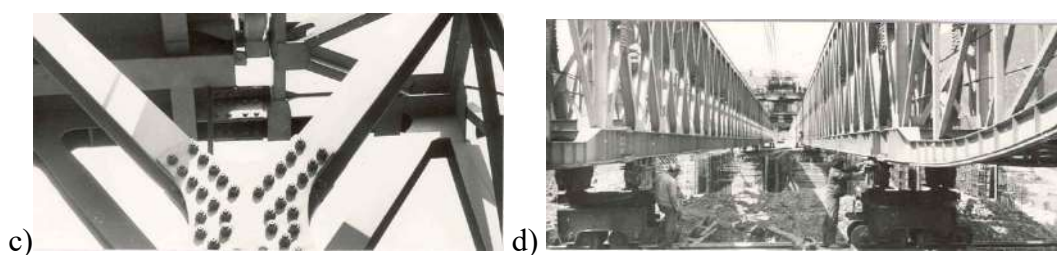


Fig.10c,d. High-Strength Bolt Link Between Two Mounting Blocks of the Equipment Structure-a); An Installing Equipment, Two Lower Rolling Trucks and Cross Rails Under Them-d)

During the movement, adjustment and operation the individual frame (truss) girder of the blank or loaded equipment can be supported either as a cantilever or to be simply supported on two consecutive columns as well to be supported as a continuous beam on three columns (bridging two spans) without or with one or two cantilevers depending on the position of the bearings. During the movement of the equipment on the lower adjusting rolling trucks, attached to the r/c frames on the bridge column heads, the bottom longitudinal chords are subjected both to axial and bending loads due to the direct action of the wheels of the trucks. This local action is large. Therefore the bottom longitudinal beams are designed and constructed stronger than the upper ones and they are strengthened by densely located short vertical ribs and by a horizontal plate too. A single rail on the upper plate of each of the two girders is located in the vertical longitudinal axis of this plate on a longitudinal rib endured by cross bars. The last members withstand the loads from the elevating two installing motor trucks and transfer them at the girder joints. In this way the upper chords are subjected to axial loads mainly and they are relieved. The lifting and pulling electrical mechanisms are installed on these trucks. The trucks move on the upper broad-gauge double-track railroad the rails of which are placed on the two particular equipment girders. The weight of the trucks is about of 40 t. The total weight of each equipment frame is of 230 t (about of 1,4 t per meter) and the total movable load on it is up to 260 t. The upper plates of the equipment are connected only by two cross members at the ends (just to the connections between the outer fork arms and the first inner units) of the two girders. These members serve also as stops of the movement for the upper trucks.

3.2. Conclusions

The rational installing structure and the mounting technology are proposed, elaborated and applied successfully. The structure of the installing equipment, the lower adjusting rolling trucks, the upper installing rolling trucks and the installing technology are technical novelties and inventions. These unique structure and technology are very effective, reliable and fine(Figs.10e,f).



b)



Fig.

Fig.10e,f. Overall view of the steel trusses – e); Cross-Section 3-3 and 4-4 of the two Trusses – f)

Before using the gantry truss for transporting the 60m bridge girders with 220ton weight Bulgarian Road Agency[2] performed series of load tests to prove its load bearing capacity and compare the structural behavior with the project developed. Under load and support conditions in accordance with schemes SH 1.1, SH 2.1, SH 2.2 and SH 2.3 vertical displacements of the particular measuring points are measured by means of Maximov's deflecto-meter and with precise geodetic level measurer "KONT".



Fig.10g. 3D FEM of the double twin gantry truss with 168m length

For the comparison of the results with the developed by the Bulgarian Road Agency tests and calculations a 3D FEM[2] of the gantry truss is developed with SAP2000 software – Fig.10.g. The connection between elements is assumed as rigid.

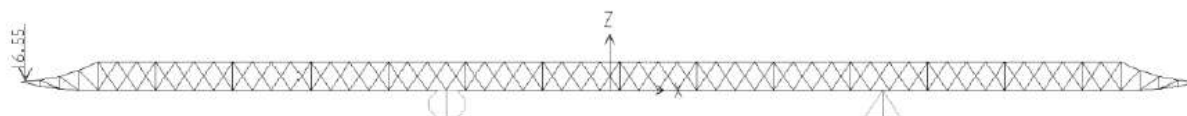


Fig.10h. Support conditions and loads for scheme SH 1.1. ($F=16.55\text{kN}$).



Fig.10i. Support conditions and loads arrangement for scheme SH 2.1. ($F=680.81\text{kN}$).



Fig.10j. Support conditions and loads arrangement for scheme SH 2.2. ($F=680.81\text{kN}$).

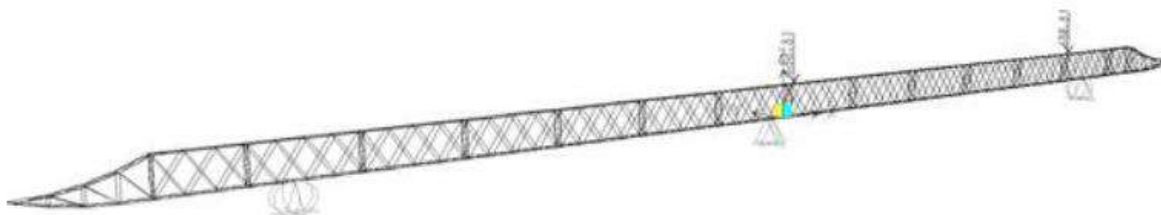


Fig.10k. Support conditions and loads arrangement for scheme SH 2.3. ($F=680.81\text{kN}$).

4. Design and Erection of Composite Road Bridge

4.1. Bridge technical details

The road bridge throws over railway line Sofia-Varna and sewer for central heating of Varna[1]. The railway traffic between the capital Sofia and the big town Varna is very intensive. It was very important the erection of the bridge not to break the railway traffic. The situation-plan gives the opportunity for using an internal support which divides the upper construction into two equal spans. Two variants are examined: continuous beam on three supports and two simply supported beams. After analyzing advantages of the two variants is chosen the second one. The upper construction consists of two identical simply supported beams with a span of 44 m. The cross-section is composed from four steel beams which are connected by a ribbed bottom and unified along their upper flange with a reinforced thick 20 cm concrete prefabricated slab (Fig.11a,b). The intermediate trusses are formed of L-profiles and are at intervals of 10 m. The effective width of the Bridge is 9.5 meters providing two lanes of traffic. Box width is 2.7 meters.

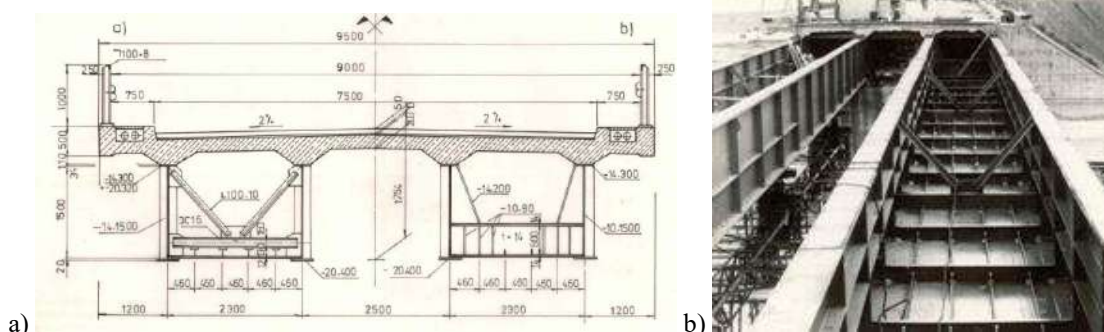


Fig.11a,b. Construction of the cross-section

4.2. Design consideration

The upper construction of the bridge can be treated as a composite prismatic shell. The static calculations include all actions of dead load, live load, wind pressure, temperature effect, erection forces and effects, seismic forces, creep and shrinkage effects, forces and effects due to removing of temporary supports. The bridge is investigated as a statically determinate composite steel-concrete beam. The composite action between steel girders and concrete slabs is ensured by means of grouped headed stud connectors at intervals of 0.5 m. The studs are welded to the beams by equipment CROMP-ARC. They are situated in precast slabs. The studs are with diameter 19 mm and height 120 mm. The prestressing of the steel beams is realized by four temporary supports for each beam elongated at distance $L/5$ one

from another. Each support bears 500 kN hydraulic jack. The separated beams are loaded by the jack with 100 kN direct upward force resulting in reverse elevation of the bridge (Fig.12a). The bridge was put into operation from 1986(Fig.12b).

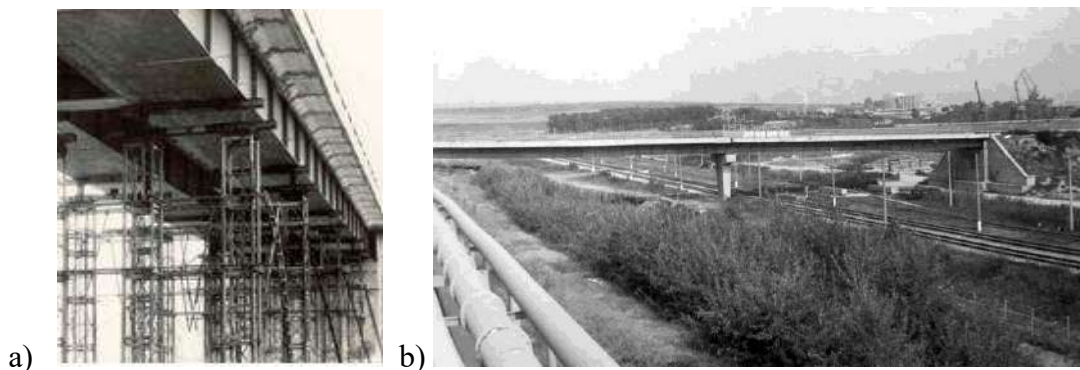


Fig. 12a,b. Reverse elevation of the structure-a); Inside view of the bridge-b);

5. Reconstruction of the 80th Old Steel Bridge Structures and their Implementation in a Road Bridge and Conditions after 35 years in Exploations

5.1. Construction of old bridge

The steel constructions were made at the end of the XIX century[1]. They consist of three separate steel truss girders of the simple beam statically system. The main girders of the 30.80m. span old bridge is with a parallel upper and lower chord. Each of them is a bi-construction of two main steel trusses, connected between each other at the nodes of the upper chords with horizontal cross web beam and vertical X-type bracing, situated under the horizontal beams. (Fig.13). A lower and upper wind bracing connected the two lower chords and the two upper chords of the main girders. The height of the every truss girder is 3170mm. every truss is divided into 10 fields by vertical struts. The distance between verticals is varying from is 3080 mm. The diagonals are extended over one field, and are inclined to the middle of the bridge. They have no bending stiffness and are therefore designed only for tensile forces. This is a reason, why the diagonals are crossed in the middle in two fields of the bridge. The distance of the main girder axes is 4,0m. Between the cross web beam, four horizontal longitudinal truss girders are situated. They are with parallel chords and crossing diagonals, so that there appear chords were in the plane of the cross girders. The height of these beams is 550mm. All elements of the trusses are made of angles and flats connected by rivets. The bridge structure is made of wrought iron. All parts were assembled and riveted together on site. It is interesting to note that this type of bridge was common at that time. Detailed instructions and advice for design and construction for this type of bridge are giving in [1].

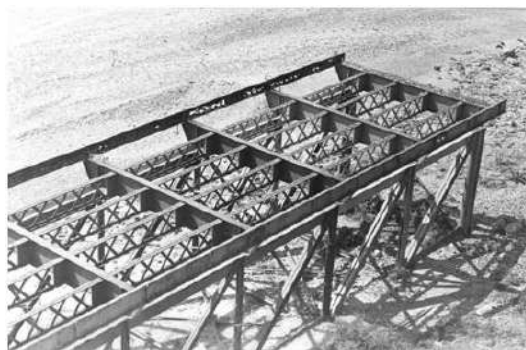


Fig.13. View on disassembled steel bridge construction

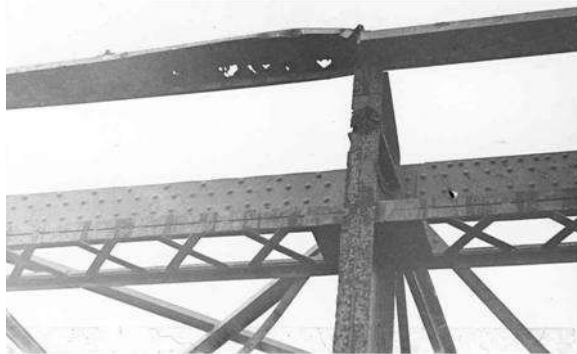


Fig.14. View on strong damage, due the corrosion

5.2. Condition of the structures

The condition of the disassembled bridge was detailed inspected, before the design stage. the main problem was corrosion of the steel parts. additionally, it was known that salt, had been used on the road surface more than 70 years. the structure had many details which needed a very efficiently method for inspecting and repairing. in some places, the damage seemed to be very bad (severe). the steel flanges had started to peel and there was no easy way, how to stop this process. also, some of the cross and longitudinal beams the upper and lower chords, knots plate, wind bracing were strong damage, due the corrosion and as a result of a incompetence by disassembling of structures loading and unloading and its transportation to the river "Lom". (Fig.14).

5.3. Concept of rehabilitation works

The rehabilitation concept consisted of:

Analysis of the structure and increasing the structural capacity of the steel truss, according to the present design code; Replacement of the deteriorated steel elements; Building of a modern composite steel- concrete road plate; Widening of the width up to 9,0m; Repainting of the whole steel structures; Repairing of the damaged structures and accessories, including the stone guard of piers

5.4. Conclusions

At the beginning of the 1980 the Ministry of Transport in Bulgaria give instruction for the disassembly of the steel construction of a road bridge over the Jebel river (near the town of Kardjaly). After a 70-year exploiting these old steel constructions were transported in May 1982 to Rogletz village for the execution of a diversion over river Lom of road III 564, leading to Rogletz village near the town of Vidin, which lay on the river Danube. The river Lom is a right tributary of the Danube in territory of Bulgaria. As a results of the analysis the upper and lower chords of the main girders, the cross girders, the longitudinal girders, the lower and upper wind bracing not proposed to be strengthened. This rehabilitation work was carried out in the time 1982-1987 [1]. It was possible to strengthen the old road bridges from the beginning of 20 century to withstand the higher loads of actual standards. The historical appearance can also be maintained (Fig. 15a,b). A practical case is presented where

rehabilitated steel structures are used for construction of a new bridge. The steel constructions were made at the end of the XIX century. All elements of the trusses are made of angles and flats connected by rivets. The condition of the disassembled bridge was detailed inspected, before the design stage. For the repair and rehabilitation of the old steel structures was applied the European practice. The new road deck was chosen to be precast concrete plate. A team of experts monitored the new bridge, which is in operation for 35 years, and concluded that its characteristics are according to the standards(Fig.15a,b).

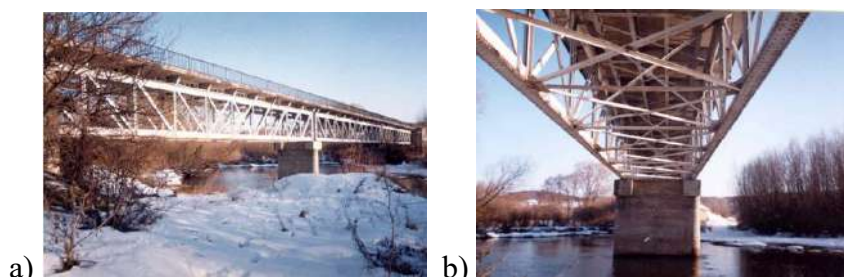


Fig.15a,b. Side of view on new rehabilitation bridge construction-a); View on bottom surface with wind bracing-b);

6. The Rehabilitation of small Asparuhov Bridge in Varna – Design and Construction

6.1. Introduction

In January 1975 Russian ship “Pavel Postishev” hit the lifting machinery of the movable span causing an accident that completely destroyed the movable part, and seriously damaged the two adjacent segments[1]. The situation was temporarily handled by constructing two military pontoon bridges, replaced two weeks later by two decommissioned steel bridge structures. Under these circumstances the design of a new combined road-railroad bridge with a movable middle section began under the leadership of Assoc. Prof. B. Bankov, Ph.D, [1]. It was proposed that two of the three damaged segments at the Varna side be replaced by new steel structures. The central span which used to be self-raising, was now subject to a limitation of a self-weight of 95 t. to allow lifting it by a 100 t crane every evening. The movable part was to be permanently fixed on completion of the big Asparuhov bridge, whose construction was under way at that time. After feasibility studies on several alternative solutions, the final conceptual design included a solid-walled steel structure between the abutment at the Varna side and pier 2, and a steel truss spanning from pier 2 to pier 3. The rest of the bridge was kept in its original state, i.e., a riveted steel structure between piers 3 and 4(Fig 16).

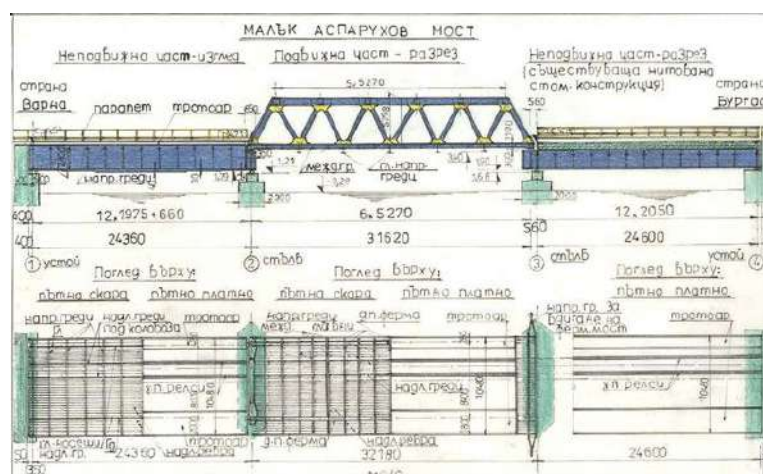


Fig. 16. The new bridge – plan view and side view

6.2. Stationary and movable parts of the bridge

The stationary part at the varna side was designed as a solid-walled simple beam with span 24,360mm measured between the bearing centres. the cross section shown in (Fig. 17a), is a classical example of a steel bridge with an orthotropic deck. the width of the bridge between the parapets is 10,480mm, with the two main girders placed at 8,000mm centers. the main girders are stiffened by cross-girders spaced at 1975mm. the main girders and a 12mm thick steel plate fastened to their top flanges form together a channel section. the 12mm steel plate is stiffened by 10x160mm longitudinal stiffeners at 300mm centres. the longitudinal stiffeners are continuous through all the length of the bridge, passing through purpose-made grooves in the webs of the cross girders. with this design the deck becomes an orthotropic slab with different stiffness in the two orthogonal directions. the pedestrian walkways are 2450mm wide at the right-hand side, and 930mm wide at the left-hand side. they are designed as orthotropic slabs and are attached to the main structure as cantilevers. the thickness of the walking surface is 6mm, stiffened by 6x60mm steel strips. the cantilevers are 200mm deep "i" beams with a bottom flange 10x160mm, a web 8mm thick, and 1.5% gradient for draining.

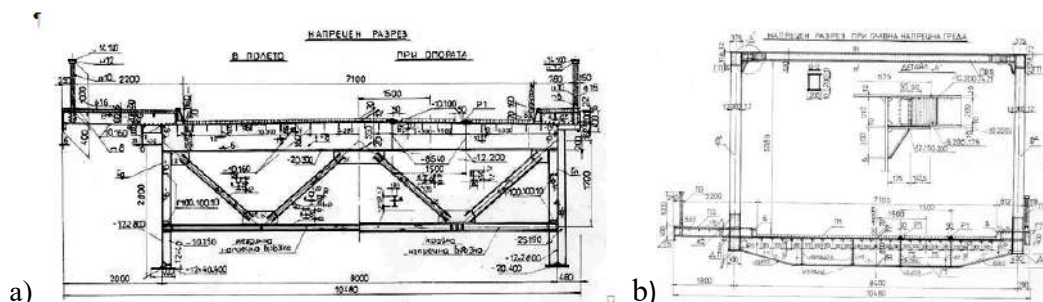


Fig. 17a,b. Cross section of the stationary part-a); Cross section of the main truss at a main cross girder location –b)

In order to satisfy the requirements of ship navigation, i.e. the bottom of the structure to be at the same level as that of the bridge before the accident, a truss solution with the road at the bottom chord level was proposed. Both from structural and aesthetic point of view, a simple triangular lattice without vertical members was adopted, (Fig. 17b). In addition, the structure had to weight no more than 95t, since it would be lifted by a 100 t floating crane during operation. In order to meet the weight requirement, the deck was designed as orthotropic slab, which includes parts of the bottom chord of the truss. The truss was designed as statically determinate, both internally and externally, with a span of 31,620mm. The bottom chord was divided into six members, with node-to-node distance 5270mm. The deck consists of a 12mm thick plate stiffened by 10x160mm longitudinal stiffeners at 300mm centres, main and secondary cross girders at 1757mm centres, where the main cross girders are spaced at 5270mm, and are supported by the nodes of the truss. The secondary cross girders are connected to the bottom chord of the truss, between the nodes, at equal intervals, three girders per node-to-node span. In order to alleviate the secondary cross girders, the main cross girders have significant stiffness. In fact, they are designed to independently carry the railroad traffic loads, without resorting to the contribution from the orthotropic deck, i.e. the orthotropic action is considered for the secondary cross girders and the longitudinal stiffeners only. The internal forces are obtained for heavy-vehicle loading HK80, and train loading type A. The secondary cross girders have a constant cross section with dimensions as follows: 8x450mm, 12x1550 and 20x160 for the web, the top and the bottom flanges respectively. The cross

sections of the top and the bottom chords, as well as those of the compressive diagonals are of box type. The tensile diagonals are designed as “I” sections.

6.3. Conclusions

The design and construction of the small Asparuhov bridge was executed without interruptions, due to the importance of the project to the national economy. The orthotropic structures were designed to modern standards, and were completely shop-welded at the Varna ship-maintenance yard. The design and construction were executed in parallel, which resulted in a fairly short completion period; just three months. All works, and the welds in particular were subject to strict quality control. All butt welds were X-ray inspected. A defective weld would then be removed, welded and inspected again. The bridge structures were shipped to site by a 100t Gants floating crane. The final assembly was done by simply placing the assembly units at their design locations. The assembly process went smoothly and was completed in a few days. The total weight of the structures was 81t. for the stationary part and 113t, for the movable part. The quality control for the design calculations and drawings was done by Prof. Dr. Hristo Popov, a prominent Bulgarian expert in steel bridges.



Fig. 18. View of the completed bridge

7. Design and Construction of the Orthotropic Steel Deck Bridge in Elin Pelin, Bulgaria

7.1. Introduction

The construction of the elevated road link on road II-165 Yordankino – Elin Pelin – Novi Han started on completion of a competitive design-and-build bidding procedure in which eight companies took part – “Nobels Group” from Belgium; “SFEM” from France, “Klos” from the Netherlands, “Krupp” and “Thyssen” from Germany, “S. M. Finsider” from Italy, “Gosha” from former Yugoslavia, “Rudnitsi i Zhelezarnitsa” from Macedonia[1]. The solution proposed by “Gosha” is an application of a unified modular system for the construction of elevated roads; its main features are described here on.

7.2. Structural solution for the elevated road at Elin Pelin

The elevated road is about 300 m long, and crosses over the railroad in the vicinity of the Elin Pelin railway station. The deck is a statically determinate plate girder system of Gerber type consisting of fifteen spans, as shown in Fig. 19. The largest span which is between piers 9 and 10 has length of 30 m. The rest of the spans vary in a close range between 18.766 m and

18.793 m. The principle girder of the Gerber system (with cantilever protrusions of 400 mm from the supports) is between piers 9 and 10, with the secondary girders between piers 1 to 9 and 10 to 16, being supported by a pier and a cantilever protrusion.

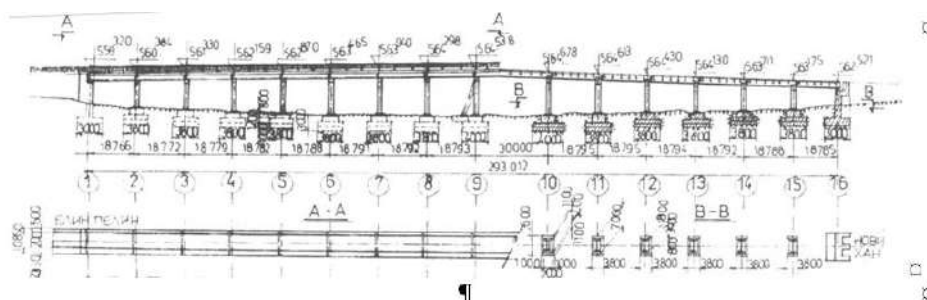


Fig. 19. Structural solution. Longitudinal section and plan view.

Three versions of the cross section were proposed with width of the road between 7.00 m and 7.50 m. The 7.00 m version shown in Fig. 20 was adopted for construction. The proposed structural solution for the elevated road is a classical example of an orthotropic road slab. The deck consists of main plate girders connected at the top by a steel plate. The steel plate is stiffened by longitudinal ribs and transverse girders. The stiffness in the two main orthogonal directions is different, hence the name “orthotropic”. The structural solution for the orthotropic slab is shown in Fig. 21. The structure for the road lanes is an orthotropic slab with thickness of the top steel plate 12 mm. The longitudinal stiffeners under the road lanes have a trapezoidal cross section manufactured by bending of a 6 mm thick metal sheet, Fig. 22. Ribs of such dimensions are sufficient for spans up to 3.00 m, and their significant torsional and bending stiffness allows them to perform well for temporary asymmetrical loads. Two pedestrian lanes (2.35 m and 1.50 m wide) are attached to the road lanes. They are also designed as orthotropic slabs with thickness of the top plates 10 mm, and 8 mm thick 150 mm deep longitudinal stiffeners.

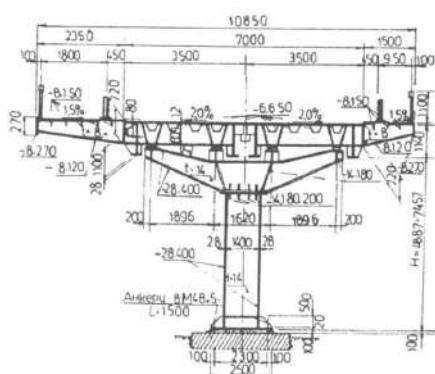


Fig. 20. Adopted cross section

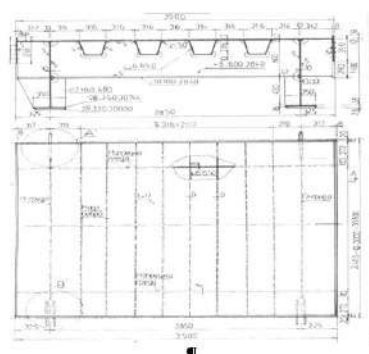


Fig. 21. Structural solution for the orthotropic slab

The road is supported by piers, at axes 2 to 15, and by the abutments at axes 1 and 16. Piers 2 to 8 and 11 to 15 are designed as plate I section columns, Fig. 8. Piers 9 and 10 are designed as box section columns.



Fig. 22. View of the sidewalks of the completed bridge (bottom).

7.3. Technical and economical requirements

The structure is delivered from the workshop coated with two layers of primer and painted. Two additional layers of paint were applied after assembly. The road surface – 7mm “Road coat” type was applied in two layers on site after assembly.



Fig. 23. The completed bridge

he total weight of the structure is 9890 kN. The design was submitted for review in June 1980, static and dynamic testing was carried out by the Road Institute (Bulgaria), and in May 1981 the facility started operation. A view of the completed bridge is shown in Fig. 23.

ACKNOWLEDGEMENT

This article is dedicated to the memory of the great Bulgarian bridge engineer: Assoc. Prof. D-r Eng. Borislav Borisov Bankov(1912-1992)

REFERENECE

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**ABOUT THE HISTORY OF EXHIBITED HALL No: 5 IN PLOVDIV,
REALIZATION WITH SPATIAL AND COMPOSITE STEEL - CONCRETE
STRUCTURES, AFTER 35 YEARS EXPLOITATION (2022-1987)**

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Abstract: In this paper the first author present the history of exhibited hall 5 in Plovdiv, realization with steel bar roof construction, type "МАрхI" and composite steel – concrete floor structures, after 35 years exploitation (2022-1987). Statics and dynamics investigation using finite element method was made and present as diploma project in 2006, with the second author of the contribution, which impressed the engineering community in our country and abroad with the depth of its investigation. Investigation is providing with SAP 2000.

Key words: MFE, steel bar, roof construction, composite steel-concrete structures

1. Introduction

The exhibition hall was intended for organization of exhibitions of the latest achievements of our and foreign building industry. The project is developed in the studio of architect H. Staynov (H.S.) in partnership with the first author (D.P.) of this article in 1987. Building consists of two main parts: exhibition hall and administrative building (Fig. 1a, b). In front of the main entrance is erected an independent spatial structure in order to demonstrate the system "MARCHI". On the east side, a monolithic RC staircase leads to the exhibition gallery at a height of 3.60 m, where there is also a cinema with 214 seats. The exhibition palace has a built-up area: 1872 m², unfolded built-up area 4675 m² and built-up volume 25902 m³.

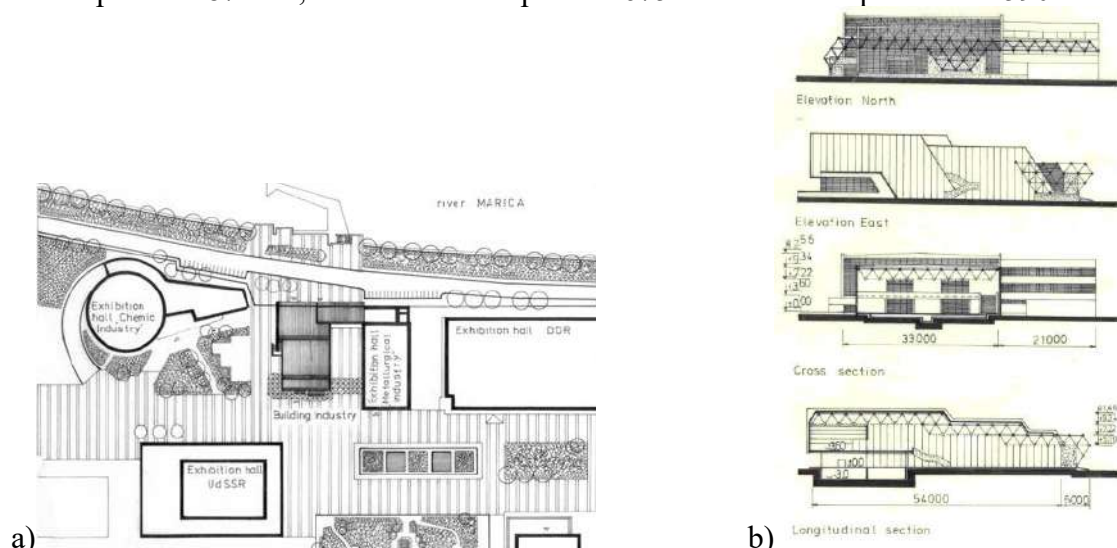


Fig.1a,b. Situation: according H.S.-a), Elevation:North, East, Cross and Longitudinal Sections:-according D.P.- b)

For the roof structure of the exhibition palace, a solution with a spatial-rod structure was proposed, the great possibilities of which allowed in this case to realize an individual architectural volumetric planning solution with different compositions in horizontal and vertical direction [1, 2].

1. Constructive solution

A spatially-bar two-layer structure with orthogonal upper and lower meshes and their connecting diagonals was chosen. The grid cells are square with dimensions of 3.0 x 3.0 m. The height of the structural plate is 2.121 m. In plan, the roof structure is a rectangle with basic dimensions: variable width 30-33-21 and length 54 m. In the longitudinal direction, the structure changes its height from elevation ± 0.00 to the elevation of the lower belt as follows: 5.10; 7.221; 9.342; 11.463 m. (Fig. 2, 3, 4, .5).

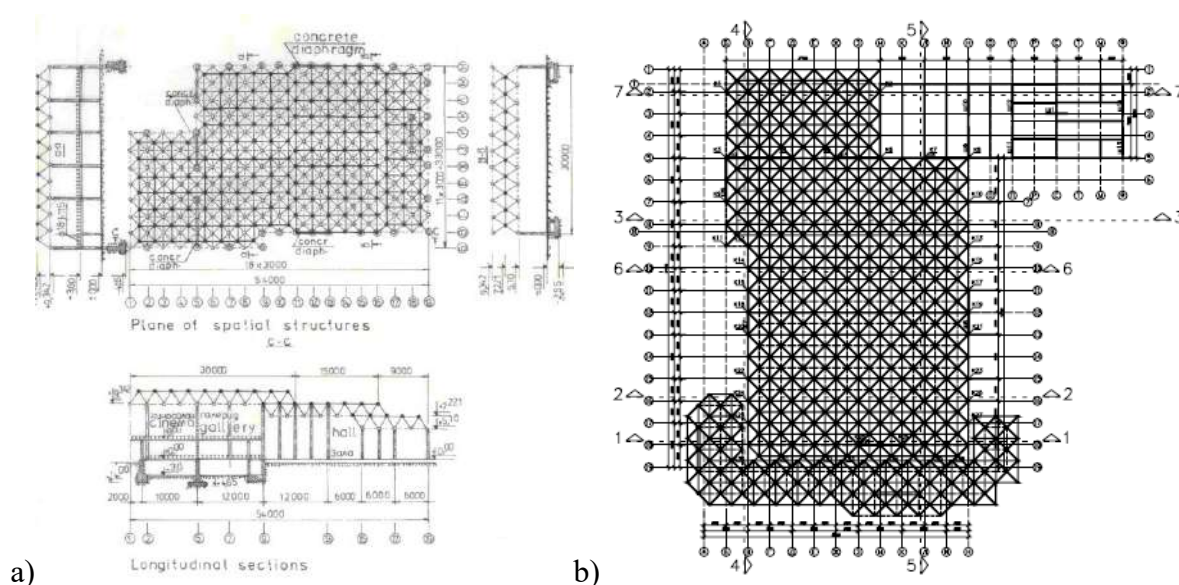


Fig.2a,b. Compositional solution: According D.Partov-a), According Z.Iliev-b)

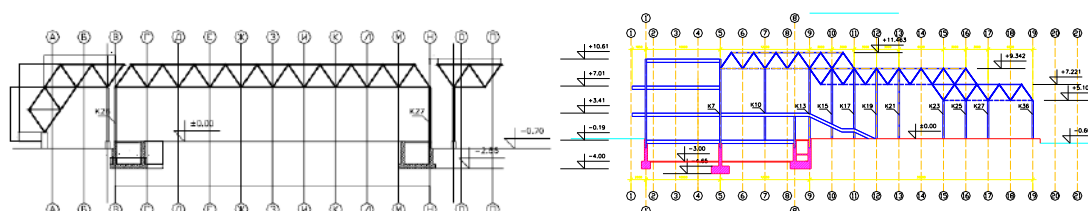


Fig. 3. Vertical section: 1-1, According-ZI, **Fig.4.** Vertical section: 5-5, According-ZI

The support of the structure is carried out not only on the lower belt but also on the upper one, where the heads for supporting the upper belt are specially expanded, with special bolts milled in one plane, rolled into their holes (Fig. 5a). In order to unify the heights of the steel columns supporting the structure to take the loads from it through the upper belt, special structures are constructed, made of a high-strength bolt ribbed along its height with steel plates, which are welded to the upper end of the columns (Fig. 5b,c).

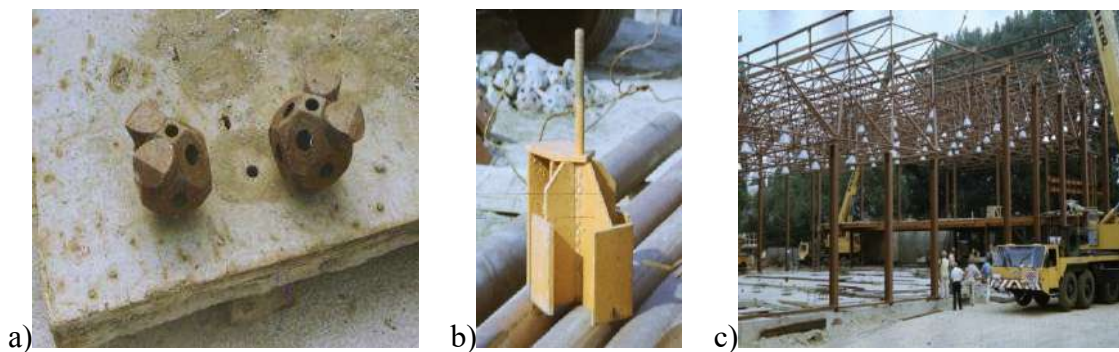


Fig. 5a,b,c. Extended of upper surface in nodal head along upper flange -a), Support, specially ribbed bolt, for additionally structure at the columns-b), Assembly of the roof structure with a ribbed bolt when pairing the spatial pyramids-c)

The columns are made of welded full-wall "I" profiles with dimensions of 200 x 340 mm (Fig. 6a). The spatial structure is executed as a continuous structural plate of two fields supported along axis 5 through 9-6-6-6 m (Fig. 6b). In order to rationally use the spatial behavior of the structures in square fields, in the places of transition from higher to bottom level, a pairing of the spatial grid was performed, in which only one-way behavior in the direction of the transverse size of the structure was excluded (Fig. 6c).

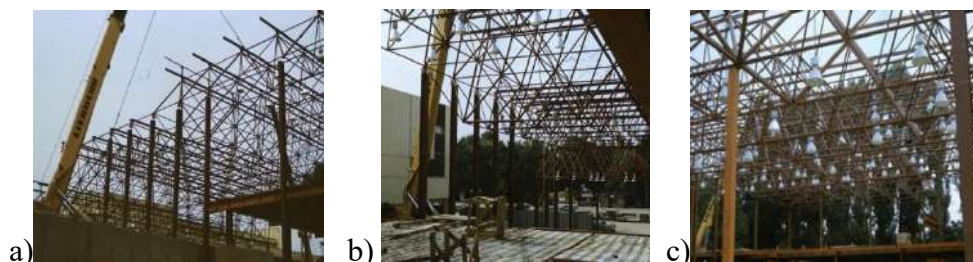


Fig. 6a,b,c. The column with dimensions of 200 x 340 mm-a); continuous plate on two fields supported-b); one-way behavior in the direction of the transverse size of the structure was excluded-c).

The rods are made of steel pipes of different diameters. At the ends of the rods are welded inserts fitted with axial bolts and moving hexagon special sleeves. The rods are with coordinating lengths of 3,0 m. The spatial structure is made of the following types of pipe sizes profiles: $\Phi 60/4$, $\Phi 76/4$, $\Phi 89/8$, $\Phi 102/4$, $\Phi 127/7$, $\Phi 133/8$ (Fig. 7a,b).

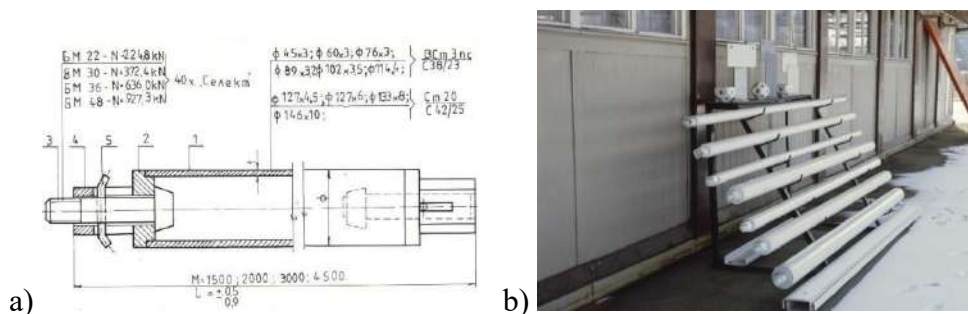


Fig. 7a,b. Structural rod -a);

Main structural rod - elements-b);

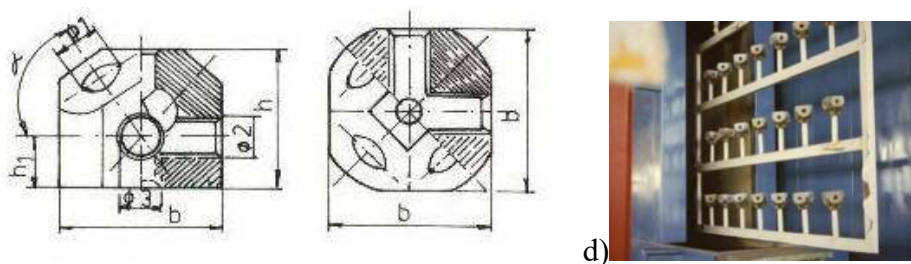


Fig.7c,d. Nodal members-c); Nodal members and anticorrosion protection-d)

The nodal members represent polyhedrons inscribed in spherical surfaces. On the flat side of the polyhedrons are drilled holes, which are threaded on the inside. Parts of the nodal members are used to connect horizontal rods at an angle of 90° and diagonal rods at 45° (Figs.7c, d). Predominantly they are used for buildings with spatial structures composed pyramids with square basis. The maximum tensile force that occurs in the structure and is absorbed by a high-strength M30 bolt is equal to 370 kN, and the maximum compressive force encountered in the structure is equal to 510 kN. The support of the structure is carried out along the contour with a total of 32 columns and inside the structure in 4 places on the RC diaphragms (Fig. 8a). The spatial stability of the hall is ensured by reinforced concrete diaphragms, rigidly connected to the columns by means of dowel connections (Фиг.8b).

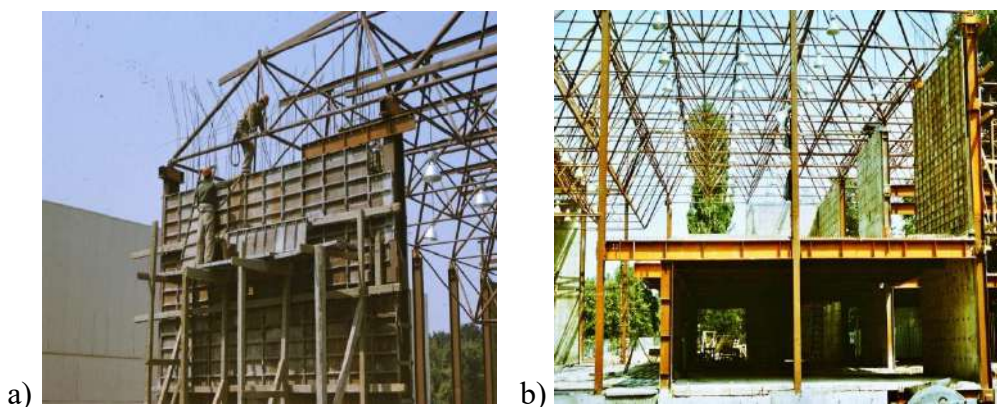


Fig.8a,b. RC diaphragms-a); RC diaphragms for spatial stability -b)

Of particular interest are the details of the spatial strengthening of the three-story administrative building [3], next to the hall, in which part of the rod members of the spatial structure had to pass through special box structures left in the reinforced concrete diaphragms (Fig.9a,b,c and Fig.10a,b,c).



Fig.9a,b,c. Special box structures left in the reinforced concrete diaphragms



Fig.10,a,b,c. Special box structures left in the reinforced concrete diaphragms

2. Application of the composite construction of Exhibited hall in Plovdiv

In 1987 for the exhibition hall of the Association "Construction and building industry" at the International Fair in the city of Plovdiv: a three-story administrative building, a cinema hall and a shop with composite floor structures, were designed (Fig. 1a,b). The structural solution at elevations 0.00, +3.60, +7.20 and +10.80 m is a joint skeleton-beam composite steel-concrete structure, which consists of transverse, longitudinal beams and columns made of full-wall "I" welded sections. The composition distance between floor beams is 3.00 and 2.50 m (Fig.11). The beams according to the static scheme are simple, with spans from 3.00 to 10.00 m. They are supported on steel columns and reinforced concrete walls. The connection between beams and columns is designed to take only vertical reaction. For this purpose, a special seat made of sheet steel is welded to the flange of the columns, on which the beam chair by means of two unequal L140.90.10 profiles welded to its stem (Fig. 12). In order to carrying horizontal exploitation forces, the corner profiles are attached to the columns by means of two M20 bolts. The connection between beams and reinforced concrete walls, is realized by means of an embedded steel plate in the seismic diaphragms.

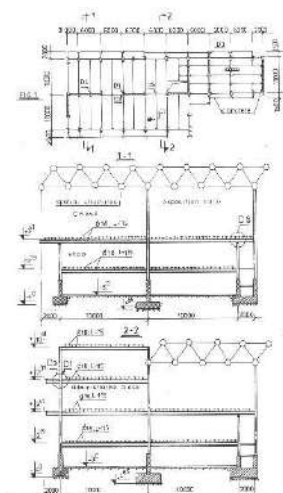


Fig. 11. Composite construction

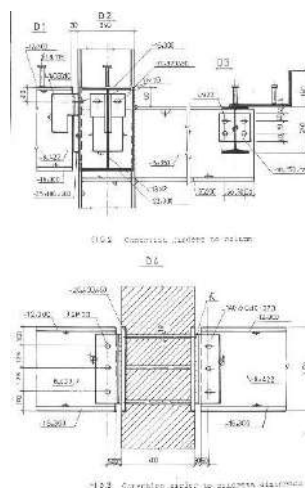


Fig.12. Joints between steel columns and RC walls

All columns of the three-story skeleton - gallery, cinema hall and shop - have uniform dimensions of 200/340mm. The columns end at their lower end with a support plate measuring 40/400/600mm (Fig.13a). The column step is connected to the strip foundation by two 30 mm anchor bolts. The floor plate with a thickness of 14cm is made of two parts: a prefabricated, pre-made in factory conditions, a pre-plate with a thickness of 8cm and a

monolithically cast part with a thickness of 6cm. The connection between the two parts is specially designed skeletons of reinforcing steel placed through 300 mm. The joint work between the steel beams and the reinforced concrete slab is carried out by means of bolted dowels with 18mm and length 115mm, welded in the axis of the upper belt of the beams evenly along their entire length with an electric welding apparatus of the English company "KROMP-ARC". The spatial stability of the building is ensured by vertical reinforced concrete diaphragms, located between the steel columns and rigidly connected to them by means of dowel connections (Fig. 13b and Fig.13c).

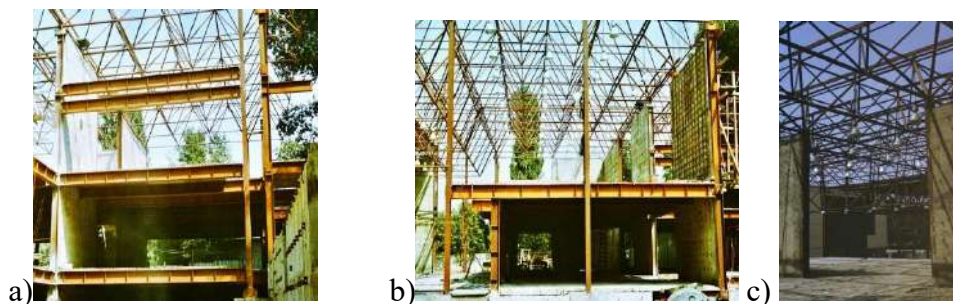


Fig.13a,b,c.Column step of the building - a); Steel columns and RC diaphragms-b,c)

The application of the composite structures for the first time at this site in our construction practice, proved the necessity of their use. Their use leads to a great saving of steel in floor structures, which amounts to 40-50% compared to traditional steel structures. With large support distances, a low structural height of the floor structure is achieved. The stiffness of the floor elements is significantly increased when considering the composite work of the two materials. In composite floor structures, the reinforced concrete slab is stiff enough to act as a horizontal diaphragm, eliminating the horizontal stability ties used in traditional steel frames. The application of reinforced concrete slabs leads to an economy of remaining trapezoidal sheet formwork]. In this way, technological time is saved for the preparation of the reinforcement, with a completely monolithic execution of the construction site.

4. Technology of installation of the steel structure

The structural plate between axes 9 and 17 is assembled on the prepared site, after which it was raised with a capital to a height of 2.12 m, above the site elevation. Next, a structural mesh was assembled between axes 16 and 19. Using an elevation of +3.60 m for an assembly platform, the structure between axes 1 and 19 was also assembled. In total, the installation of the entire structure was carried out by 4 installers at the Druzhba plant in 14 days. The installation of the roof structure together with the capitals, chairs, purling and lighting fixtures was carried out by three cranes. (Fig.14 a,b,c and Fig.15a,b,c) on 13.08.1987. The roof construction is assembled synchronously at the level: ± 0.00 and +3.60 m for 7 days by 4 workers and erected in place with the aid of three cranes "Liebherr", "Grove", "Faun".



Fig.14a,b,c. Technology of installation of the steel structure



Fig.15a,b. Assembly - view: north –a); Installation - view: west –b)

The roof coverings are made of "Bulgarmonopanel" with a thickness of the insulation layer of 50 mm. The support of the panels on the capitals from CFP 160 x 55 x 4. The slope of the roof lines is achieved by varying the heights of the chairs, made of WFP 100 x 100 x 8, connected to the joints means of bolts M20.

5. Static and dynamic investigation of the exhibition hall using FEM and SAP 2000

The static solution of the structure was carried out for a load of: self weight, snow, wind and horizontal and vertical load from earthquake forces using the software product "SAP 2000" [4,5]. In addition, the structure is examined in the assembled state (supported on four points). For the static and dynamic study of the structure, frame elements with six degrees of freedom in each node were used - three translational and three rotational components (Fig. 16) and shell elements with six degrees of freedom in each node – three translational and three rotational components (Fig. 17).

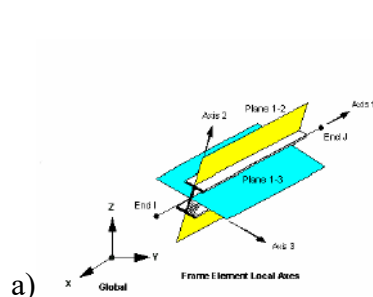


Fig.16. Frame element

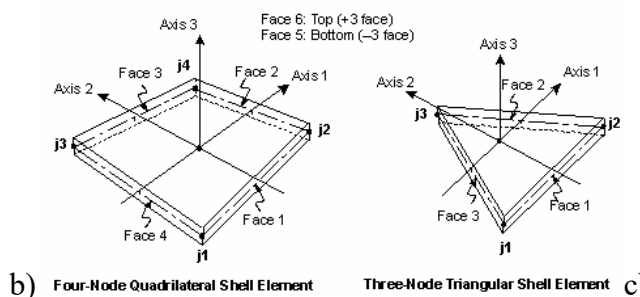


Fig.17. Shell element

During the geometric construction of the model and its discretization, a mesh of 3656 frame elements and 4984 shell elements was generated. The spatial rod construction consists of 1595 frame elements (Fig.18)



Fig. 18. The structure in the assembled state

The foundation of the exhibition hall is filled with strip foundations under the vertical bearing elements and a partial foundation slab under the stair well, and elevators haft in the administrative part. The ground base is modeled using the "joint springs" option, with which it is possible to calculate the coefficient of the ground bed (the Winkler constant) (Fig. 19) and (Fig.20).

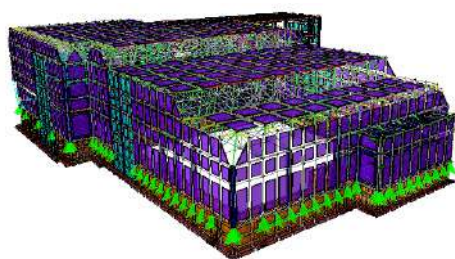


Fig.19. Facades - north and east[5]

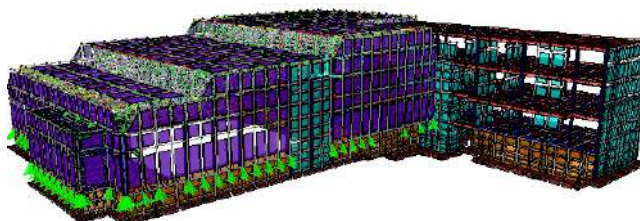


Fig. 20. Facades - north and west[5]

5. Conclusion

The total amount of steel for the roof pipe structure is 42 tons. The spatial structure was manufactured at the "Druzhiba" plant in Mihailovgrad, and the steel columns at the "RaichoKirkov" plant in Plovdiv. The construction of the exhibition hall of the Association "Construction and construction industry" was carried out from 1.03.1987 to 15.09.1988 with quality and on time under the skillful leadership of the technical manager D. Chukalov. The attempt to implement a continuous spatial rod structure supported on 36 supports was successful. The accuracy of execution of the steel columns and the accuracy of their assembly allowed the structure to step simultaneously on all support points without much difficulty. An architectural-constructive composition, very original in its appearance, was realized in a very short time, which reflects the modern achievements of building constructions in our country, (Fig.21, Fig.22, Fig.23, Fig.24, Fig.25).



Fig. 21. Outside view



Fig. 22. Front view : side: river Maritza



Fig. 23. Front view



Fig. 24. Outside view



Fig. 25. Outside view

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In memorium

for Civ.Eng. Zdravko Iliev (1980-2021).

The article dedicated to the short but meaningful and rich with professionally executed and designed interesting construction sites during the life of: Civ. Eng. Zdravko Iliev (1980-2021). He worked on this diploma thesis with great enthusiasms and defended with an excellent grade in 2006 at VSU, L.Karavelov. It was reported at a conference in the Department of “Structural Mechanics” in Bratislava, where he was applauded and praised for the great creativity he had put into the static and dynamic research and this complex building structure in 2006.

PS.The results obtained in the 1987 study of this exhibition hall with the computer program STRUDEL of IBM, were the same with those obtained by SAP 2000. To illustrate the achievements for investigated of the exhibition hall using FEM and SAP 2000, are shown some pictures with 3D models from contents of this diploma work(Fig.26, 27, 28, 29, and 30).

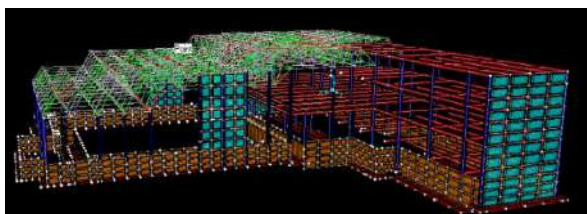


Fig. 26. Outside view

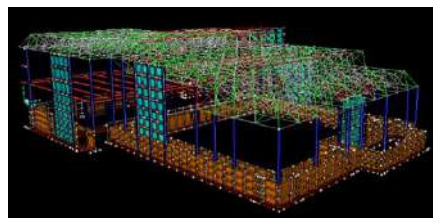


Fig. 27. Outside view

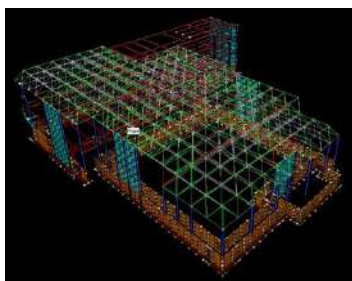


Fig. 28. Outside view

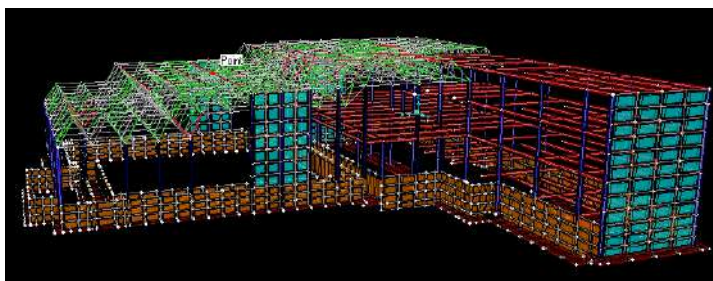


Fig. 29. Outside view

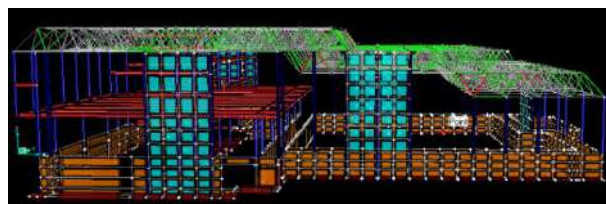


Fig. 30. Front view

METHODS FOR INVESTIGATION AND RECONSTRUCTION, MANAGEMENT AND ADAPTATION OF THE IMMOVABLE CULTURAL HERITAGE (ICH)/ IN BULGARIA IN THE CONDITIONS OF GLOBAL SUSTAINABLE DEVELOPMENT

F. Rangelova¹, Y. Kandulkova², A. Apostolova³, Y. Cenkova⁴

Abstract

One of Bulgaria's greatest treasures is its cultural heritage, including structural and architectural values. Nowadays, the cultural heritage of every nation is perceived as a common human wealth. The world community stands for common principles to respect the cultural variety and traditions of various nations.

Renovation of architectural and structural values is a topic of great importance worldwide, especially in the most developed societies. It comes because of the necessity to improve the condition of existing buildings of high importance in compliance with the requirements for sustainable development, as well as preserve the immovable cultural heritage for future generations.

One of the main aspects of the renovation of architectural and structural values is the restoration and reinforcement of their structure, which ensures their preservation and safety. Structural sustainability and safety evaluation of existing values is a complex task to be performed, as the applied methods in that case substantially differ from the methods for design of new buildings. On the other hand, side such structural reinforcement could possibly end up in conflict with the cultural identification and value of an architectural monument.

Therefore, the choice of method to restore and reinforce the existing architectural and structural value depends on its cultural value, ranging from basic maintenance where no cultural value of the building is sought to be changed, to a complex restoration and renovation, in the case of need to improve its operational state.

The need for structural renovation of cultural and historical monuments is justified by the objective of achieving one or more of the following circumstances: the presence of visible damages in the building; damage or destruction occurred after an event that affects the stability and load-bearing capacity of the building structure (as earthquakes and other natural disasters, and accidents); change of the function of the building; and implementation of the requirements of modern norms and standards in terms of stability and the load-bearing capacity of the structure (Eurocodes, modern design standards for seismic).

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In the framework of the proposed paper, main object of the study is the immovable cultural heritage in Bulgaria as a bright symbol of national identity with relevance to the values of European and global scale, coming from the era of Antiquity, Middle Ages, after the Liberation and Architecture of XXth and XXIst century. This raises the team's interest to develop, define and classify the methods of examination and reinforcement/recovery of immovable cultural heritage in Bulgaria in compliance with global sustainable development, to future generation.

Keywords: *investigation, reconstruction and conservation, management and adaptation, immovable cultural heritage, global sustainable development*

Introduction One of Bulgaria's greatest treasures is its cultural heritage, including structural and architectural values. Nowadays, the cultural heritage of every nation is perceived as a common human wealth. The world community stands for common principles to respect the cultural variety and traditions of various nations.

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2 Problems addressed to the management of the immovable cultural heritage

One of the greatest treasures of Bulgaria is its cultural and historical heritage, including architectural and construction values. The rich cultural heritage of Bulgaria is distinguished

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with unique and complex character, including yourself: high cultural value of the various historical layers that represent impressive examples of the types of construction and historic sites and monuments with special meaning to the values of the era of Antiquity, the Middle Ages, after the Liberation of Bulgaria and the Architecture in XX and XXI centuries; a rich historical rearrangements of the environment resulting from the significant an exchange of influences between the West and the Orient in a European cultural crossroads; pluralism of the cultural heritage - stylistic, religious and ethnic, which allows areas of the harmonious coexistence of different religions, ethnicities and aesthetic concepts.

In the context of globalization, immovable cultural heritage has exclusive value that should be protected by measures of public interest. One of the most important documents for the preservation of immovable cultural heritage is International Charter on the Conservation and Restoration of Monuments and Sites – the Venice Charter. (The Venice Charter, 1964) Rigorous methodology for interventions is defined in the Charter, to be preserved the heritage. Article 9 from the Charter in most decisive and it becomes a law on the restoration and renovation activities. It states: “The process of restoration is a highly specialized operation. Its aim is to preserve and reveal the aesthetic and historic value of the monument and is based on respect for original material and authentic documents. It must stop at the point where conjecture begins, and in this case moreover any extra work which is indispensable must be distinct from the architectural composition and must bear a contemporary stamp. The restoration in any case must be preceded and followed by an archaeological and historical study of the monument.” (The Venice Charter, 1964, article 9). “Any intervention in an immovable cultural heritage (ICH) found full intrinsic value of site and recreates for the society the significance of Cultural Property and creativity of the author and the social consciousness. The basis of criteria determined the limits of the possible and permitted interventions in the life and the future of Immoveable Cultural Heritage Sites “. (Mikhailovsky, 1971).

The adaptation is one of the eligible ways of interfering with the ICH in the process of its preservation. According to the current Cultural Heritage Act “Adaptation is the protection of immovable cultural property by ensuring their contemporary use, consistent with the conservation and restoration methodology”. (Cultural Heritage Act (of Bulgaria), 2009). The understanding of adaptation evolved into a complex method that includes activities for conservation and restoration and sets the function of the site, it restores its usability and adapts it to the actual structure. The role of adaptation in conservation is in the rebirth of old or introduction of a new function. Every new function on a site brings a new attitude to the object itself. The adaptation allows preserving the nature of the object while it persists over time. The adaptation is intervention, which is determined by the need for modern use of the site. It can be treated in two ways - first as compromise with site's documentary, but on other hand as an only way for continuing existence on the site. The adequate new function provides excellent reintegration of the site in modern life without reduce its value and preserve its viability.

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The preservation of the original spirit is the most important characteristic, which will remain a major carrier of the historicity of the site. While at the beginning of the previous century the processes of adaptation run relatively smoothly and the basic rule is to accumulate historical layers, the subsequent economic, political, and social changes lead to a change in attitudes to architecture and urban planning - clearing urban layers and replacing them with new components. The result of these actions is sharp and nasty intervention in historical zones and in urban structure. Today these areas, usually in the central areas are attractive places for replacement of building with "higher," "more representative", "more comfortable" or "commercial". The inherited values do not adapt to modern need, this is reason for the appearance pseudo-ancient houses or modern faceless structures that do not harmonize with the surrounding buildings.

Today the immovable cultural heritage of every nation is perceived as human wealth, the protection of which the world community, respecting cultural specifics and traditions of different peoples backs general principles. Knowledge and respect for these principles is a sign of civility and professionalism in the attitude to the heritage. Modern principles of the protection of architectural and construction values embody:

- 1) The renovation intervention is preceded by a thorough understanding and research of the value of its kind, state, typology, style of similar objects on it etc.
- 2) Respect to the original in cultural property - the authenticity of value is its essential feature (if an object is not authentic certificate, it just is not valuable as defined by it). In this sense, the preservation of the original is a-priori objective of its conservation.
- 3) The renovation concept is a function of the heritage value.
- 4) The renovation additives are different from the original as well as the material and the manner of construction.
- 5) Architectural monument should be preserved in its development, preserving, and proclaiming the valuable contributions of all periods of his life and traces of passed the renovation activities.
- 6) Value of the monument in all its aspects (scientific – documentary and aesthetic - art) is the renovation intervention. Each renovation is based on a thorough understanding of the monument.
- 7) The renovation must demonstrate a lack of knowledge about the monument and allow future studies of unclear fragments - this feature of the renovation approach is extremely important because it is a contribution for the European renovation theory, it is a jump to the modern principles of the renovation.
- 8) The renovation intervention rather than physical preservation of the monument is to be achieved and its readability - renovated value must be understood and liked by the people to enable them to appreciate and preserve.
- 9) Renovation of values is associated with the first idea of using the monument, but still such an intervention is seen as a compromise with the principles of its conservation.
- 10) Communication of the monument is an important tool for inclusion of the general public to architectural heritage and understanding as a social value, but it should be

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achieved on a strictly scientific basis without any compromise on the authenticity of the knowledge of antiquity.

11) With renovation we apply modern building materials.

12) The renovation intervention is preceded and implemented in documenting the memorial and performed the renovation activities.

13) The environment of the monument is subject to equivalent care protection. The term "cultural environment" is gaining in importance, which raises the question as to sustainable development; Recalling are links between different categories of heritage on the one hand, and policies for urban planning and development of territories on the other. Architectural value, although set in a bygone era, is part of the modern environment and should be viewed as a vital element of modern reality and potential for its development.

Renovation of cultural property (architectural and construction) is a topic of great importance worldwide, especially in the most developed societies. It is the result of the need to improve the condition of the buildings of the existing values in accordance with the new conditions for sustainable development and preserve it for future generations.

Existing cultural property (architectural and construction) are subjected to processes of destruction over time, leading to an inability to perform the functions assigned to them by their construction. In some cases, the renovation is a necessity dictated by the desire to improve conditions in them or adapt them to new functional requirements.

One of the main aspects of the renovation of cultural values is the restoration and strengthening of their structure, which ensures their preservation and safety. The evaluation of the sustainability and safety of the structure of existing values is a complex task because applied for these methods differ from the methods for designing new ones. On the other hand, the possible strengthening of the existing structure of a cultural property could conflict with their cultural identity and value.

Therefore, the choice of method to restore and strengthen of the existing cultural property (architectural and construction) depends on its cultural value, ranges from basic maintenance where we don't aim to change the cultural value of the building, to a complex restoration and renovation, in the case of need to improve the serviceability state.

As stated above, the cultural property (architectural and construction) is regarded as existing buildings with significant cultural value. They may be buildings, towers, bridges etc. They are mainly performed by ceramic masonry and wood, sometimes with elements of steel or iron.

The need for structural renovation of cultural and historical sites is justified by the objective of achieving one or more of the following circumstances: the existence of visible defects in the building; damage or destruction occurred after an event that affects the stability and load-bearing capacity of the structure (as earthquakes and other natural

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disasters, and accidents); change of the function of the building; and implementing the requirements of modern norms and standards in terms of sustainability and the bearing capacity of the structure (Eurocode, modern design standards for seismic loads).

The construction of immovable cultural heritage poses a challenge for modern methods of diagnosis, analysis and renovation, rehabilitation, and restoration, as the application of modern codes and standards in construction is very limited in their nature.

Moreover, structural rehabilitation of cultural and historical sites carries with it consequences in the architectural, historical, structural, economic, and social terms, depending on the extent and duration of the process of renovation. In implementing the structural renovation of cultural property all these aspects must be considered. The implementation of structural rehabilitation necessarily involves the application of engineering knowledge, considering the cultural specificities of the site of renovation.

Only when there is interaction between the engineering requirements and compliance with the cultural characteristics adequate decisions can be taken for the renovation of immovable cultural heritage. To be successful from an engineering and cultural point of view, an intervention has to be implemented based on generally accepted principles for the restoration of immovable cultural heritage, must be the principles set by one of the referenced documents for restoration of architectural heritage, namely the International Charter on the Conservation and Restoration of Monuments and Sites – the Venice Charter 1964 / in more detail above / more mainstream, including: ensuring the structural stability and safety; preservation of the cultural value of the building; ensuring minimal intervention; reversibility of the intervention; integration of the entire building; compatibility of the materials; minimal price.

In practice, however, it is not always possible to follow all these principles simultaneously, as they sometimes are in conflict with each other. Usually achieving structural stability and safety of immovable cultural property conflicts with the preservation of its cultural and historical value. Hence the need for structural strengthening of immovable cultural heritage imposed compromising these principles within accepted norms.

In practice these principles are considered the reference conditions for intervention. The study of the immovable cultural heritage in Bulgaria, as a symbol of national identity with particular relevance to the values of European and global era of Antiquity, the Middle Ages, after the Liberation and architecture of the XX and XXI centuries for development, refinement and systematization the methods of investigation and conservation/restoration/rehabilitation of ICH in Bulgaria is very important and significant in the direction of supporting the country's efforts for the preservation and transmission to future generations of this cultural-historical heritage in compliance with global sustainable development.

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Nationally representative data on the state of ICH in Bulgaria, alternative for their conservation/ restoration/ rehabilitation, management, and possible adaptation methods, and analyzes is not only a significant benefit to stakeholders but give adequate understanding of the extent and nature of the upcoming their implementation work.

All these considerations explain why the study of methods for investigation and restoration/rehabilitation, management, and adaptation of ICH in Bulgaria are of such importance.

The collection of empirical data and analysis on the set of problems give valuable information required for process management and sustainable development of ICH in Bulgaria; have great value in the development of modern legislation, which is enshrined as a priority strategy for the development of national, European, and global level, and national identity; as well as it is relevant to the fields of advertising, tourism, and business.

3 Determination of Methods for investigation and reconstruction, management, and adaptation of the ICH - hypotheses and approaches

With respect to architectural and construction values belonging to Global and Bulgarian heritage, the investigation and research of the issues related to the Methods for investigation and reconstruction, management and adaptation of the immovable cultural heritage are declaration of solidarity with the efforts of modern society to preserve this heritage for future generations.

First are development, definition, and systematization of methods of investigation and conservation/ restoration/rehabilitation, management, and adaptation of ICH. The investigation and research are addressed and systematized also the used typical construction materials and structural systems as well as typical defects and failures in the structure of ICH.

Important research is subjected on the study of factors influencing the state of ICH. Classifying them according to their degree of influence in the hierarchical relationship will enable us to establish which of the tested methods require refinement or adjustment in accordance with the declared common principles and requirements for the protection of world cultural heritage, considering cultural specifics and traditions of different nations.

During the study the following assumptions must be verified:

Hypothesis 1 - Verification on the assumption that the state of ICH is determined by the influence of various factors such as period of construction, type of design material and system, location, natural resources, and the environment, etc., The degree of their influence is different for different objects.

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Hypothesis 2 - Verification of the assumption that there is a different value of the separate ICH's. This will help the scientific justification of the decision-making process of their investigation and conservation/ restoration/rehabilitation, management, and adaptation.

Hypothesis 3 - The methods of investigation and conservation/ restoration/rehabilitation, management and adaptation of ICH determine the type and value of the separate ICH. There are various alternative methods of investigation and restoration/rehabilitation, management, and adaptation of ICH whose selection must be carried out strictly individually to preserve the value of the site, subject to generally accepted principles for this.

Hypothesis 4 - ICH in Bulgaria presents the specifics of Bulgarian history and culture in terms of architectural and structural shapes, functionality, and appearance. Must be assumed that in terms of taking measures management and adaptation of ICH in Bulgaria in the direction of modernization, they will prove more definitely conservative.

Hypothesis 5 - Communication of the monument is an important tool for inclusion of the public to cultural heritage and understanding as a social value, but it should be achieved on a strictly scientific basis without any compromise on the authenticity of the knowledge of antiquity, suggesting that the renovation intervention than physical preservation of the monument is to be achieved and its readability. It is likely that the research must establish an attitude to compliance with certain rules in the conservation/ restoration / rehabilitation of ICH, but it is violated because of the need of renovated value can be understood and liked by the people to enable them to appreciate and preserve it.

Multi-criteria analysis of the Methods for investigation and reconstruction, management and adaptation of the ICH provide more comprehensive methodology for the management of ICH with possibility for alternative and innovative solutions in the context of global sustainable development. The complex methodology is determined by the interdisciplinary nature of the problem, which is embedded in the Architecture and Art, Construction and Geodesy.

The depth comparative analysis of the methods for investigation and conservation/ restoration / rehabilitation of the immovable cultural heritage (ICH), in Bulgaria, that is extracted from theoretical and experimental research can be used as a starting point and the basis for making adequate science-based solutions for sustainable development and preservation of ICH for future generations. Although ICH was built in the past, but the ICH is a part of the modern environment and should be seen as a vital element of modern reality and potential for its development.

4 Conclusions

Renovation of architectural and structural values of Cultural Heritage is a topic of great importance worldwide, especially in the most developed societies. It comes because of the

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necessity to improve the condition of existing buildings of high importance in compliance with the requirements for sustainable development, as preservation of the Immovable Cultural Heritage (ICH) for future generations.

The study of methods of investigation and restoration/rehabilitation, management, and adaptation of the immovable cultural heritage (ICH) as general, as in Bulgaria, contributes for the transformation of human society into "Society of Knowledge"; provide new knowledge about the ways for the conservation of ICH, which are obligatory in the context of sustainable development. This is achieved through the analysis of: ICH status, factors that affect their architectural and structural rigidity; the type of materials of which are built; their structural systems; damage; methods of investigation; methods for restoration/rehabilitation and the possibility of adaptation.

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FOOTBRIDGES ACROSS HISTORIC SITES : CABLE SUPPORTED ALTERNATIVES

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Abstract: *Three cable supported alternatives have been developed for footbridges over historical sites. The variants are cable structures and thus exhibit a completely different behaviour from the arch structures, previously developed. The first variant is a straight intersection of variable width. In comparison with an arch, it is striking that the deformability is much greater and the fundamental frequencies are significantly lower. However, the characteristic values for dynamics, such as the Scruton number and the accelerations of the bridge deck are significantly better. To span the site of 200 m in length, pylons of 46.7 m in height are needed. As a second alternative, a semi-circular suspension footbridge of variable height has been developed. Direct comparison with an arch counterpart is not evident. However, it could be deduced that this structure has a fairly high frequency and is also not sensitive to the important types of vibration. A full elevated walkway around a circular site can certainly be compared to its arch-shaped counterpart. Despite the lower fundamental frequency, the higher flexibility and the increased consumption of steel, the cable-supported structure performs better as for its dynamic behaviour.*

Key words: *Historic sites, cable-supported structures, vortex shedding, human induced vibration, Scruton number, conceptual development*

1. Introduction

Covering historic sites has been implemented on several sites in order to protect them from unwanted intrusion, from disturbing of the archeologic activity and to provide shelter from weather, erosion and weir. However, these shelter structures may have negative influence on the protected remains [1]. This generated the idea that partial cover, including movable additional shelters by foils, might provide a limited answer to this issue. In addition, the public should be allowed to see valuable remains and archeologic activity, without interfering with it. For this, several historic sites have been equipped with elevated walkways around the places of interest. The latter mostly allow a bird's eye view on the remains.

This idea has inspired to try develop concepts for walkways above historical sites. In previous research 3 concepts, mainly based on arch forms, have been developed. For each one satisfactory results were obtained, although 2 out of 3 alternatives required structural damping due to vortex as is proven to be the most detrimental condition. Vortex shedding

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might be improved by using cable systems, rather than arch members. This has inspired the present research for finding cable supported walkways.

To remind of the former work, 3 arch-type alternatives are reported in [2]. Each alternative consists of 2 lateral load-carrying tubular members connected by crossbeams. The first one is a straight crossing of a single arch, offering a public view at the arch top. Alternative 2 is a tripod-arch with a central viewpoint and reinforced connection of the 3 arch branches. Alternative 3 is a circular walkway around the site, both ends descending to the ground floor.

2. Cable-supported alternatives

The emphasis of all objects seen on a historic site should be the heritage objects and remains. This implies that either the number of pylons for cable-supported structures, or the height of the pylons should be limited. Thus the focus should be drawn away from the new structure, which must be as light as reasonably acceptable. As the arch principle gave rise to heavy member's sections, cable structures may become more advantageous.

However, one may expect that cable-supported structures will be more flexible and suffer from dynamic effects such as vortex induced vibrations and human induced stability problems. Hence, any analysis will require nonlinear calculation.

As for the live loads on this type of structure, conditions similar to the former research, valid for non-public or controlled occupation, have been applied. This includes a vertical live load of 1.5 kN/m² for any location on the bridge superstructure and 2.5 kN/m² for those locations where visitors may come together, such as viewpoints. For public walkways the load would equal 2.5 kN/m² all across the footbridge's deck. The floor of the walkway consists of 31.52 mm laminated glass, which allows to have a maximum distance of crossbeams of 3 m. Limiting the number of crossbeams adds to the transparent character of the structures.

2.1. Alternative 1 : straight crossing

This type of walkway is particularly adapted to cross a circular or rectangular site of 200 m diameter or relevant dimension. The vertical clearance below the structure is at least 8 m. The latter makes it not adapted to higher remains than some 4 to 5 m. Nevertheless, for a bird's eye view, visitors will have a good view on the site. The largest slope of the walkway equals 6%, which might give difficulty for motor disabled persons. The width of the walkway varies from 14 m at the entrance to 5 m at the narrowest point and to 12 m at the centre where a wide space for viewing is provided. It is expected that visitors will gather at this particular location and the load of 2.5 kN/m² will be practical. Although disturbing the view, the vertical hangers, connecting the main suspension cable to both the stiffening girders are arranged at a distance of 6 m. The number of hangers may be large, yet their diameter of 16 mm is kept small.

The main suspension cable, consisting of 37 T 15 strands, is a single flexible tension member. Hence, the hangers show an inclination, which may cause hindrance to the public when walking along the railing. Therefore the railings cannot be straight at the edge of the walkway deck and must revolve around the hangers. This should prevent distracted people from running into the hangers.

Fig. 1 shows the general concept of this alternative. The distance of pylons equals 209.38 m, their height being at 46.7 m above the level of the walkway. Obviously, the main cable cannot sag lower than some 11.3 m above the walkway. This should avoid further hindrance for visitors due to the sloping hangers. Fig. 1 also exhibits the 3 backstays of each pylon. The efficiency of this arrangement may be discussed. The idea is that all 3 backstays are equilibrating the pylon's top and the two outside stays are either loosened or stressed by horizontal wind. It appeared that none of these backstays becomes slack.

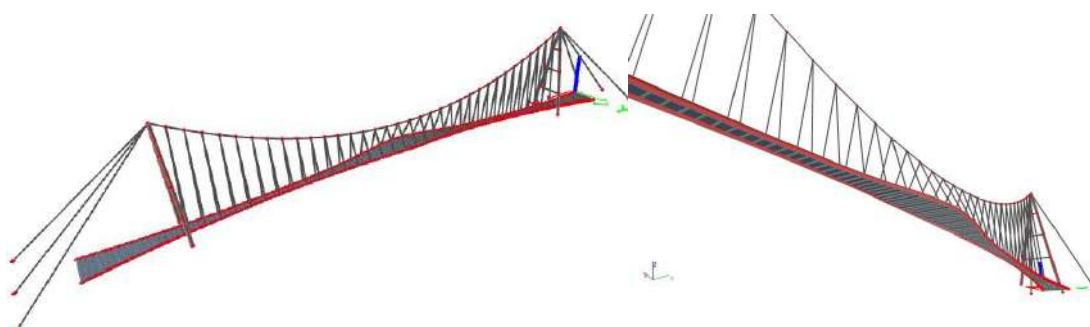


Figure 1 : Alternative cable-supported straight crossing

2.2. Alternative 2 : semicircle tour

The counterpart of a tripod cable supported structure proved to be an inadmissible solution, since it would have required to supplement the structure with either a central pylon, to be built inside the historical remains, or several huge pylons that would have to be connected to the central platform. In view of the guiding principles to respect, this alternative is not a complete walkway around the site. The elevated part is limited to one side, since the aim is to allow the public to approach a higher part of the historic site, such as a small tower or hilltop. The part at ground level then is used to view the lower side of the remains. Obviously this structure is less ambitious than the former. It is adapted to circular historic sites of 200 m diameter, although the circle centre is somewhat shifted to outside the location.

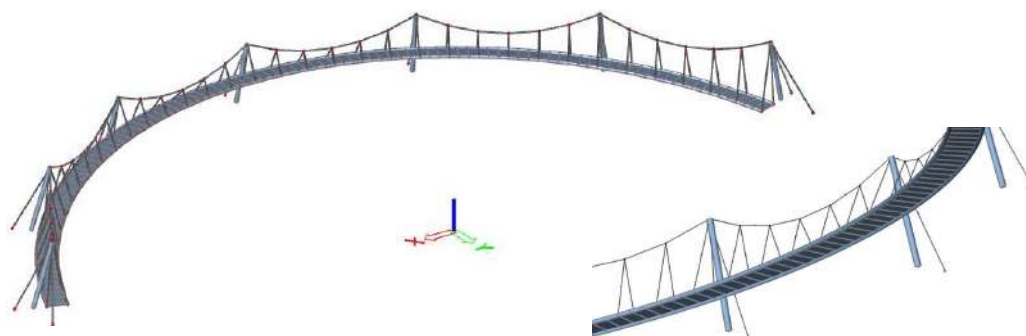


Figure 2 : Alternative 2 semicircle tour

The maximum elevation of the walkway is now slightly higher as for alternative 1 and reaches 10 m thus allowing closer view of an elevated part. The walkway width is constant and equals 6 m, since there will be no preferred gathering point. The pylons reach 10 m above the walkway and are located at a curved distance of 51.33 m. As they are located outside the circle, the pylons have an inclination of 15° . The hangers connecting the main cable with both edge beams of the have a slope of 8.7° for the outside hanger and 23.5° for the inner hanger. This somehow increases the risk of having insufficient horizontal clearance on the walkway and explains why it is 1 m larger than for alternative 1. The hangers are at a distance of 8.5 m and a small diameter of 12.5 mm, whereas crossbeams are provided at a distance of 2.8 m. Each pylon, including those at the ends, has a single backstay.

The main suspension cable consists of 21 T 15 strands and shows a sag of 9%, which certainly is lower than the 13% of alternative 1. Again, the main cable is a single element. Fig. 2 exhibits the general layout of this alternative.

2.3. Alternative 3 : Full circle tour

The third alternative is a complete walkway around the site. The diameter equals 200 m and the public has access to the walkway by a straight ramp, which is rigidly connected to the superstructure. The full circular walk around a site might be more attractive to the public than the hybrid alternative 2. The main characteristics of this structure are similar to the former one, albeit the details are different. Instead of 7 pylons, there are 11 of the same height at a distance of 54.1 m. Each pylon has a double backstay, the first and last having an additional long backstay to counteract the horizontal cable force. The walkway is 6 m wide and the hangers are located at a distance of 9 m and have a diameter close to 15 mm. Crossbeams are provided every 3m. The single main suspension cable of 19 T15 strands shows a sag of 11.5 %. Fig. 3 exhibits the general layout of alternative 3.

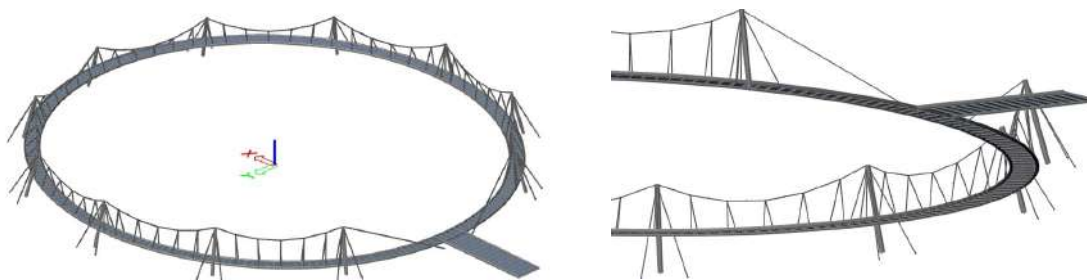


Figure 3 : Alternative 3 full circle tour

3. Results from nonlinear analysis

3.1. Straight crossing

As mentioned in 2.1, the vertical load of 2.5 kN/m^2 is concentrated near the span centre. This is exhibited in Fig. 4. Since the ends of the superstructure can easily be fixed to abutments and excessive deformation must be prevented, the boundary conditions were chosen as to simulate hinges at the 4 extreme points of the superstructure. This implies that thermal effects will cause the superstructure to rise during warming and to sag while cooling. However, no negative reactions were found.



Figure 4 : Distribution of pedestrians load (blue = 2.5 kN/m^2 , red = 1.5 kN/m^2)

Since the exact location of the site is yet unknown a basic wind speed of 26 m/s^2 has been assumed. Since the characteristics of the various elements like main cable, hangers and bridge deck are different, the wind pressure varies from 1.43 kN/m^2 for the pylons to 1.77 kN/m^2 for the main suspension cable. An important load is the horizontal force of 1 kN/m at the top of parapets or railings. This is due to persons leaning on these closures, which is most probably what happens in practice. The main cable as well as the hangers need prestressing in order to avoid slackness and to obtain the desired shape of the structure. In this, the effect of pretensioning on the deformation modulus is taken into account.

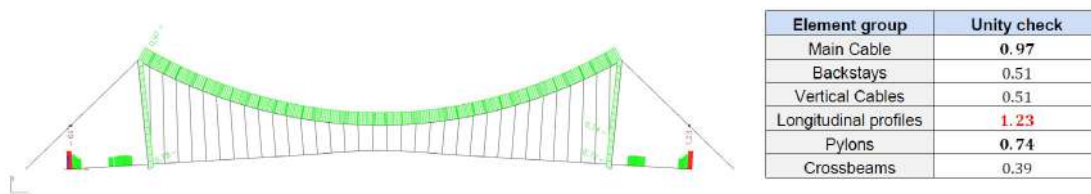


Figure 5 : Unity checks Alternative 1

Results of ULS-calculations can be expressed in the unity check of equivalent stress. Fig. 5 shows the adequate design of cables and beam sections, since for most members the unity check approaches 1. For instance, in the main suspension cable it reaches 0.97. Yet the extreme 2 m sections of the main lateral tubes of the walkway deck need some reinforcement, since the unity check reaches 1.23.

The flexibility of the structure can be assessed by the deflections caused by the live load. Compared to the overall length between both pylons, the relative deflection equals $426 \text{ mm} / 217479 \text{ mm} = 1/637$. There is no reference value for the deflection, yet since it is below $1/500$ this is generally qualified as flexible but within acceptable limits.

3.2. Semicircle tour

In this case various positions of the 2.5 kN/m^2 vertical loads have been considered. This is exhibited in Fig. 6. A chessboard combination seemed to generate the most detrimental situation. Identical conditions for wind load have been implemented. This leads to wind pressures varying from 1.46 kN/m^2 on the pylons to 1.99 kN/m^2 on the main tube girders of the walkway deck. Obviously, since the main suspension cable is lower than for alternative 1, the wind pressure is decreased. The effect of persons leaning on railings and parapets is all the more relevant in this case. The boundary conditions are similar as for alternative 1 and are fixed by hinges. Prestressing of the main cable is almost unnecessary, whereas the backstays need considerable prestress of 3000 kN.

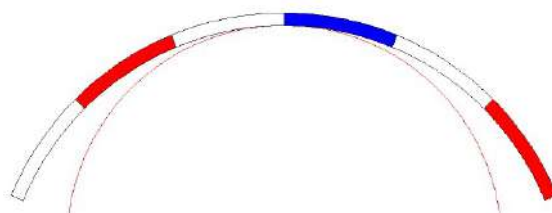


Figure 6 : Chessboard loading (blue area 2.5 kN/m^2 red : 1.5 kN/m^2)

The ULS verification by unity-check renders values between 0.23 for the main suspension cable to 0.79 for the pylons, 0.80 for backstays and 0.82 for the vertical hangers. The low value for the main cable is due to dynamic effects and the large deformations of the cable. This is altogether not a surprise, since the deformations of the walkway deck reach to $186.3 \text{ mm} / 53558.7 \text{ mm} = 1/287$. This value implies that the structure is highly flexible and might cause problems for wind and vortex shedding, as will be explained in 4.3. The alternative of reinforcing the lateral tube members of the walkway deck introduce higher vibrating mass, which probably lead to an increase of dynamic effects.

3.3. Full circular tour

The effects of the vertical load reached a maximum value either for full loading or for alternating spans between pylons as shown in Fig. 7. Again, prestressing of the main

suspension cable and the hangers is virtually unnecessary, due to the short distance of the pylons. The backstays require pretensioning of about 3000 kN. Since the whole structure is elevated, the wind load is slightly higher than for alternative 2 and varies from 1.46 kN/m² on the pylons, 1.57 kN/m² on the main suspension cable to 1.99 kN/m² on the walkway deck.

In ULS the unity-check shows all parts are designed to maximize the material, since the values vary from 0.7 for backstays to 0.85 for lateral tube profiles to 0.87 for the pylons. The exception is again the main suspension cable with a unity check value of 0.23. The same reason as for alternative 2 applies.

The stiffness of this structure is higher than for alternative 2, since the vertical deformation due to live load equals 123.3 mm / 628319 mm = 1/425. This is still flexible, albeit the structure is almost fully cable supported and less hybrid as the former alternative

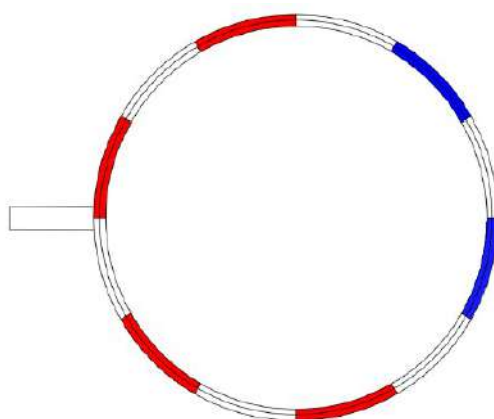


Figure 7 : Variable load on full circular tour

4. Dynamic effects

4.1. Relevant types of dynamic effects

In this type of footbridges, two important dynamic effects may cause failure or have detrimental consequence. First, the effect of wind may generate vortices and thus resonance or at least unacceptable structural accelerations. Thus failure or fatigue may cause collapse. The vortices may originate directly from wind or be combined as wind-rain vibration. A second threat to the structure are human induced vibrations. The latter are caused by walking or running people, including the lateral effect of modifying the centre of gravity of people's body. If the frequency of these movements approaches the natural frequency of the structure, resonance can occur. At least the structure's accelerations due to this effect should be determined and compared to acceptable limits.

Table 1 : Critical ranges for natural frequencies of footbridges according to [3]

Type of mode vibration	Critical range (Hz)
Vertical	$1.25 \leq f \leq 2.3$
Lateral (torsional)	$0.50 \leq f \leq 1.2$

In summary, the procedure for vortex shedding consists of the following steps. First the natural frequencies and free vibration modes should be determined. The nature of the vibration modes should be carefully determined, in particular whether the vibration is in the vertical, horizontal or torsional sense. A first test is to compare the frequencies with the critical range in [3] as mentioned in Table 1.

In addition, the natural frequency has to exceed 5 Hz to be absolutely free of human induced vibration. The real vibration accelerations can be found as in [4]

$$a_{\max} = \frac{p^*}{m^* 2 \xi} \quad p^* = \frac{2 p L}{\pi} \quad m^* = \frac{\pi L}{2} \quad (1)$$

In these equations p and m are the generalized load and mass and ξ is the damping factor. The value of the pedestrian load P is prescribed in [3]. From this value the value of p is determined by :

$$p = P \cdot n' \cdot \psi \quad n' = \frac{10.9 \sqrt{\xi n}}{L} \quad (2)$$

The value of the reduction factor ψ can be found in [3]. Recommended limiting values of the accelerations are 0.7 m/s² in the vertical sense, 0.2 m/s² in lateral sense or for torsional movement and 0.4 m/s² for exceptional crowd conditions.

Assessment of the wind induced vibrations consists of determining the critical wind speed from equation (3) :

$$V_{\text{crit}} = \frac{b \cdot n_i}{S_i} \quad (3)$$

b being the depth of the bridge deck, n_i the i -th frequency and S_i the Strouhal number of the structural element under consideration. In case of cylindrical cross-sections the number is found from equation (4).

$$S_i = \frac{D \cdot n}{V_{\text{wind}}} \quad (4)$$

D being the cross section diameter, n the vortex shedding frequency and V_{wind} the corresponding wind speed. The degree to which wind vibration actually occurs depends on the structural damping δ_s , the linear mass m and is expressed by the Scruton number S_c as in equation (5)

$$S_c = \frac{2 \delta_s \cdot m}{\rho \cdot D^2} \quad (5)$$

If the Scruton number is lower than 20, no wind vortex will be generated [4].

4.2. Dynamic effects of alternative 1

The lower range of frequencies for the structure correspond to free vibration of some of the vertical hangers. The modes, corresponding to the lowest frequencies of the various bridge parts are shown in Table 2. Fig 7 exhibits the lowest mode for vertical vibration of the walkway deck and vertical hangers. The Table also mentions the accelerations due to human induced vibration.

Table 2 : Frequencies and accelerations alternative 1

Bridge part	frequency	Type of vibration	Critical	Max acceleration (m/s ²)
Lateral tubes deck	0.55	vertical	yes	0.023 m/s ²
Lateral tubes deck	0.57	Lateral-torsional	yes	0.066 m/s ²
Vertical hangers	0.64	Vertical	Yes	0.043 m/s ²
Lateral tubes deck	1.07	Torsional	yes	0.161 m/s ²

None of these accelerations may disturb neither pedestrians nor the structure itself.

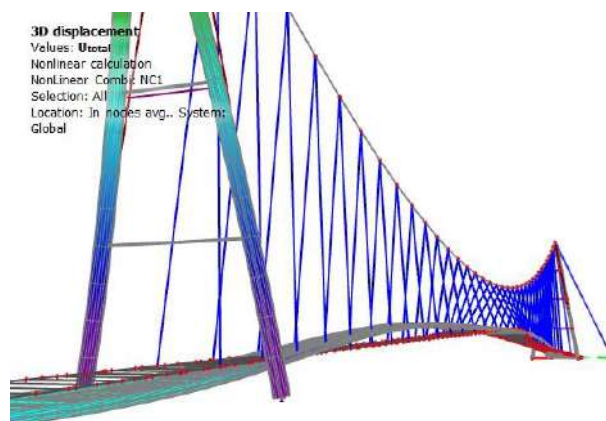


Figure 8 : Fundamental mode alternative 1

Concerning vortex shedding, the Strouhal number equals $S_t = 0.18$ and the Scruton number has been derived in Table 3. As can be read from Table 3, all values of the Scruton number exceed 20; the pylons being closest to any vortex effect. The table also exhibits the maximum displacement due to any possible vibration. These values are also below any critical threshold.

Table 3 : Scruton numbers alternative 1

Bridge part	b (m)	n (Hz)	type	S_c	y_{max} (mm)
Lateral tubes deck	0.813	0.55	vertical	30.48	7.54
Main suspension cable	0.081	0.55	vertical	197.29	$6.26 \cdot 10^{-3}$
Vertical hanger	0.016	0.18	lateral	197.29	$1.24 \cdot 10^{-3}$
Backstays	0.070	0.55	vertical	197.29	$6.11 \cdot 10^{-3}$
Pylons	1.219	3.91	horizontal	23.79	5.97

In conclusion, it is believed that the design of alternative 1 is not sensitive to unwanted dynamic effects and needs no damping of any sort.

4.3. Dynamic effects of alternative 2

For this alternative the vertical hangers are again corresponding to the lowest natural frequency of 0.20 Hz. The first vertical vibration mode of the walkway deck corresponds to 0.78 Hz. Obviously this alternative seems to resist better to vibration. A particular effect is a transversal mode parallel to the line connecting both ends of the semi-circle. Apparently only the anti-symmetrical mode with a single belly and bump, can be generated. Fig. 9 shows the general view of a typical mode, with various simultaneous bumps. Table 4 exhibits the frequencies and accelerations of human induced vibrations.

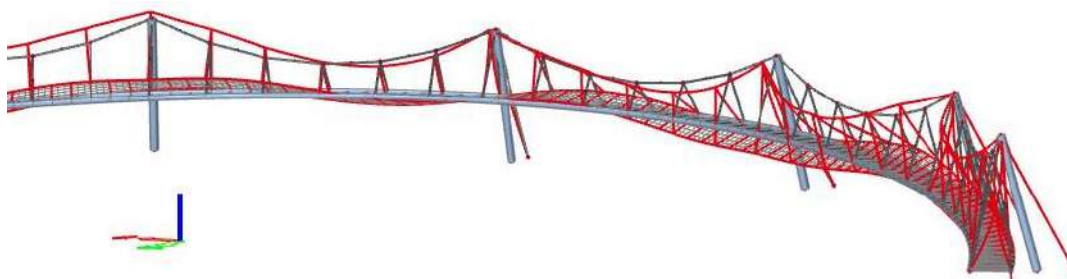


Figure 9 : Typical vibration mode alternative 2

None of these accelerations may disturb neither pedestrians nor the structure itself.

Table 5 exhibits the results of calculations for vortex shedding.

Table 4 : Frequencies and accelerations alternative 2

Bridge part	frequency	Type of vibration	Critical	Max acceleration (m/s ²)
Lateral tubes deck	0.78	vertical	yes	0.076 m/s ²
Lateral tubes deck	0.87	vertical	yes	0.096 m/s ²
Vertical hangers	0.98	Vertical	Yes	0.091 m/s ²
Lateral tubes deck	1.05	Horizontal	yes	0.064 m/s ²

4.4. Dynamic effects of alternative 3

The lowest natural frequency of 0.40 Hz corresponds to a mode of shifting the whole structure parallel to the access ramp's axis. The following modes concern the vertical hangers and show frequencies between 0.48 and 0.62 Hz. At a frequency of 0.63 a general mode shifting the structure along an axis perpendicular to the ramp axis is generated. This demonstrates that the whole cable-suspended structure may move in this direction, the access ramp being the sole fixed location. Therefore an additional horizontal support has been provided opposite to the access ramp. This is materialized by a simple couple of sloping bars as a type of bracing.

Fig. 10 shows the general view of a typical mode, with various simultaneous bumps. Table 5 exhibits the frequencies and accelerations of human induced vibrations.

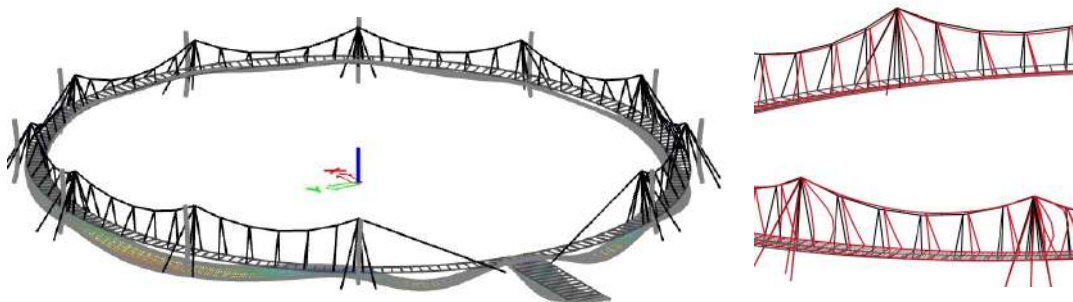


Figure 10 : Vibration mode alternative 3

Table 5 : Frequencies and accelerations alternative 3

Bridge part	frequency	Type of vibration	Critical	Max acceleration (m/s ²)
Lateral tubes deck	0.40	Horizontal // to ramp	no	0.010 m/s ²
Vertical hanger	0.48	lateral	yes	0.028 m/s ²
Lateral tubes deck	0.63	Horizontal ⊥ to ramp	yes	0.020 m/s ²
Lateral tubes deck	0.74	Vertical	Yes	0.049 m/s ²
Lateral tubes deck	0.77	Vertical	yes	0.053 m/s ²
Lateral tubes deck	0.81	Vertical	yes	0.060 m/s ²

None of these accelerations may disturb neither pedestrians nor the structure itself.

Table 6 : Scruton number alternative 3

Bridge part	b (m)	n (Hz)	type	S _c	Y _{max} (mm)
Lateral tubes deck	0.914	0.40	Horizontal II ramp	27.15	25.6
Main suspension cable	0.060	0.40	Horizontal II ramp	197.29	8.01 10 ⁻³
Vertical hanger	0.0267	0.40	Horizontal II ramp	197.29	3.73 10 ⁻³
Backstays	0.060	0.40	Horizontal II ramp	197.29	12.3 10 ⁻³
Pylons	1.219	6.47	Horizontal	23.79	9.14

Table 6 exhibits the results of calculations for vortex shedding. Obviously, strong indication exists that alternative 3 is not sensitive to unwanted dynamic effects and needs no damping.

5. Comparison of cable-supported and arch type structures

Since this conceptual research supplements previous proposals of arch-type walkways, a comparison is of capital importance. The 3 cable-supported alternatives are by far more flexible than their arch-type counterpart. Hence, it was expected that consistent damping measures should be taken to avoid vortex, generated by wind, as well as human induced vibration. None of these appear as the accelerations are all within acceptable values. Tables 7 to 9 exhibit the relevant values after comparing. Obviously, the alternatives 2 of both types are fundamentally different and the comparison may be qualified as inaccurate.

Table 7 : Compares arch-type and cable-supported alternatives 1

characteristic	Arch-type alternative 1	Cable-supported alternat. 1
Mass per unit length of main cable	NA	40.65 kg/m
Steel Grade	S 460	S 460
Diameter lateral tube profiles	1100 / 1300 mm	813 mm
Thickness tubes	80 mm	16 mm
Crossbeam short type	CHS 500/40	HEA 240
Crossbeam long type	CHS 500/40	HEA 360/500
Vertical deformation due to live load	16 mm	426 mm
Max value unity check	0.97	0.97
Fundamental frequency	0.29 Hz	0.55 Hz
Type of lowest mode	2 wave lateral	1 wave lateral
Scruton number lateral tubes	94	30.48
Max vertical acceleration	0.0035 m/s ²	0.1609 m/s ²
Max horizontal acceleration	0.0069 m/s ²	0.2651 m/s ²
Dampers needed ?	No	No

For both type 1 structures, the lateral tubes are heavier in the case of the arch-type. This implies the mass of structural steel is considerably higher. However, the cable steel is more expensive than regular structural steel. The arch-type alternative has a limited range of flexibility, and the substantial mass of this construction accounts for its lower frequency. Additionally, compared to the cable-supported option, the Scruton number of the arch type is significantly higher, which helps to explain why the accelerations for both vortex shedding and human-induced vibration have clearly lower values. All things considered, the arch-type structure seems to perform better.

Table 8 : Compares arch-type and cable-supported alternatives 2

characteristic	Arch-type alternative 2	Cable-supported alternative 2
Mass per unit length of main cable	NA	22.20 kg/m
Steel Grade	S 460	S 460
Diameter lateral tube profiles	1100/1300 mm	914 mm
Thickness tubes	80 mm	16 mm

Crossbeam	CHS 700/30	HEA 220
Vertical deformation due to live load	41.3 mm	186.3 mm
Max value unity check	0.97	0.82
Fundamental frequency	0.64 Hz	0.78 Hz
Type of lowest mode	General torsion	1 wave vertical
Scruton number lateral tubes	16.12	27.15
Max vertical acceleration	0.00082 m/s ²	0.0962 m/s ²
Max horizontal acceleration	0.00082 m/s ²	0.0638 m/s ²
Dampers needed ?	yes	no

Both second alternatives are hard to compare. However, the arch-type option again has a higher structural steel mass per length of the structure. There is no basic issue because torsion is the fundamental mode for the arch-type and this form of vibration can be successfully and simply suppressed by sealed tuned liquid column dampers [2]. A semi-circular cable-suspended structure, however, might actually be more cost-effective than a tripod.

Table 9 : Compares arch-type and cable-supported alternatives 3

characteristic	Arch-type alternative 3	Cable-supported alternat. 3
Mass per unit length of main cable	NA	22.20 kg/m
Steel Grade	S 460	S 460
Diameter lateral tube profiles	480/500 mm	914 mm
Thickness tubes	12 mm	16 mm
Crossbeam	CHS 200/10 300/14	HEA 360/500
Vertical deformation due to live load	40.3 mm	123.3 mm
Max value unity check	0.86	0.82
Fundamental frequency	1.48 Hz	0.40 Hz
Type of lowest mode	torsional	1 wave lateral
Scruton number lateral tubes	18.90	27.15
Max acceleration	0.13 m/s ²	0.06 m/s ²
Dampers needed ?	yes	no

A proper comparison is available with option 3. The cable structure's steel walkway is heavier than the arch's superstructure. The mass of the cable system must also be included. However, in the latter scenario, the construction is sufficiently robust, and the characteristics that govern the vibration modes are far more favourable. Consequently, one would be tempted to think the cable structure is preferable..

The comparison mainly considers the technical feasibility, in particular related to dynamic effects. However, both the public and the owners must be convinced about the building of such structures, given their form and character, as they appear above or next to heritage remains. In this context, arch-type structures are less imposing and might be more acceptable. Therefore the structure with the highest both visual and technical quality may well be the tripod arch structure. It also performs higher as a separate structure than any other. However, the complete circular cable-supported structure may be highly appreciated by owners and the public, since it is the least imposing and particularly well adapted to circular historic sites.

Conclusion

Following the development of arch structures that can serve as walkways for visitors above or around historical or archeologic sites, similar typologies have been developed as cable-supported structures. Since cable systems are definitely more flexible than arches, special care has been given to this particular aspect. It appears that the deformations can be relatively large, but that both the aero-elastic stability and the effects of human-induced vibration fall very well within the set limits for comfort and stability. This is the case because both the accelerations due wind and those induced by people walking, are quite low and because the Scruton number is always higher than 20. A comparison of all relevant parameters has been proposed and additional damping is not required in any of the cable-supported systems. In general, the vertical hangers are the most critical parts.

An important question is also whether the owners and operators of historic sites and the authorities would give a favourable opinion on these types of structures, so that they would be considered as an alternative to full covering of sites. The condition for this is that they should not so much attract the attention of the public, but would be modest as a concept and not require an excessive amount of space. In order to meet the latter requirement, one arch-type and one cable-supported type are ultimately proposed as the most promising alternatives.

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МЕЖДУНАРОДНА НАУЧНА КОНФЕРЕНЦИЯ БАНИ'2022 INTERNATIONAL SCIENTIFIC CONFERENCE BASA'2022

CHURCH № 2 OF KOKALYANSKI URVICH

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Abstract: *The archaeological research of Kokalyanski Urvich began in 1969 led by Dimitar Ovcharov, an archaeologist at the National Museum of Military History. During the excavations were determined the dating and name of the church: "St. Prophet Elijah". In 2022, a team of the National Museum of History and the Department of Medieval Archaeology of NAIM-BAS conducting research of the Citadel's Fortress found a new Christian temple dated back to the 14th century. It is significantly smaller in size in comparison with the "St. Prophet Elijah" church, but has a high concentration of mural fragments scattered throughout its ruins. The main goal of this paper is to present a primary review of the newly discovered remains.*

Keywords: Kokalyanski Urvich, christian culture, middle ages, church, in fresco

The studies of Kokalyanski Urvich began in 1969 by Dimitar Ovcharov, then an archaeologist at the National Military History Museum (Ovcharov, Petrunov, 2021, 19-33). As a result of his archaeological research now we know the name of the church "St. Prophet Elijah" and its date. The excavations of the Citadel of the Fortress started in 2021. In season 2022, a team of the National Museum of History and the Medieval Archaeology Department of NAIM-BAN, during their research discover a new Church at the same place. It dates from the 13th-14th century. The Church is significantly smaller in size, but with a high concentration of mural fragments scattered throughout its ruins. The subject of this publication is to present the documented remains and a primary review of the new data. The initial plan of the survey was to trace the probable enclosure wall, the rampart of the Citadel, from the east to the west Urvich Hill. As

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soon as the excavations began, it was established that we had come across a Church¹. The destructions and the treasure hunter dig were the reason to choose this place. This allowed us to outline a plan of a facility with dimensions of 4.90 x 8.55m. The building is placed on the very edge of the plateau, after which an almost vertical slope begins. The dimensions of the Church are very compromised by various aggressive impacts, duration of the destruction, vegetation and the natural slope of the terrain. The width of the walls is from 0.75m to 0.90m in depend of the level of preservation, and the construction is double-faced with a white mortar joint. In terms of construction, many reused materials from Antiquity were found. Bricks and tiles embedded in the structure, but not as structurally defining elements, but as separate segments inserted during the construction, as if accidentally found and reused from an earlier construction. The entrance from the west is 1m wide and was formed with an arch of slab stones, destroyed in place in the passage. They have the same presence of ceramic inserts, without a specific sequence. Between the dozen cleared slabs, three bricks with a thickness of 0.03 m, located in an uneven sequence, stand out to form the arch. The depth of the apse is not big in size - 0.96m, and yet it came across a carefully shaped base for an altar table with dimensions of 0.80x0.60m. This make Church No. 2 not a chapel, but a functional Church building in which all type of services took place. Violeta Dimova wrote in detail about the medieval religious architecture in her work (Dimova 2008).

At the very first moment of the initial scraping of the sediment, a large amount of highly destroyed mural fragments was found. Their findings can be localized throughout the study area, but the highest concentration is in the interior of the church, as well as from the west, where the building is dispersed in a western direction and, along with the stone of the superstructure, numerous plasters and coloured frescoes have been found. As we know, the foundation of the Second Bulgarian Kingdom was precisely connected with the Christian culture. Only on Trapezitsa in Veliko Tarnovo are documented more than twenty churches related to the period of the 13th-14th centuries. (Dimov, 1915, 130-137.)

At this stage of the research, it is difficult to determine the date of the wall painting of the Church in the Citadel. We provisionally fix the dating between the 13th and 14th centuries, due to the lack of detailed clarity, at this early stage of the research, upon the materials involved in the technology for the preparation of the wall paintings. The base of information is constantly changing and being supplemented in the direction of interdisciplinary - technical-technological, colour, art history-iconography, historiography, ethnography and other studies. The fragments collected during an archaeological survey "in situ" are conditionally sorted according to their location, being studied in groups. Under visual, microscopic examination, readable plaster layers are documented, which are clearly distinguished from the fine, upper layers. And here, as in most medieval buildings, fine-grained sands are characteristic of the lining plaster. The plaster is hard, dense, grey with coarse sand and pebbles. The plasters of all the fragments are calcareous, but

¹ I would like to thank to the whole research team and especially to the Excavation Director Violina Kiryakova

with different sand content, as well as with different fineness of the sieved sand. Visibly rough, it is clearly distinguished from the other, calcareous - fine plaster (intonaco). The levelling, lining plaster has significant sand impurities, while the layer on which the painting rests has a much finer structure, consisting primarily of white lime, with minimal amounts of impurities - grains of sand. As a feature of the temple "St. Elijah" here in the lime plaster we find an organic filler. Its composition significantly lightens the second layer and firmly attaches it to the lower one. The building plaster consists of well-sifted mortar - fine sand, straw (grass) and, above all, lime. There are only traces of organic impurities, plant fibres and stems have been decomposed. In places, several layers of lime smears with chalk can be seen, aiming at stabilization and alignment. The upper surface is polished, fully in the traditions of ancient and medieval (Byzantine) technologies. It can be said that the amount of organic filler in lime plaster is significant. As on the last, polished layer, the mural decoration is executed.

The pigments used in the paints decoration of the Temple are made of natural materials. The paint at the bottom is applied ethereally. It has a very fine structure, with the exception in few places it thickens is with a dense texture, where traces of the painter's brush can be observed. Colour pads are applied evenly. The analyses of the material display partial use of classical "in fresco" which was modified as "in secco". Usually, the technological implementation of the mural decoration took place in stages. The first one included the application of graphics and "in fresco" - technique, with the application of preparatory colour pads. This was carried out in parts, since the fresco technique required a quick application of the paints on the still wet plaster. The final completion of the murals was done by applying paints with the addition of binder. What are its characteristics and its composition would be determined by the subsequent physico-chemical studies. The architectural and construction data complement the specifics of the stone structure of the wall with mortar from the building and the brickwork. Registering the repairs and renovations would also provide additional clarity about the life of the Church. The organic material of the structural elements – wooden beams traces of the iconostasis and other structures are parallel topics to the future research process.

During the survey of Church No. 2, a small number of artefacts were found, but their discovery in between the destructions to the east and northeast of the Temple, helped to precise its dating. Between them are three triple, openwork earpiece elements, made of gilt silver, with the characteristic filigree and soldered beads. The preliminary dating of the ornaments is the 14th century, and their location is between the destroyed columns of bricks and mortar that blocked the altar niche. Another important highlight among the finds is the burial of an ecclesiastical vessel, almost centrally in front of the apse, from the east. A cross and a lamp were buried in a small pit, partially surrounded by small stones. Both objects are made of iron, and their hoarding must be dated to the time after the end of the functioning of the building. This most likely happened at the very end of the 14th century or later in the 15th-17th centuries, and we must note

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that the burial falls at a height above the contemporary terrain of the building, at the level of the substructure. Another important highlight of the study of Church No. 2 is the unearthed grave of a young woman located northeast of it. The grave is very disturbed as a result of bioturbation, but the earrings found on the spot, finely made of silver, date it to the end of the 14th century. Thus, together with the representative fragments of the fresco decoration of the Church, its architectural features and the objects discovered so far, a dating within the 13th-14th century can be established. The refinement of the chronological duration and the establishment of the date of construction of Church No. 2 will be possible after the complete uncover of its interior and the exhaustion of the layers in the surrounding terrain and in the Church itself.

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Fig. 1 – The Citadel of Kokalyanski Urvich, view from the east.



Fig. 2 – Church No. 2, view from the southeast.



Fig. 3 – The apse space, view from the west.



Fig. 4 – Burial of relics, cross and lamp, in front of the apse from the east.





Fig. 5 – The relics, cross and lamp, after restoration.



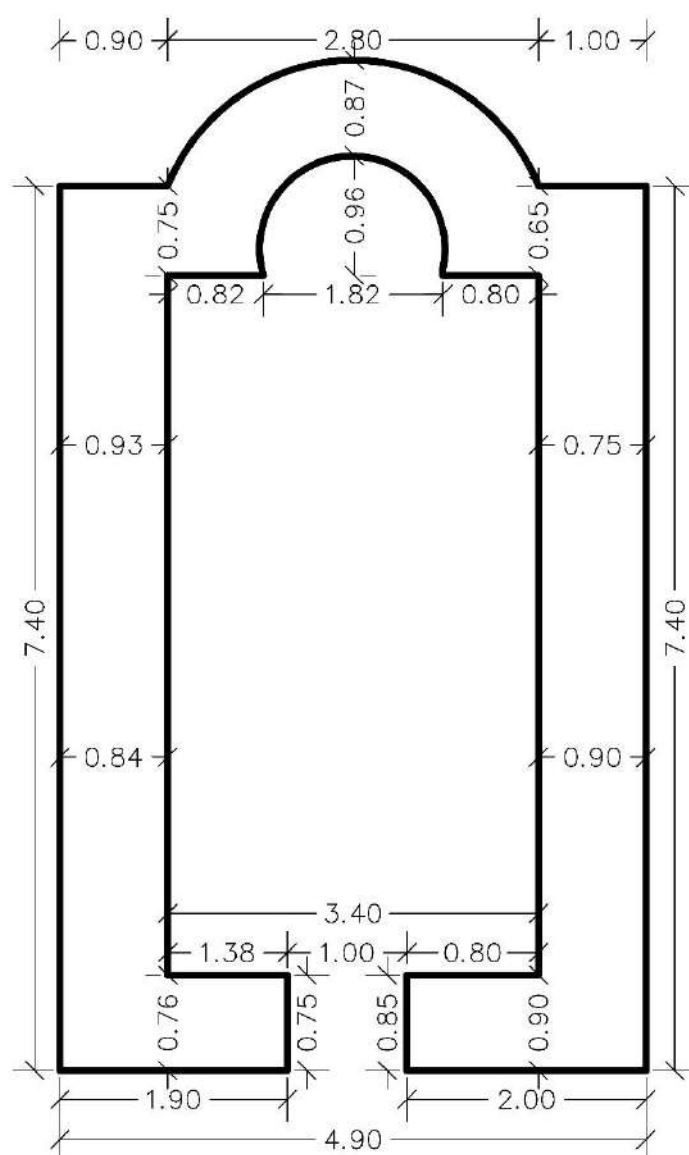


Fig. 6 – Measurements of Church No. 2.

RECONSTRUCTION, MODERNIZATION AND BUILDING OF FOOTBALL STADIUMS IN THE SLOVAK REPUBLIC

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Abstract: *Roof constructions of football stadiums must mainly comply with safety criteria. In terms of security risks, they must therefore meet the strictest criteria. Durability is also an important factor. At the same time, they should also meet architectural requirements, which are often influenced by the budget. The development of new technologies and materials is also reflected in various roofing projects for football stadiums in Slovakia.*

Key words: *football stadium, reconstruction, modernization, building*

1. Introduction

The modernization of football stadiums in Slovakia has moved forward significantly in the last ten years. It was a logical development. Almost all football stadiums in Slovakia did not have adequate technical infrastructure. The only stadium that gradually underwent reconstruction was the MŠK Žilina stadium. It was also used for matches of the Slovak national team. Some stadiums were no longer suitable even from the point of view of safety and their parts, or the entire grandstands had to be taken out of use.

Their maintenance, as well as inappropriate structural solutions, had a significant impact on the state of the supporting structures of the stadiums. That is also why the government of the Slovak Republic by its resolution no. 115 of February 27, 2013 approved the financing of the reconstruction, modernization and construction of football stadiums for the years 2013-2022 in the amount of 45,000,000.00 euros. By concluding a contract between the Ministry of Education, Science, Research and Sport of the Slovak Republic and the Slovak Football Association, stadiums were selected that will be able to draw funds for their renovation. Stadiums were still divided according to UEFA criteria. Two stadiums in Žilina and Trnava were included in category 4, the others in categories 3 and 2. It was a significant shift in the modern era of modernization of football stadiums in Slovakia.

2. Safety and durability

Football is becoming a bigger and bigger phenomenon. It is a source of huge funds

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and some gigantic constructions of stadiums are connected with it. Most of the stadiums in the respective countries have been modernized or built new, mainly in connection with the organization of international competitions - European or world championships. The development of new technologies, materials and construction procedures has also significantly affected this sector. In many countries, modern stadiums are their pride and rank among important buildings that testify to the strength of the economy or the maturity of the nation. However, safety and durability are the most important attributes of all types of load-bearing structures.

In football stadiums, this aspect is all the more significant, because in these objects there is a large accumulation of people - in the order of tens of thousands. An important part of football stadiums are the roofs of the grandstands. Most of the stadiums that are included in the highest category with four stars must have all the grandstands covered. It is a significant shift in terms of protection of grandstands from the effects of the weather. Grandstands are the most heavily loaded structures. Therefore, their safety comes first. Whether the constructions of the grandstands are designed from concrete or steel, their roofing does not cause a significant degradation of the materials as with uncovered structures. New technological procedures in the field of concrete prefabs contributed significantly to the safety and durability of tribune structures. Most of the stadiums built in recent years have grandstands made of prefabricated elements made of high-quality concrete. Prefabrication has enabled the realization of elements of various shapes and sizes in recent years, but mainly contributes to increased safety and durability. Even from an architectural point of view, such elements are preferred. The proportion of prefabrication compared to old stadiums is incomparable. However, it should not be forgotten that despite this, it is most important to design the right type of prefab for the given part of the structure and to choose a proven type of detail. When assessing prefabs, their verification at the I. ultimate limit state and II. serviceability limit state is very important. In this regard, the dynamic stress on the grandstands caused by the crowd plays a significant role in safety.

Another important part of the stadium is its roof. In most cases, this forms the main architectural element. In most stadiums, these roofs are realized using steel structures of various types. The type of proposed structure depends on several factors. At the beginning of the design project, the architectural requirements are set and their fulfillment in terms of the overall architectural expression and its integration into the site in question. However, the roof must again meet the criteria set for safety and durability. A properly designed static system and the use of suitable materials is the first prerequisite for a building to be safe and to have the required lifespan. Climatic changes play a significant role in the design of the roofing structure. Even in European conditions, we are already exposed to extreme weather fluctuations, especially the effects of wind. Therefore, we must prepare for situations that cannot be underestimated in connection with the effects of frequent storms or hurricanes. This issue is related to the reliable design of the tribune roofing. An important role is played by the dynamic component of the wind and its flow. For such important buildings as stadiums, it is advisable to prepare a wind study and verify the roof in a wind tunnel. The costs associated with such verification are incomparable to the level of risk that may arise from an inappropriately designed construction. That's why it's important to appeal to investors to have such a wind study drawn up already during the proposal.

An integral part of the project documentation for such important buildings as stadiums should be a statics audit. An independent assessment of the audit is thus another important one point that contributes to the safety of the building. It is important that the checking structural engineer (Prüfingenieur) - auditor has experience with similar buildings. An example is the established and functioning static control procedures in Germany.

Even if all the conditions associated with the design of the stadiums are met, the building may not meet the criteria of safety and durability if the quality requirements imposed on the delivery and assembly of the building are not met. In order for the construction to meet these requirements, it is necessary for the selected contractor to meet the strictest quality assurance criteria for such constructions, not only during its implementation, but also during the warranty period of the work. These aspects should already be taken into account during the selection procedure of the construction contractor. The same must apply when choosing a designer. The general designer must guarantee not only the quality of the project but also professional author supervision during the entire construction of the work. The same applies when choosing a construction supervisor. The construction supervisor should have sufficient experience with such constructions. Therefore, in most cases, it is advisable to choose a larger company for supervisory activities.

So, if we can define the main attributes of safety and durability of the supporting structures of stadiums, then they are:

- suitability of the designed static and supporting system,
- appropriate selection of structural elements,
- suitability and type of designed materials,
- performance of control static audit,
- quality of work implementation,
- control activity during the implementation of the work,
- maintenance and care of the building during its lifetime.

3. Supporting structures of the grandstands and their roofs

The design of the support structure of the tribune is primarily influenced by the architectural design. However, it is important that such a structure is also designed safely. A suitable static system of the tribune's transverse structure and global stability in the transverse and longitudinal direction are the basic prerequisites for the reliable transfer of permanent, imposed and climatic (variable) actions. From the point of view of the materials used (with the exception of wooden ones), the support structures of the grandstands can be divided into:

- steel,
- concrete,
- combined.

In the last period, the supporting structures of the grandstands built from concrete are mainly used. The reasons are simple. Concrete is one of the oldest building materials, and in recent years, mainly due to modernized prefabrication procedures, it has been increasingly used. The technical, technological, aesthetic and economic advantages include, for example, permanent color, low maintenance costs and a long durability, which practically coincides with the life of the object. The surface treatment of concrete elements plays an important role here. With a suitable combination of all aspects, brilliant surfaces with surface porosity of up to 1% can be achieved. Fig. 1 shows an example of the combined construction of the western tribune of the City Arena football stadium in Trnava. This tribune was the only one subject to reconstruction. The original main supporting steel frames of the roof were left. The main steel beams of the tribunes were newly realized, on which prefabricated concrete benches for the auditorium were placed. As standard, the benches were equipped with slats for seating. A new ramp was built to install lighting up to a height of 23,788 m.

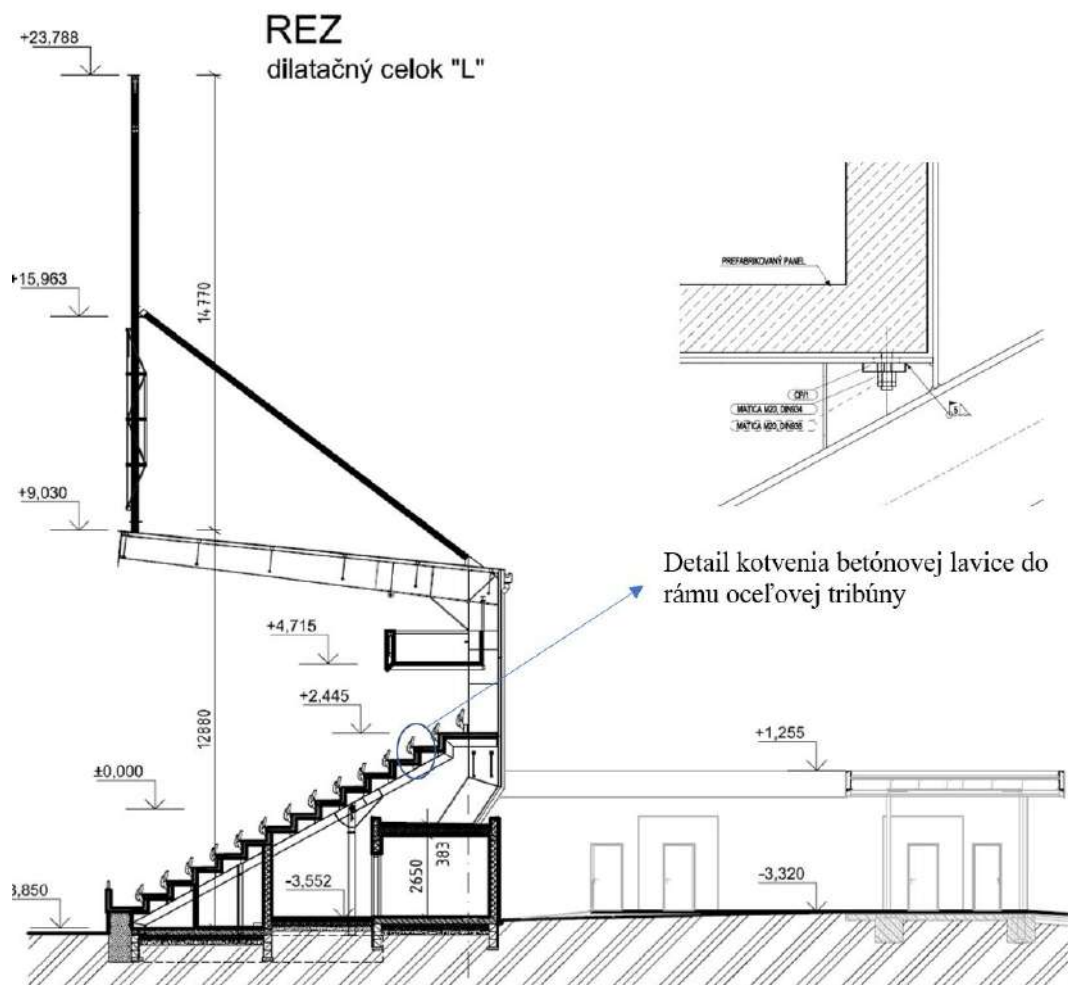


Fig. 1. Transverse link of the combined structure of the west tribune of the City Arena in Trnava. Cross section, dilatation unit L. Prefabricated panel. Detail of the anchoring of the concrete bench to the frame of the steel tribune.

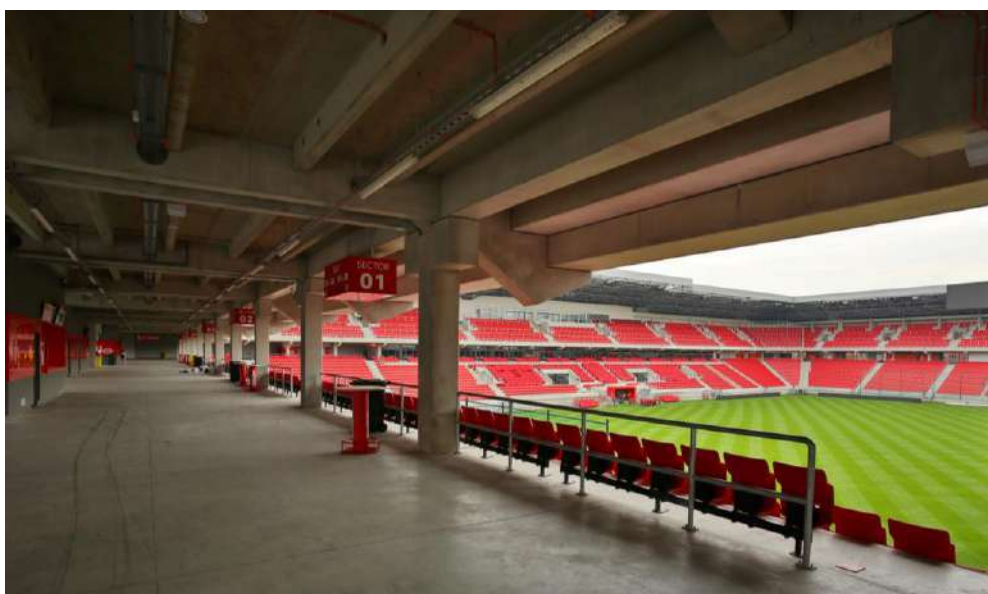


Fig. 2. Example of the structure of the northern prefabricated tribune of the City Arena in Trnava.

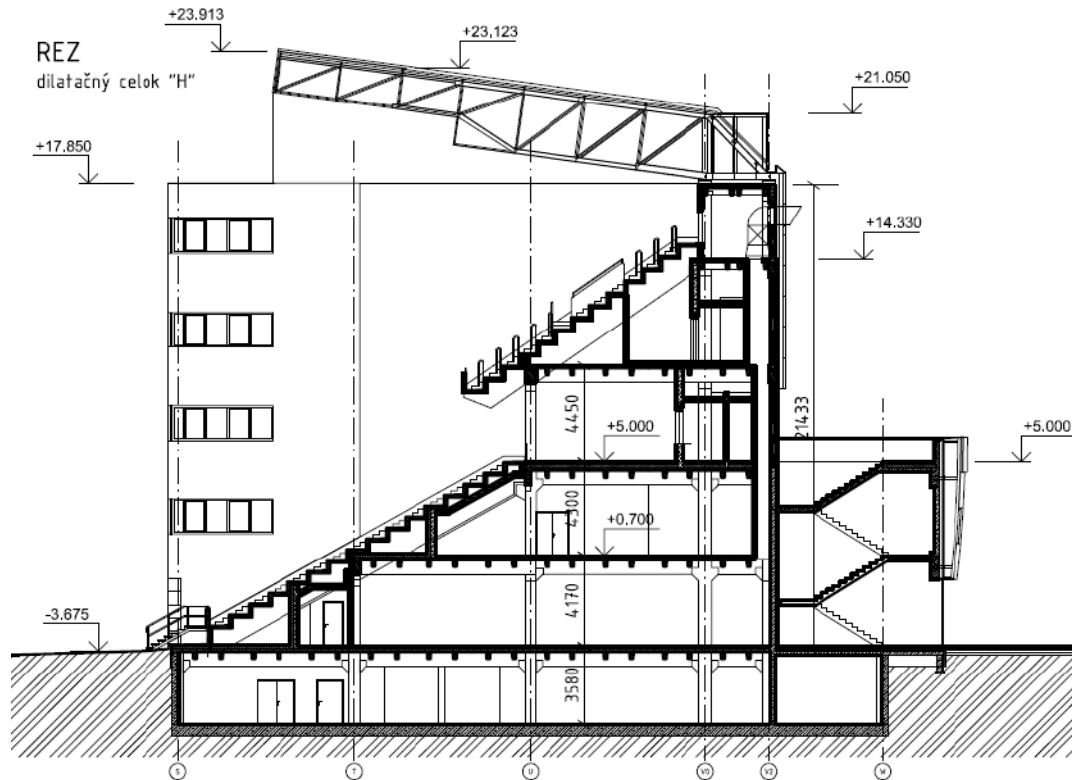


Fig. 3. The supporting structure of the northern tribune of the City Arena in Trnava
Cross section, dilatation unit H.

In Fig. 3 is an example of a solution for a multi-storey tribune of the City Arena football stadium in Trnava. A part of the premises serving as parking areas is scattered below ground level.



Fig. 4. City Arena in Trnava, view of the completed building.



Fig. 5. City Arena in Trnava, view of the pedestrian zone – part of a multifunctional building.

4. National Football Stadium in Bratislava

The construction of the national football stadium in Bratislava was the largest and most significant investment in the modernization of football stadiums in Slovakia. The stadium is built on the site of the former stadium (legendary Tehelné pole – Brick field), thus preserving the continuity of the environment. The project is the author's work of Ing. arch. Kállay. The stadium with a capacity of 22,500 spectators meets the highest criteria of UEFA standards for category 4. There are multi-storey VIP zones, SKY-boxes and congress halls. More than 1,000 parking spaces are available in the stadium underground. The construction of the stadium was carried out by the company Strabag Pozemné a inžinierske staviteľstvo, s.r.o. The entire project also includes commercial parts - an administrative building and a residential complex of buildings "Tehelné pole". These buildings were also realized by the firm Strabag Pozemné a inžinierske staviteľstvo, s.r.o. The construction was completed in 2019. From the point of view of the project solution, the most modern methods were used on the construction, such as the production and assembly of the steel structure of the grandstand roofing and the roof membrane, or in the field of monolithic reinforced concrete structures, the use of COFRA 220 elements and prefabricated structures. From a static point of view, it was a complicated load action, especially in the assembled state. The most complex phase in this issue was the debugging of structures from the point of view of II. serviceability limit state. The company Stavokov projekt s.r.o. provided the static part of the implementation project for the entire stadium and also for both commercial buildings. Work on the implementation project began in the fall of 2016.

The National Football Stadium won the prize in the Construction of the Year 2019 competition in Slovakia for an exceptional and progressive project solution. The expert jury was impressed by the mastery of the demanding static solution of the steel structure of

the grandstand roof with exceptional loading of the cantilever-like parts of the structure, which forms the largest membrane structure in Slovakia and, for the challenging solution, is atypical of the prefabricated structures of the stadium grandstands. The new football stadium Tehelné pole was nominated for the 'Stadium of the Year' award for the best stadium opened in 2019. The award is awarded in two levels. One is awarded by sports architects, but football fans can also vote. The competition is announced by stadiumDb.com, one of the world's leading websites dedicated to football stadiums. Tehelné pole won 11th place from 21 selected participants of the competition.



Fig. 6. Visualization – National football stadium and complex of commercial buildings. Author: Ing. arch. Karol Kállay.

5. Bearing concrete structures of the upper structure

5.1 Tribune beams

The supporting elements of the grandstand, both lower and upper, are designed as prefabricated. They are placed on supporting monolithic frame structures in the column heads or to brackets on columns. The width of the tribune beams is 500 mm and the static height is 900 mm. They are fitted via end teeth with holes formed by HWR $\varnothing$ 80mm, on elastomeric bearings with thickness 10 mm with the prescribed bearing capacity - for the spikes of the column brackets. Holes or HWRs are thoroughly poured with concrete min. C30/37 finer fraction, or better - Emcekret.

Due to the implementation process, the part of the ceiling at the elevation of +14.0 is solved with a pair of prefabricated beams.

5.2 Tribune benches

Tribune benches were mounted on the tribune beams. The clear width is 800 mm and the step height is 360 mm for the lower grandstands up to level +6.00 and 550 mm for the upper grandstands above level +6.00. There are atypical elements resulting from the technical solution at the edges and near the stairs. The basic shape type of the grandstands is the so-called inverted and double-bent-L so called "double-headed", due to the max.

restrictions of horizontal joints. Where, for composition and structural reasons, it was not possible to preserve this principle, the cross-section is simply turned - L.

The first outermost row from the grandstand and on the gallery has the shape of an unequal-armed letter U due to the creation of the railing border. The uppermost row of benches is supported by a prefabricated railing brace. To better ensure watertightness, the joints of the benches are placed in the upper position of the seats, with placement on a rubber sealing profile in the insertion groove. Statically, the benches act as hinged, simple beams with the possibility of deformation due to temperature effects. The material used was: concrete STN-EN 206-1-C40/50-XC4, XF1(SK)-CI 0.4-Dmax16-S3.



Fig. 7 Assembly of the structure of the western tribune and tribune benches.

6. Basic characteristics of the grandstand roof

The roof of the grandstands of the national football stadium is characteristically divided according to the orientation of the world sides into:

- North grandstand
- East tribune
- The western tribune
- South tribune

The steel structure of the South and East grandstands are structurally identical. The structure of the northern grandstand is placed lower so that it does not cause excessive shading of residential buildings. The structure of the western grandstand is mounted on the load-bearing reinforced concrete structure of the roof. The maximum heights of the grandstand structure were defined in the building permission. From this, the heights of the steel structure were derived, which were increased by the value of the deformation from the structure's own weight and the roof shell. The roof shell is made of a PVC membrane.

6.1 Static action and stability of the structure

The structure of the roof of the grandstands of the national football stadium is designed from steel. The basic static scheme for all grandstands consists of a cantilever. From a static, but also an architectural point of view, only the eastern and southern tribunes have the same shape. The layout of the cantilevers is different for individual types of grandstands:

- North grandstand: 23,30 m
- East grandstand: 29,00 m
- West grandstand: 24,38 m
- South grandstand: 29,00 m

The cantilevers are designed as lattice systems anchored to reinforced concrete structures. The cantilevers on the northern grandstand are hinged to a single-bay steel frame anchored to the load-bearing reinforced concrete elements of the grandstands. The rear part of the structure consists of a bracket and a tie rod for anchoring the shell. However, these elements also perform a static function to balance the system. The cantilevers on the western grandstand are hinged to a two-bay steel frame, anchored to the load-bearing reinforced concrete roof elements.

The cantilevers on the east and south grandstands are hinged to the steel column. In the rear part of this structure, a balancing system is designed, designed from a steel horizontal bracket and a system of two truss rods.

The stability of the roof constructions in the longitudinal direction for all types of grandstands is ensured by lattice braces. In the case of membrane constructions, the upper chords of the trusses correspond to the convex curves of the membranes and are offset with a margin so that they do not come into contact with the membrane, which would cause problems with water drainage. Lattice stiffeners also perform an important function during the assembly stage when tensioning the roof membranes, where large tensile forces occur perpendicular to the longitudinal axes of the upper straps of the trusses due to prestressing. In the front part of the roof in the length $L = 12,0$ m, a structure for skylight is designed. This construction is designed from rolled IPE profiles and Jäckel profiles, on which full polycarbonate plates with thickness 10 mm were placed.

6.2 Description of the supporting structures of the roof

The upper chords of the truss brackets are designed from Jäckel profiles. The diagonals of the vertical members and the bottom chords are designed from tubes. Vertical lattice longitudinal stiffeners for a span of $L = 8,15$ m ensure the stability of the brackets in the assembly stage, but mainly perform a stiffening function against the action of the horizontal effects of the membranes. The steel constructions of the cantilevers were made in the workshop, delivered in one piece.

7. Conclusion

This contribution is primarily dedicated to the two largest stadiums in Slovakia. The multifunctional complex City Arena in Trnava, designed according to set of Eurocodes [1], won all the awards, including the main prize in the Construction of the Year 2015 competition. The construction of the National Football Stadium in Bratislava received an award in the Construction of the Year 2019 competition for an exceptional and progressive project solution.

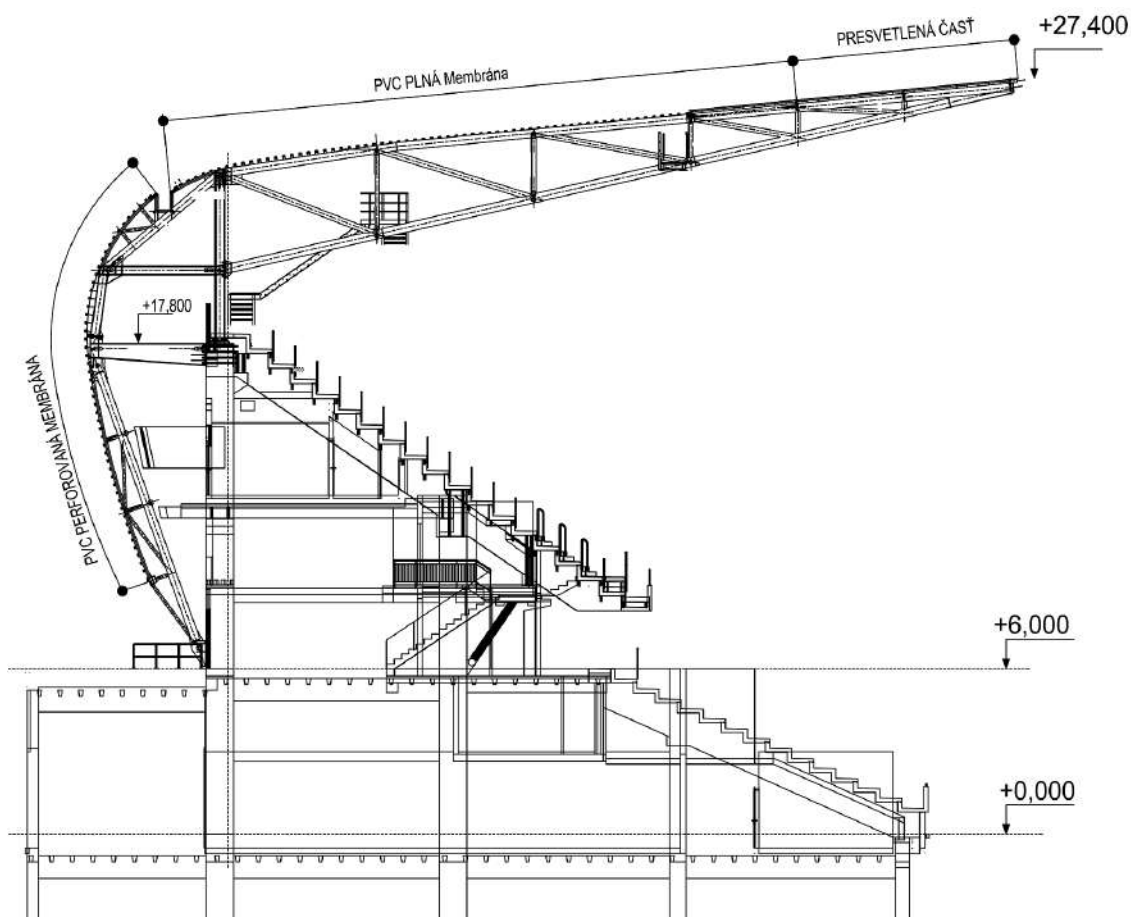


Fig. 8. Schematic cross-section of the eastern grandstand. PVC perforated membrane. PVC full membrane. Lighting part.

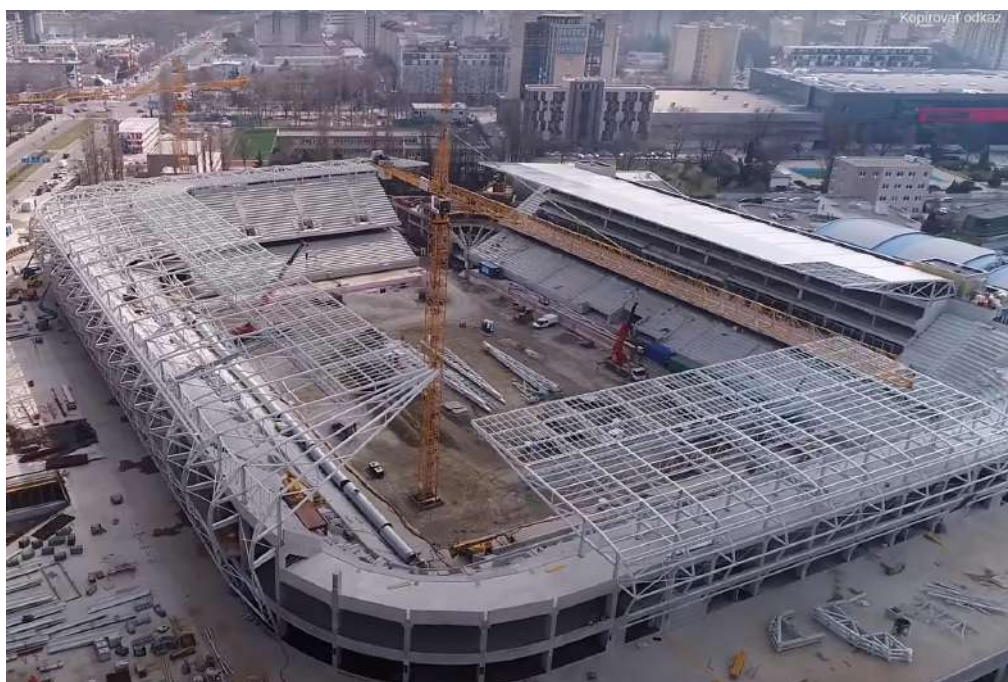


Fig. 9 Instalation of the steel structure of the grandstand roof.



Fig. 10 National football stadium, Tehelné pole apartment complex and administrative building.



Fig. 11 National football stadium in Bratislava, illumination lighting.

Acknowledgement

With this I want to thank to all my colleagues from the company Stavokov projekt s.r.o. for the professional approach, patience and precision in designing the load-bearing structures of such socially significant buildings as football stadiums.

References

- [1] Set of Eurocodes E 1990-EN 1999.

TRADITIONAL RESIDENTIAL HOUSE IN OSOJE

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Abstract

The settlement of Osoje in Travnik represents a unique historical and architectural-environmental complex of traditional residential construction of the Ottoman cultural and civilizational heritage in Bosnia, which at the beginning of the 21st century is under the threat of complete destruction and oblivion.

The remains of the authentic physical structure are few and disappearing rapidly in the daily effort of the local population to achieve, first of all, more optimal, better living conditions, and then to create "architecture" of an inappropriate expression, which was not generated by the legal framework of construction in the historical context, which resulted in degradation wide scale.

The aim of the work is to determine and justify the existence of a characteristic autochthonous type of residential house in the settlement of Osoje, which will enable further procedures for protection, restoration and the use of this spatial unit.

The results of the research-analytical procedure should serve as the initial basis for determining protection and restoration measures, which range from legal preventive protection - recording in local and central institutions for the preservation of cultural-historical heritage, which would further enable preventive protection through consolidation/structural restoration, and finally a complete procedure of physical restoration with conservation and restoration works in correlation with a comprehensive plan for the use of the locality in accordance with the needs of the local community and the wider urban area of the city of Travnik, with an emphasis on the preservation of cultural identity.

Key words: *traditional residential house, Travnik, Osoje, Dimlučara, upper floor, Ottoman period.*

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1. Introduction

1.1. Osoje in Travnik – historical-architectural-urbanistic analysis

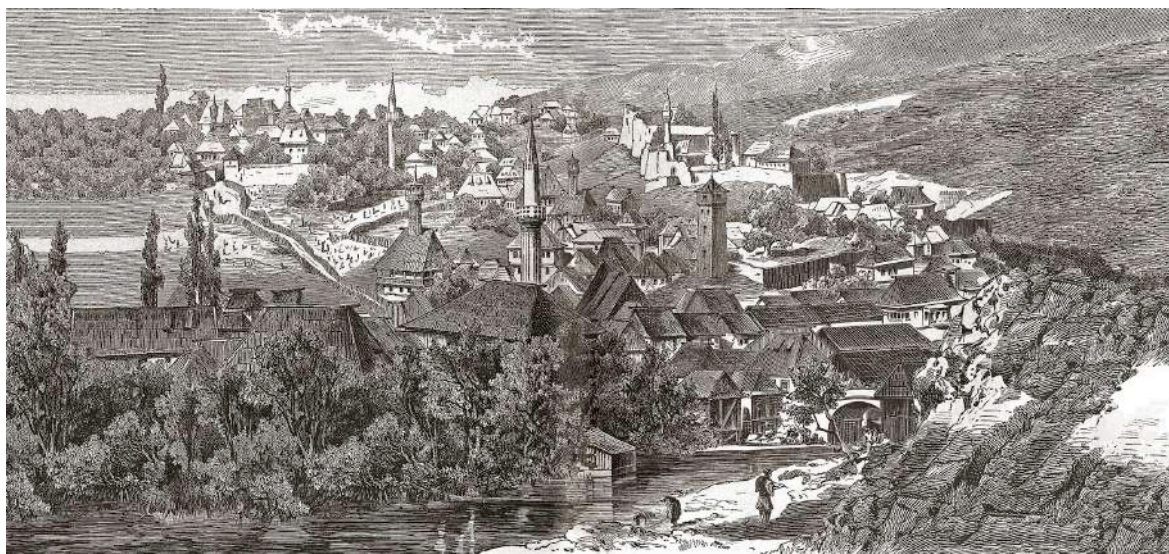


Fig. 1. *Travnik as seen by an unknown artist at the end of the 19th century - view from the settlement of Podilovača, Travnik.*

Travnik has always been a meeting place of different cultural and civilizational influences. Created by the formation of a medieval fortress at the beginning of the 15th century during the reign of King Tvrtko, it soon came under the administration of the Ottoman Empire, and took on all the characteristics of an oriental bazaar modified by local conditions and traditions, which it has mostly kept until today. This is understandable given the four-century rule of the Ottomans and the decentralized administrative and geographical position of the city after the annexation of Bosnia to the Austro-Hungarian Monarchy. Such an existence on the margins and the inadequacy of the urban center of Travnik for transformations according to the principles of European cities resulted in only sporadic architectural interventions dating back to the period from 1878 to 1914 – the only significant impact was played in the development of the city from an industrial aspect. Thus, a city that could boast of over 150 years of viziership gradually stagnates and becomes a kasaba. Nevertheless, the non-enterprising spirit of construction favored the preservation of the physical structure of the historical core. Major destruction occurred in a fire in 1903, and in the period after World War II, when in the first years of reconstruction and in a different socio-cultural atmosphere, in the need to create a mere roof over the head, there was inappropriate construction, the demolition of valuable historical-architectural units and construction of buildings without relation to the traditional and found.³ Moreover, it was traditionally considered "backward", and as such, purpose-built buildings are also demolished – *Konak* (lodging), *bezistan* (covered bazaar, where there are shops with manufactured goods and haberdashery), *han* (a house that serves as an inn and lodging for travelers) and numerous other buildings from the Ottoman period suffered such a fate.

Only in the 60s of the last century, with the awakening of awareness about the values of cultural and historical heritage on a wider level, did such demolition activities stop.

³ See more about history of Travnik, Jovišić, Vanja, "Time Machine : Tvrđava Stari grad Travnik" Zavičajni muzej Travnik, Travnik 2010.

Urbanization of the city begins with an emphasis on the protection of these historical architectural zones and individual construction within them.

Through private investments, this project had a significant impact on the rehabilitation of the old parts of the city.



Fig. 2. *Part of the Osoje village - in the foreground part of the Šumeća village (1892).*



Fig. 3. *Panoramic view from the fortress Stari grad in Travnik to the village of Osoje (2021).*

At that time, recordings were made of most of the preserved old traditional houses in Travnik. Forty eight of them were published in the *Study – Travnik Houses of the Travnik Local Museum* (H. Zeljković). The study served as a basis for the architects to create seven possible typical projects for the selected zones.⁴

"Architectural solutions were made in a contemporary way, with the exterior appearance of an old residential house with the necessary preservation of original exterior materials (stone, lime, *šindra*/wooden cover, carpentry)."⁵

In addition to the construction of typical buildings, a detailed regulatory plan was drawn up in that period, which provided for construction according to the laws of "traditional urbanism".

The implementation of these plans was quite successful at first, but soon there was a disrespect for the postulates of traditional construction – generally at the urban planning level, and then also in terms of design. Unfortunately, today we can witness the anarchy of newly built buildings, as well as projects financed from the public budget – such an example is the high school in the settlement of Osoje.

⁴ Sujoldžić, E., "Travnička kuća". Divan, the newspaper of the cultural society of Muslims „Preporod“ Travnik. Number 14. March 1994.

⁵ *Ibidem*.



Fig. 4. *An inappropriate modern intervention in a historical setting. a) High school in Osoja where one of the traditional residential houses in the Osoja settlement is visible on the left. b) Traditional residential house in the settlement of Osoja. Status recorded in May 2014.*

Although it would be possible to discuss the appropriateness of the construction of typical residential buildings in the 60s last century, which do not interpret the established legalities of building an old Bosnian house into a modern architectural language, but almost completely take over the form, with such detail in the construction that we get confused about the time of creation of these buildings. It can be said that recent interventions degrade the environment to a much greater extent due to the neglect of the spatial relations of the traditional matrix.



Fig. 5. *One of the typical buildings built using traditional building principles. Status recorded in May 2014.*

Approaching the city from the east, on the right bank of the Lašva river,⁶ one notices the medieval fortress, at the foot of which the original *čaršija* (bazaar) developed with the surrounding *mahala* (city District), which are arranged according to the concept of an oriental town with public and sacred buildings (Elči Ibrahim Paša *medresa* founded in 1705/Muslim religious school, numerous mosques – of which the Jeni mosque was built in 1549, *sahat kula*/tower clock, *mezarja*/cemeteries, *turbeta*/mausoleum, *šadrvani*/fountains).

⁶ Lašva is a river in central Bosnia. It flows through Travnik, from west to east, and then through Vitez, and after a total of 49.4 km, near Zenica, it flows into the Bosna River. It is formed by two rivers, Karaulska and Komarska, which join in the settlement of Turbe.

The settlement of Osoje developed on the right side of the river Lašva and represents a typical example of the functional separation of intimate and public traditional urban settlement of the Ottoman cultural climate (*mahala* – city District / *čaršija* – bazaar), with narrow streets and mutual communication between Upper, Middle and Lower Osoje.

While the original residential settlement of Travnik was formed under the castle, Osoje was created later. Although in the immediate vicinity of *Donja čaršija* (Lower bazaar), the area remained uninhabited – the extremely steep terrain there was unsuitable for construction and living. Only when the expansion of trade and commerce in *Donja čaršija* (Lower bazaar) led to the need to expand the city, construction began on the slopes on the other side of the bridge.



Fig. 6. The original residential settlement of Travnik is formed under the castle, and the settlement of Osoja is formed on the right side of the river Lašva. a) The lower bazaar in Travnik - view from the village of Podilovača (cca. 1900). b) View of the settlements (*mahala*) Šumeće and Osoja (1903).

A logical assumption would be that the settlement originally developed along the river itself in the lower part of Osoje, for reasons of direct contact and access to running water. In support of the hypothesis about the origin of the settlement in the lower part, the fact that there was a *džamija* (mosque) across the street from Arnautović's house speaks for itself. However, the architecture of the found buildings and the position of the second mosque, which today exists in the upper part of the *mahala* (city District), tells us something completely different.



Fig. 7. a) Configuration of the terrain of the Osoje settlement. b) View from the direction of Šumeće and Lončarica settlements.

Why the settlement was enthroned precisely on such a high location remains unknown, and one of the possible reasons could be access to a source of drinking water – a well. The well present in Gornje Osoje (Upper Osoje) leads us to this conclusion, and the name *Bunar* (Wells) of one of the streets in the upper part of the settlement is also indicative.

Seen from an urban and architectural-design point of view, the settlement of Osoje was shaped by several basic factors – socio-political, religious, climatic-spatial. The specificity of each of them, directed by further interactions, generates conditions for the development of the form of a characteristic image of the environment, which consists of the multiplied form of a recognizable residential house.

The configuration of the terrain of the settlement of Osoje conditioned the coexistence of the built structure with the surroundings, and the buildings subtly rest on the terrain, without disturbing the views towards Donja čaršija (Lower bazaar). The network of streets, created in accordance with this natural terrain configuration, has remained unchanged, forming three staggered units – Lower, Middle and Upper Osoje.⁷

1.2. Determination of typology

Historical, sociological and natural influences are intertwined in a unique formula – it is impossible to determine the original causes and basic elements of the final form of the traditional Osoj house. The response to the climate and location in the need to simultaneously create – shelters and interaction with the community, space for work, leisure, *akšamluk* (the usual evening sitting), intimacy and openness to nature, presentation of abundance limited by religious and ethical principles, led to the adaptation of the type of country house to these requirements.

The skill of the builder combined the characteristics of urban and rural traditional habitats into a unique appearance, taking something from each in accordance with needs and possibilities.

Osoje houses, as the basic building units of the settlement, represent an autochthonous type, a transitional form between a village house and a city house.

We came to this conclusion in the process of research and analytical work, which included:

- analysis of the original state of the settlement through available historical data;
- documenting the found state of the entire Osoje settlement with the creation of record cards of all found objects. Record cards include general data about the object, data about materialization, state of physical preservation and preservation of authenticity, as well as data about the significance and traditional values of the objects, as well as protection and restoration measures;
- detailed architectural recording of traditional houses that have largely preserved their authenticity;
- analysis of materialization, construction, and analysis of authenticity and degradation on recorded objects.

⁷ See more about *Osoje*, “Osoje u Travniku – historijsko-arhitektonsko-urbanistička analiza”, in: Chabbouh Akšamija, Lemja and Šabić, Lejla, “Tradicionalna travnička kuća : spontano stvoreni prostorni obrasci” (*Traditional Travnik House : Spontaneously Created Spatial Patterns*), Zavičajni muzej Travnik end Arhitektonski fakultet Univerziteta u Sarajevu, Travnik/Sarajevo 2018, pp. 49-55.

There are a total of twenty nine buildings recorded in the settlement, whose creation dates back to the period of the second half of the 19th and the first half of the 20th century, and which have preserved authentic elements in a large percentage range, from well-preserved 8 objects that have been recorded in detail, to objects that have been greatly transformed through renovations, additions, changes in materialization, and only the basic form or some authentic elements are visible.⁸

The analysis of the obtained data led us to certain conclusions in terms of determining the typological characteristics, that enabled the recognition of the development path through comparison with already established types.

The house represents a modified type of central Bosnian one-story country house - "dimalučara".⁹

The indicative elements that led us to this conclusion are, above all, the form, disposition and materialization of the found objects.

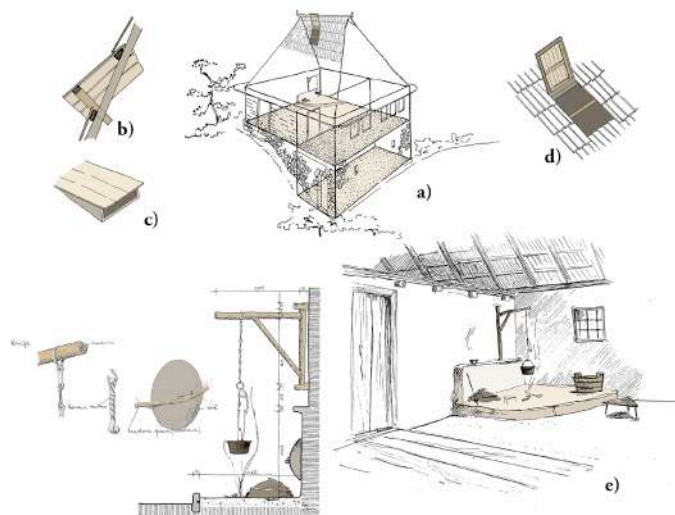


Fig. 8. a) *Spatial-dispositional concept of 'dimalučara'; with constructions of different types of 'badža' (an opening in the clapboard roof of a house through which smoke escapes) - b) 'blind badža', c) 'fixed baža' and d) 'chimney badža' (left and right). e) The space of the open hearth on the ground floor is full of smoke with accented details that define it.*

1.3. Central Bosnian two-story country house

This type of house was established by Astrida Bugarski in her scientific work "Old Bosnian country house with a čardak in the attic"¹⁰.

⁸ See *Ibidem*.

⁹ *Dimalučara* - house with smoke shaft. This is the most usual type of traditional house; it manifests invention of the solution for the numerous basic house problems-organization of space, construction, possibility of the partition, disposition of the smoke...; the smoke is disposed through a shaft (*dimaluk*) and then dispersed freely into the roof space the smoke conserved timber roof construction; the space of *dimaluk* was used for meat smoking; the upper floor rooms are approached by wooden stairs that are added to the house, and on the first floor there is a gallery from which the rooms can be entered; *Abdestluk* (the ablution place) and small storage are located on the gallery, sometimes there is an extension with the toilet as well. See more about *dimalučara*, "Central Bosnian two-story farmhouse", in: *Ibidem*, pp. 58-65.

¹⁰ Bugarski, A., "Starobosanska seoska kuća sa čardakom u potkrovlju", ART 7, Sarajevo 2008.

This type of house was built in the territory of central Bosnia in the second half of the 19th and the beginning of the 20th century.

The population of the villages where it was created called it *čardaklija* and *dvobojka*¹¹ – *čardaklija* after the word of Persian origin – *čardak* (summer house), which described a beautiful, usually two-story house or according to the name of the room *čardak* or according to the name of the *čardak* room, which was located on the first floor of the house. The name *dvobojka* is of more recent origin and simply means a house with two floors.¹²

This type of country house was established for the first time by M. Kadić in his publication "Antique country house in Bosnia and Herzegovina"¹³ and "*Dimalučara* – highly developed type of one-story country house"¹⁴, however without defining the spatial and temporal framework and the cause of its origin.

Dimalučara is the most represented form of an older country house in the entire territory of Bosnia, and in addition to the variety in the materials used, external appearance and spatial layout concept, their basic characteristic is the open fireplace space on the ground floor, the so-called "house", which extends through the upper floor all the way to the attic, from where the smoke is led outside through a smaller opening in the roof. This open space above the hearth – *dimluk*, *dimaluk*, *dimnjeluk* (chimney) had an extremely important function in rural households, it served as a drying room for a large number of foodstuffs.

Although an open hearth with a chimney is not exclusively a feature of a country house in central Bosnia, Bugarski says that the characteristic of an open hearth with a chimney also appears in older village houses.¹⁵

The area of the Bosna river basin is the area with the highest representation of this form, and as a rough territorial border a line drawn from Brčko, through Tuzla, Olov, Sarajevo, Prozor, Bugojno, Travnik, Teslić, Doboj, Gradačac and back to Brčko. This finding is in favor definition of the Osoj house type in relation to the spatial appearance of this country house.

When it comes to the distribution of *dimalučara* in central Bosnia, it was very uneven, it appears together with other types, and in some villages it is not built at all. *Dimlučara* is primarily a house of Muslims, that stratum of the population that was first of all the owner of the land and secure in its territory, and then wealthy enough to afford this comfortable residential house. In addition, this type corresponded to the specific traditional concept of Muslim family life in the community where each married couple had their own room within the main residential building, and the growth of the family he demanded and enabled the reconstruction of the house.

¹¹ Škaljić, A., "Turcizmi u srpskohrvatskom jeziku". Biblioteka *Kulturno naslijeđe*, Sarajevo 1965, p. 164. *Čardaklija*: large room on the first floor of the house with a good view. See more about *čardaklija*, "Materijalizacija i konstrukcija", in: Chabbouh Akšamija, Lemja and Šabić, Lejla, "Tradicionalna travnička kuća...", pp. 415-417, 451. See more about *čardak*, *Ibidem*, pp. 58, 62, 65, 78, 87.

¹² The word represents a coin of Bosnian Turkism (Bos. *dva* + Tur. *boy* = *two floors*). *Ibidem*, p. 146.

¹³ Kadić, M., "Starinska seoska kuća u Bosni i Hercegovini". Biblioteka *Kulturno naslijeđe*, Sarajevo 1967, p. 66.

¹⁴ Kadić, M., "*Dimalučara* - visoko razvijeni tip spratne seoske kuće", god XI, sv. 41-43. Narodno stvaralaštvo, Sarajevo 1972, p. 91.

¹⁵ Bugarski, A., "Starobosanska seoska kuća sa čardakom...", p. 72, drawing 6.

Bugarski further establishes that the occupation of the population had no influence on the appearance or absence of a chimney house as a *dimalučara* in a certain locality.¹⁶ The frequency of the appearance of this one-story building was conditioned by the tendency, i.e. the tradition, to build in a typical way.

As a rule, *dimalučara* has a basement or warehouse, which is built under one part of the ground floor, which was created as a result of construction on sloping ground. It is a very rare case that this type does not have a basement or that the basement is located under the entire area of the ground floor. There is also a rare occurrence of *čardak* (a building with a hipped roof) among the horns.

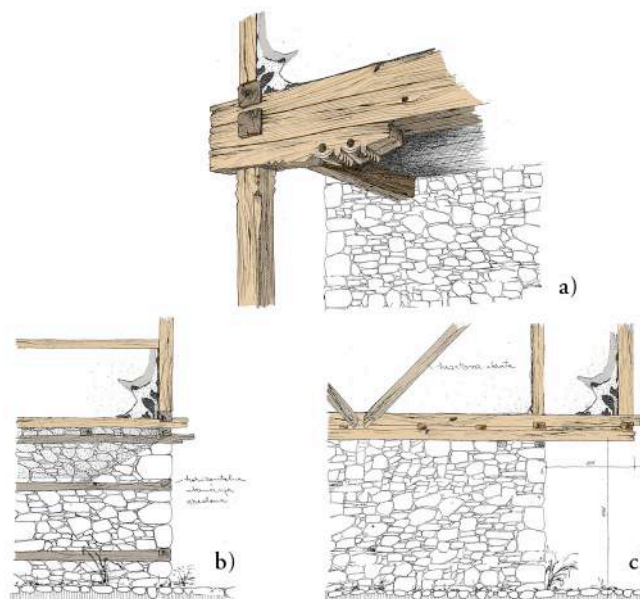


Fig. 9. a) *Istak* (protruding edge) of the upper floor for a *divanhana* (spacious entrance hall on the first floor) with a support column. b) The solution without protruding edge and c) with the projection of the upper floor with and without without additional reinforcement of load-bearing stone walls with oak beams.

The external appearance of the house varies – form appears without the ejection of the upper floor, which is related to the earlier time of constructive expulsions, and with projections, allowances or overhangs (i.e. a wider first floor compared to the ground floor), which appear again on one, two, three or all four sides of the object in the size of 15-60 cm. Also, a frequent phenomenon in later built houses without exits is represented by annexes in which the staircase is located, and open, semi-closed and closed galleries. In the case when the house is shared between several households, external access ramps are added as necessary – *basamci* (staircase). Galleries or so-called *divanhana* (spacious entrance hall on the first floor), *trabozan* (thicker stakes used to fence the staircase), *šerefe* [a fence and a narrow enclosure (like a balcony) on a mosque *minaret* (slender tower of a mosque)] are attached mostly to the first floor and are semi-enclosed, and mostly extend the entire length of the building. The outer *basamci* (staircase) often form a whole with the upper gallery and are closed.

¹⁶ *Ibidem*, p. 41.

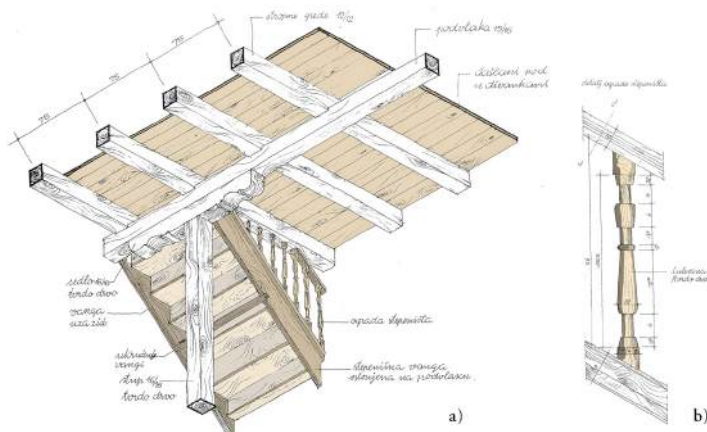


Fig. 10. a) Detail of gallery and one-legged staircase. b) Detail of the staircase railing - trabozan (thicker stakes used to fence the staircase)

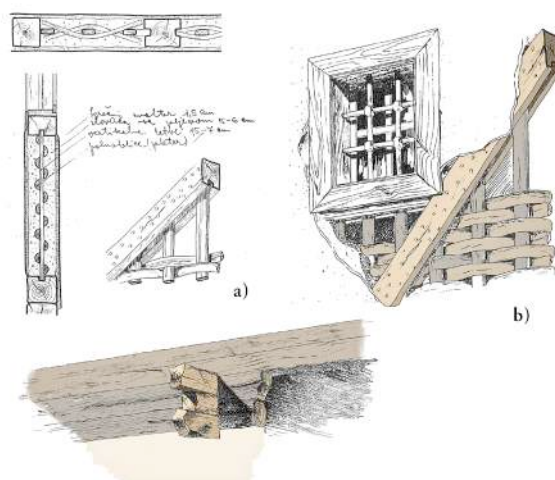
The material used for the construction of *dimalučara* is very different, and there is no difference between this type of building and other buildings built in the same territory. The use and choice of material depended on its availability near the locality and on the climate.

The most common combination of materials for *dimalučara* is a stone wall and a wooden skeleton wall with infilling made of interlaced stakes and wicker, plastered with soil – *šeper*, and a stone wall and a skeletal wall with a filling in one part of masonry boards – also known as *brvna*, and the other part made of *šeper*.¹⁷

In addition, there are infills made of horizontally lowered beams or planks, and vertically lowered beams – beams or uprights. Houses completely built of stone or wooden structures are less common, as are houses with a wooden grill made of adobe (*čerpić*, unbaked brick).

The half-buried basement was most often used as a *magaza* (storage room, basement), and very rarely the inhabitants also kept livestock in them.

The original forms of *dimalučara* on the ground floor had only two rooms – the "house" or space with an open fireplace and the room where people lived, primarily in winter.



¹⁷ See more about *šeper*, Chabbouh Akšamija, Lemja and Šabić, Lejla, "Tradicionalna travnička kuća...", p. 62, 67, 79, 416.

Fig. 11. a) 'Šeper' - a wooden skeleton wall with filling made of interlaced stakes and wicker, plastered with earth. b) Details of the constructive systems of connection and reinforcement of the mezzanine structure, which is wedged into the side walls transversely.

There is a *basamci* (staircase) in the area of the "house" – ladders, balustrades or a bridge for access to the upper floor, or there were no stairs at all, the upper floor was already accessed from the higher ground where the building.

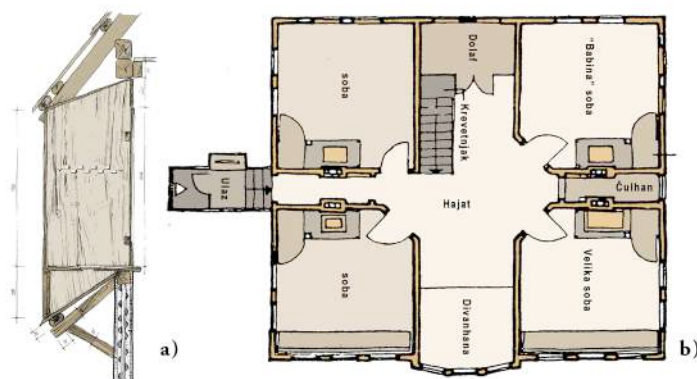


Fig. 12. a) Section on the facade of the house of the added ablution room (space for ritual washing). b) Schematic representation of the floor plan of a large house.

In addition to these two rooms, several smaller rooms appeared later with the development. Some of them were used for living, and for them the name *donja soba* (lower room) or small room became established in relation to the orientation and size, as well as pantry – which can sometimes be used for living or cooking. Her name was *hudžera* or *mutvak*.

On the first floor, there are rooms for living – the *čardak* (the space inside the roof structure), which, depending on the size and orientation, are called upper and lower *čardak* or upper and lower *čardačić* (smaller space). Next to them on the first floor there is also a *hajat* (hallway on the ground floor) as a communication prostor, followed by *abdesthana* (area for ritual washing) and *divanhana* (spacious entrance hall on the first floor) or galleria.

1.4. Comparative analysis - form, disposition, materialization

Analyzing the recording data of the objects found, we conclude that the two-story buildings of Osoj in terms of form, disposition and materialization/construction are a modified type of Central Bosnian *dimalučara* (houses in which the entire space represents a certain type of chimney).

Although, most likely due to the later creation or changes made in the interior, the houses that we approached in the interior did not have an open hearth. However, constructive values are visible that continue from the ground floor to the first floor and then take the smoke to the attic space that serves as a drying room. A semi-underground warehouse is present on all buildings – the availability of stone as a material and the slope of the terrain made it possible to raise the ground floor of the building by creating a storage space and possibly keeping small livestock.



Fig. 13. Part of the interior of a restored *čardaklija*, built before 1840, near Travnik. a) view from the *hayat* (hallway on the ground floor); b) *divanhana* (spacious entrance hall on the first floor); c) large room; d) *basamci* (staircase).

According to the *dimalučara* principle, the ground floor has a characteristic arrangement of rooms – on the front side facing a smaller slope, there are or so-called rooms. lower rooms or large rooms – depending on the size, while on the side facing the higher slope there are smaller rooms – upper rooms or small rooms. From the *hayat* (hallway on the ground floor), you can access the *basamak* (staircase), which leads to the gallery, the communication space. Living rooms are located on the first floor – upper and lower *čardak* (space in the attic of the house) and small *čardak*, with the larger and lower *čardak* always oriented down the slope, on the north side.

The external appearance is also similar in many respects – the form is closed, with an element of an external staircase and a gallery/sofa room. The buildings mostly have highlights on at least one facade, with variants of placing the staircase inside the house or as an annex on one side, leading to the with a gallery – if the latter is the case, the staircase area is closed and forms a whole with the gallery. The difference that occurs in relation to the rural *dimalučara* is the *doksat* (a prominent place on the front of a house with windows on three sides) on the street facade of the buildings that face the street with their southern facades.



Fig. 14. *Čardaklija*, built before 1840. years, surroundings of Travnik.

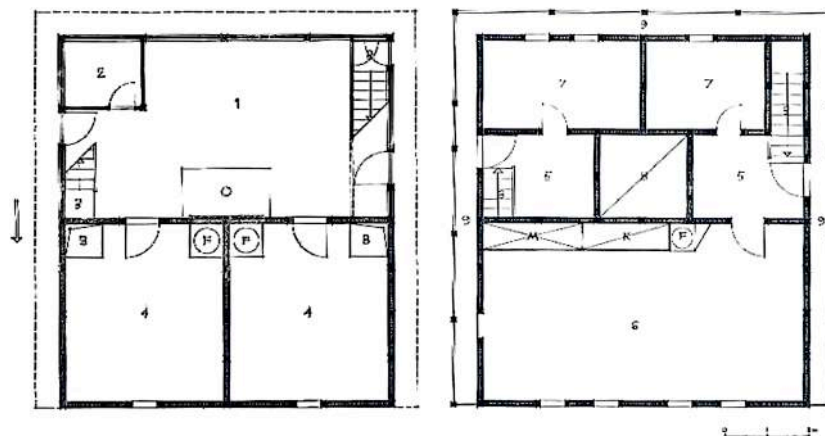


Fig. 15. Floor plan of the ground floor and first floor. 1. "house"; 2. pantry for milk and dairy products; 3. space for leaving shoes and 'basamak' (staircase); 4. lower room; 5. 'divanhana' (spacious entrance hall on the first floor); 6. lower porch; 7. upper porch; 8. chimney; 9. 'trabozanes' (thicker stakes used to fence the staircase); O. hearth; B. 'banjica' (bathroom); F. furnace; K. bed; M. 'musandara' (built-in wall cabinets).

Another exceptional characteristic is the shape of the roof structure. The very steep roof of these houses, which responded to the climatic characteristics of extensive snowfall and long retention of snow on the Osoja side, alternates at the very bottom with a eaves with a milder slope, which is constructively solved very skillfully. The change made it possible to create deeper eaves to protect the building from precipitation, which would have been impossible to achieve if the roof structure had been left with a roof slope of 50-60 degrees. In addition to the role of creating an eaves, the shift also served as a snow guard.

The materialization of the houses in Osoje almost completely corresponds to the type of *dimalučara*, but following the conclusion given by Bugarska that *dimalučara* do not have a specific materialization in relation to other types of rural houses,¹⁸ **we can say that the house in Osoje is actually rural in terms of materialization.** As a rule, the semi-buried area of the basement in contact with the ground floor is built in stone, most often limestone, and local tufa and slate from the slopes of Vilenica (mountain southwest of Travnik), while the upper floors they always represent the skeletal system with variations in fillings. The following infill variants have been registered: infill made of interlaced stakes and wicker, stuck with earth – *šeper*, infill made of horizontally lowered beams or planks, and vertically lowered beams – *dizma* (a wall of wooden beams lined up next to each other). The most common type of wainscoting is where such a variant is represented by the method of cladding – wattle on both sides, a slatted *dizma* or only clay with wood sawdust and reeds as reinforcement. Such an approach had no effect on the appearance of the upper part of the house, which had lime-painted walls.¹⁹

How and why do we come across a type of house in an urban settlement that is predominantly rural in its characteristics?

Given that the occupation of the population had no influence on the spatial occurrence of *dimalučara* at all, we can conclude that the main causes of taking over the *dimlučara* type are: the tendency to build in the same manner – the vicinity of the locality where the *dimalučara* is located, then the specific location of the Osoje site – the steep slope of the terrain and orientation, and finally the availability of materials in the immediate area.

¹⁸ *Ibidem*, p. 33, fig. 8.

¹⁹ See more about **materialization**, "Uporedna analiza – forma, dispozicija, materijalizacija", in: Chabbouh Akšamija, Lemja and Šabić, Lejla, "Tradicionalna travnička kuća...", pp. 66-74.



Fig. 16. The steep slope of the terrain and orientation of the houses towards the north realizing the right to a view, which is one of the basic design principles here.

Encountering the problem of orientation of the hillside part towards the north, the builders responded in the best possible way. As the achievement of optimal sunlight is prevented by the steep slope to the south, the house is completely closed to the south. While all rooms for living and staying are oriented to the north, realizing the right to a view, which is one of the basic design principles here. Also, the steepness of the terrain made it impossible to realize the intimacy of the open spaces within the courtyard. These conditions were optimal for the undeveloped layout and closed floor plan – one of the characteristics of the *dimalučara*.

An urban house with a courtyard gets semi-buried warehouses as a response to the need to raise living space from the ground. This need is twofold – by building a basement, living spaces rise from the damp and cold ground, and in addition, in search of a view and opening down the slope, intimacy is achieved in one's own yard. They are vertically separated as much as possible from the street – thus they are satisfied and intimacy and the opening of views of the bazaar, Lašva and the castle across Lašva.

On the other hand, the availability of stone made it possible to build these massive walls that will carry the weight of one or two floors. All these factors, along with the present tradition of building a village *dimalučara* in the immediate vicinity of Travnik, created ideal conditions for adapting the type of *dimalučara* to the minimal needs of urban housing.

The principle of urban housing is present in the somewhat visible characteristic facade canvas of houses and courtyard walls – and this only in buildings with a southern facade oriented towards the street.

Due to the steepness of the terrain, the walls of the courtyard gates represent a boundary of separation in the classic sense only for buildings with a southern street facade.

Orientation and opening down the slope is respected both in the buildings whose front southern facade faces the street and in the buildings whose northern facade is oriented also towards the street.

The difference in appearance of the street facade defines two basic variants of the Osoja house.²⁰

The first version of the house is located on the north side of the street. The southern street facade is completely closed and represents the outer wall of the first or second floor, depending on the number of floors of the building.

Another version of the house is located on the south side of the street. The northern street facade develops and progressively opens vertically. Living spaces are unusually oriented towards the street in order to achieve a view down the slope.²¹

Such a difference in the street fabric from the south and north side of the alley must have been dramatic, if you walked here at the end of the 19th century, when the settlement was still intact. While on the north side we have a typical *mahala* (city District) scene – a closed courtyard wall and the facade of a low building, on the south side the facades of very often two-story buildings rise steeply.

Depending on the number of floors, both variants of the building have two sub-variants: ground floor and upper floor. One-story buildings are buildings with a more modest disposition and external appearance, while two-story buildings are buildings of larger dimensions, often with emphasized horizontality despite their considerable height.



Fig. 17. Typical 'mahala' (city District) - scene Osoja. One-story residential building with a southern facade facing a greater slope.

2. Variant 1 – one-story residential building with a southern facade facing a greater slope

2.1. Location

²⁰ Due to the limited textual content, we will analyze only the first version of the house located on the north side of the street.

²¹ See more about the *second version of the house*: "Varijanta 2 – stambena spratnica sa čeonom južnom fasadom okrenutom nižem nagibu U ZONI VI" (*Variant 2 - residential floor with a south facing facade facing a lower slope IN ZONE VI*), in: Chabbouh Akšamija, Lemja and Šabić, Lejla, "Tradicionalna travnička kuća...", pp. 83-90.

The building is located in Donje Osoje street, on the ground that is in direct contact with the Lašva river. This extremely steep terrain was used to the best extent – the building cuts into the terrain with a semi-buried warehouse, above which the ground floor and first floor are developed, the lower of which is below street level.

The south wall of the floor is perceived as a street facade. The facade is completely closed and originally turned into a low wall made of roughly pressed tufa blocks to the west and east. Meanwhile, the western wall has completely disappeared – in its place today there is a low concrete sub-wall, while the eastern wall is partially preserved.

The entrance to the building is located on the western side of the building, at ground floor level. The entrance is accessed from a stone landing to which several stone steps lead. On the same side, from a lower level, you can also enter the storage space – *magaza*. The north side of the building opens completely towards Lašva and the garden, which originally descended all the way to the river, with a jutting *divanhana* (spacious entrance hall) on the second floor. Such an extraordinary original location of the building, the harmonious relationship between built and natural environment, has been completely destroyed today by the construction of a more recent building in the northern part of the complex, on the river bank.



Fig. 18. The building is located in Donje Osoje street, built in the Ottoman period. View of the entrance (western) facade. Condition of the facility in 2014.

2.2. Disposition

The building consists of: a semi-buried storage space – *magaza*, a ground floor, a first floor and a roof space that is not used for living.

The storage space in the basement occupies half of the area of the building. Apart from the entrance door, there are no windows for ventilation and the possibility of natural lighting of the space. Most likely, the ventilation window was not installed due to the immediate proximity of the river and the greater presence of moisture. These characteristics confirm the purpose of this space as a basement storeroom with a stable temperature and relative protection from the influence of moisture.

On the ground floor of the building, on the front side facing the smaller slope, there are two rooms – the larger room is most likely the "house" (although no open fireplace is recorded) or the so-called lower room or large room. A smaller room, the so-called the little room or the lower little room, was most likely used for staying in the winter months.

Opposite the entrance, on the side facing the greater slope, there is a smaller room without natural lighting – a pantry or *hudžera*.

The larger room, on the wall opposite the entrance door, has built-in wardrobes (niches for storing dishes), which have preserved their original appearance. On the eastern wall, in front of the entrance to the smaller northern room, there is another door, which in its original state²² led to a smaller closed space in the form of a lateral extension on the eastern facade. This space most likely had the function of a *ćenifa* (toilet). The construction of the space was improvised – wooden with a single-pitched roof, as evidenced by the remains of wooden elements on the facade and incomplete data on the drawings of the original state.

One-legged woodens *basamak* (staircase) are located in the *hajat* (hallway on the ground floor), right next to the entrance to the ground floor of the building. The staircase is projected on the western facade and leads to the upper floor in the communication space, which is separated by a wooden partition from the *veranda* (sofa) hall oriented with openings to the north, with which it forms a unique whole. Two rooms are accessed from the communication area. In a larger room, the so-called in the large porch or the lower porch (due to the orientation on the side facing the smaller slope) with openings on the northern wall, there is a *furuna* (oven) and a *hamamdžik* (home bathroom). A smaller room – the *čardačić* (smaller space) or the upper *čardačić*, is located on the southern front side, with openings in the western wall. Access to the third, smaller room, located on the southeast corner of the house with openings only in the eastern wall, is indirect – from the porch. Both *čardačić* are without *musander* (wall cabinets) and interior elements.

2.3. Construction and materialization

The wall of the basement floor – the warehouse space in the lowest zone in contact with the ground is built of irregular stone, while in the upper zone there are finer pressed blocks and angular tufa elements. The wall is 50-60 cm thick, made in a double layer with a brick filling, bound with lime mortar. The wall structure is additionally reinforced with longitudinal and transverse braces, at a vertical distance of about 1.00 m.

The walls of the upper floors are made as a *šeper – bondruk* system (a wooden skeleton wall with filling made of interlaced stakes and wicker, plastered with earth and wooden skeleton that is filled with unbaked brick – *čerpić*) with infills of wattle and adobe in the greater part, while in some parts the infill appears as a *dizma* made of vertical beams, finally plastered and whitewashed, and an infill of horizontally hammered planks. The parts of the wall, between the *bondruk* structures, were replaced in later periods with full of baked bricks infill, which, due to the heavy weight, led to the settlement of the horizontal wooden beams of the skeletal system.

Pletar appears in several alternations – as a large one on the street and western facade (woven vertically and horizontally) and as a small one on other parts of the building.

The mezzanine construction consists of beams of different cross-sections, wedged between two horizontal beams at the end of each floor, resting on a stone wall.

The roof is hipped, inclined at about 45-50 degrees, and is structurally solved as a roof with purlins. The roof is made in the form of a shift with beams of a gentler slope, supported on the perimeter beam wedding dress of the last floor, in order to direct the water away from the building.

²² The original condition is taken to be the condition recorded in the 60s of the last century, when 48 traditional residential buildings were recorded. See more about, Sujoldžić E., “Travnička kuća”...

Above the original shingles (*šindra*, wood cover), today a steel sheet is forged.

On the north side, the first floor is overhanging by approx. 80 cm.

The staircase that connects the first and second floors is single-legged with two wooden supports – *vange*, into which treads and fronts are inserted. According to the drawings of the original state, the upper landing rested on an external wooden pillar, which has since disappeared. The staircase area is closed with wooden planks, nailed to the substructure, illuminated in the upper part through one smaller window.

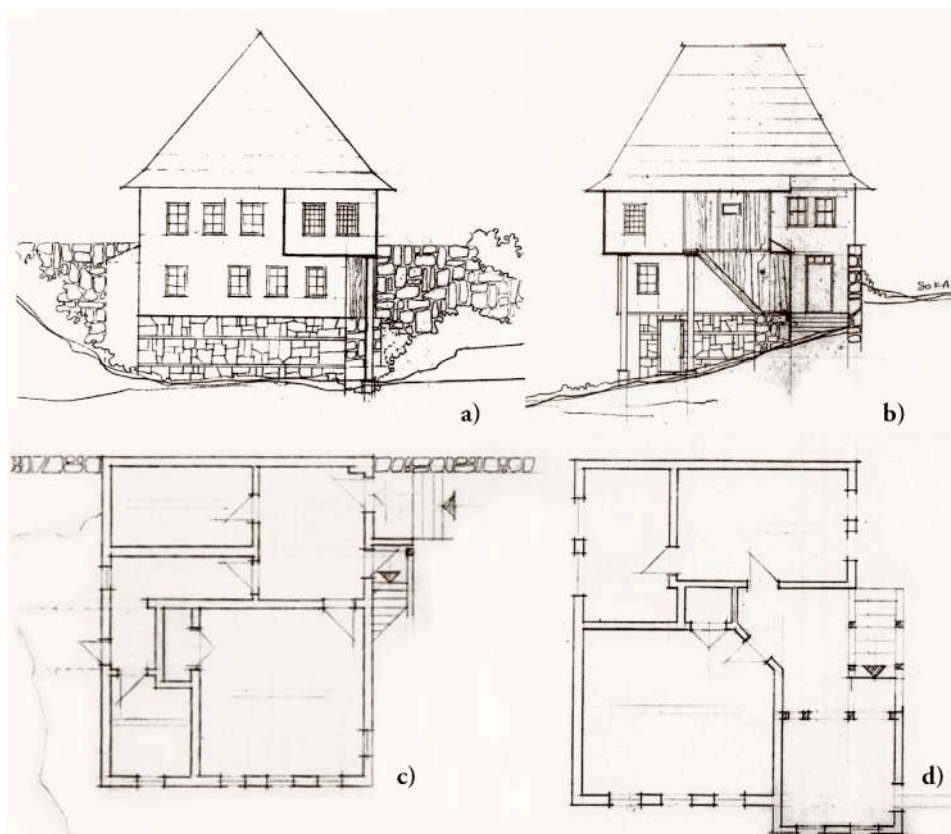


Fig. 19. Drawings of the original state - the building is located in Donje Osoje street. a) North facade; b) Western facade; c) Base of the first floor; d) The base of the second floor. Native museum of the city of Travnik - transferred archival documentation.

The *doksat* (a prominent place on the front of a house with windows on three sides) on the *divanhana* (spacious entrance hall on the first floor) is supported on the northwest corner by a column with a circular section, supported on the ground by an irregularly shaped stone step, which replaced the original column. The window construction consists of wooden frames placed in the raster *bondruk* system (wooden skeleton that is filled with unbaked brick – *čerpić*)²³, with wooden double single-wing and double-wing window sashes. The windows of the *divanhana* have preserved original *demirs* (iron profiles that make up the grid on the window).

The floors are made on the principle of a ship's floor, hammered onto the substructure, while the ceilings are hammered with simply processed wooden planks – *šiše* (ceiling).

²³ See more about ***bondruk system***, Chabbouh Akšamija, Lemja and Šabić, Lejla, “Tradicionalna travnička kuća...”, pp. 79, 80, 88, 416, 418, 420.



Fig. 20. Record card of the building on cadastral parcel 1062. - the building is located in Donje Osoje street.

1) Entrance to the warehouse - part of the western facade; 2) Street view of the eastern facade; 3) Appearance of the northern facade; 4) Street (southern) facade; a complex system with infilling made of horizontally and vertically braided undergrowth; 5) appearance of the northwest facade; 6) He structure of the eastern facade.

4. Conclusion

The town or town house in Travnik has not completely defined its appearance, as is the case with, say, Sarajevo's urban residential house. Very often, the town house in Travnik is associated and identified with the recognizable image of Osoj and its characteristic houses.

Taking into account the position of the location, which is directly connected to the craft and business part of the city, we conclude that Osoje is a residential part, i.e. a *mahala*, a unique functional entity that separates the public and intimate zones of the narrower – urban fabric.

In the search for an archetypal form, in the process of identifying the form, materialization/construction and disposition, the undoubted origin of the Osoj residential house was established in the form of a central Bosnian residential loft-*divanhana*.

The country house here was clearly a model for the creation of a suburban form, and not the other way around – as the general origin of the urban concept of life in the cultural-civilizational sense is always in the rural. It is easy to conclude that the original country house from the Travnik area was taken over as a suitable form in the given specific space-time circumstances and adapted to the extent that it is necessary and possible.

Why was the Osoja house not further transformed into new forms of urban form? First of all, due to the limitation of the locality in terms of spatial expansion, which could lead to further modification resulting from different conditions of the locality and the emergence of new subtypes, and due to the decentralization of Travnik as a regional administrative center. Which also led to stagnation in the expansion of the city and the

disappearance of the population layer that was able to materially contribute to progress and changes in the housing concept.

The arrival of the Austro-Hungarian administration in Osoj left visible sporadic changes in the sense of adapting formal elements to new stylistic forms, but there was no essential change in the spatial concept and application of traditional materialization and construction.

Unable to develop further, the settlement retains its archaic form, leaving traces of the development and creation of the original suburban to this day, which represents a valuable material testimony of the path of the creation of urban settlements and their architectural expression in these areas.

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INSTRUCTION

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A STOCHASTIC APPROACH FOR HISTORIC RC STRUCTURES STRENGTHENED BY TIES TO PREVENT SEISMIC PROGRESSIVE COLLAPSE

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Abstract: Historic old existing industrial reinforced concrete (RC) structures are often subjected to removal of structural elements due to degradation caused by environmental effects. So they are in a risk of seismic progressive collapse. In order to prevent this risk, a strengthening by cable elements (tension-ties) can be used. The probabilistic analysis of such RC structures under critical seismic loading is herein numerically investigated. The unilateral behavior of the cable-elements, which can undertake tension stresses only, is strictly taken into account. Attention is given to uncertainties concerning structural input parameters, common for such old RC structures belonging to built Cultural Heritage. For their treatment, Monte Carlo techniques are applied. The proposed methodology is explained in a numerical example.

Key words: Historic industrial RC structures, removal of columns, strengthening by ties, Monte Carlo methods.

1. Introduction

As well-known [1,2], many of old existing industrial reinforced concrete (RC) structures can be considered as elements of the recent built Cultural Heritage. Such historic

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(RC) structures are subjected to various environmental actions, e.g. corrosion, earthquakes etc., which can often cause significant damages. A main such defect is the strength degradation, resulting into a reduction of the loads bearing capacity of some structural elements. For some of such degraded elements is sometimes obligatory to be removed, and so a further reduction of the whole structure capacity is caused, which can lead to progressive collapse [2,16].

After structural assessment, a strengthening of the remaining structure is usually suggested, in order to overcome the above defects. Among the available strengthening methods [3-5], cable-like members (tension-only tie-elements) can be used as a first strengthening and repairing procedure [1, 6]. Cables can undertake tension but buckle and become slack and structurally ineffective when subjected to a sufficiently large compressive force. Thus the governing conditions take an equality as well as an inequality form and the problem becomes a high non-linear one. So, the problem of structures containing as above cable-like members belongs to the so-called Inequality Problems of Mechanics, as their governing conditions are of both, equality and inequality type [6-10]. A realistic numerical treatment of such problems can be obtained by mathematical programming methods (optimization algorithms).

Moreover, for the numerical analysis of such old RC structures, many uncertainties for input parameters must be taking into account. These mainly concern the holding properties of the old materials that had been used for the building of such structures, e.g. the remaining strength of the concrete and steel, as well as the cracking effects etc. Therefore, an appropriate estimation of the uncertain input parameters must be performed. For the quantification of such uncertainties, probabilistic methods have been proposed, e.g. Monte Carlo procedures, see [17-22].

This paper deals with a numerical approach for the stochastic analysis of existing old industrial framed RC buildings, which are strengthened by cable elements in order to avoid progressive collapse [16] after the obligatory removal of some degraded structural elements. The computational approach is based on an incremental problem formulation. Finally, an application is presented for a simple typical example of an industrial RC frame strengthened by bracing ties after the removal of some ground floor columns.

2. Method of analysis

The stochastic approach for the seismic analysis of RC frame-buildings strengthened by ties can be obtained by using Monte Carlo simulations as reported in [17]. For completeness of the present paper, the above approach is herein briefly summarized.

Monte Carlo simulation is simply a repeated process of generating deterministic solutions to a given problem, see e.g. [20-22]. Each solution corresponds to a set of deterministic input values of the underlying random input variables. A statistical analysis of the so obtained simulated solutions is then performed. Thus the computational methodology consists of solving first the deterministic problem for each set of the random input variables and finally realizing a statistical analysis.

Concerning the numerical treatment of the deterministic problem, a reinforced concrete (RC) framed structure containing cable-like members is considered. For the general analysis of such a structure, see details as described in [1,6,17]. Generally, a double discretization is applied: in space by finite elements and in time by a direct time-integration method. The RC structure is discretized to frame elements with generally non-linear behavior. For the cables, pin-jointed bar elements with unilateral behavior are used. The rigorous mathematical investigation of the problem can be obtained by using the variational or hemivariational inequality concept, see Panagiotopoulos [9]. So, the

behavior of the cables and the generally non-linear behavior of RC elements, including loosening, elastoplastic or/and elastoplastic-softening-fracturing and unloading - reloading effects, can be expressed mathematically by the subdifferential relation:

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

Here, for the example case of a typical i -th cable element, s_i and d_i are the (tensile) force and the deformation (elongation), respectively, $\hat{\partial}$ is the generalized gradient and S_i is the superpotential function [9,10].

For the numerical treatment of practical inequality problems, a piece-wise linearization is usually applied to relation (1), see e.g. [5-8]. So, for the case of cables, the unilateral behavior of the i -th cable-element ($i = 1, \dots, N$) is expressed by the following relations [7]:

$$e_i = F_{0i} \cdot s_i + e_{i0} - v_i, \quad (2a)$$

$$s_i \geq 0, \quad v_i \geq 0, \quad s_i v_i = 0. \quad (2b)$$

Here e_i , F_{0i} , s_i , e_{i0} and v_i denote the strain (elongation), "natural" flexibility constant, stress (tension), initial strain and slackness, respectively. Relations (2b) consist the Linear Complementarity Conditions (LCC) and express that either a non-negative stress (tension) or a non-negative slackness exists on cables at every time-moment. The above considerations lead to formulate the problem and to solve it at every time-moment as a Linear Complementarity Problem (LCP). The numerical treatment of this LCP is obtained by using optimization methods [1, 6-10].

In an alternative approach, the incremental dynamic equilibrium for the assembled structural system with cables is expressed in matrix form by the equation:

$$\underline{M} \cdot \Delta \underline{u}(t) + \underline{C} \cdot \Delta \underline{u}(t) + \underline{K}_T \cdot \Delta \underline{u}(t) = \Delta \underline{p}(t) + \underline{T} \cdot \Delta \underline{s}(t) \quad (3)$$

Here $\underline{u}(t)$ and $\underline{p}(t)$ are the time dependent displacement and the given load vectors, respectively. $\underline{C}$ and $\underline{K}_T(\underline{u})$, are the damping and the time dependent tangent stiffness matrix, respectively. Dots over symbols denote derivatives with respect to time. $\underline{T}$ is a transformation matrix. By $\underline{s}(t)$ is denoted the time dependent cable stress vector with elements satisfying the relations (1)-(2).

The above matrix equation combined with the initial conditions consist the problem formulation, where, for given $\underline{p}(t)$, the vectors $\underline{u}(t)$ and $\underline{s}(t)$ are to be computed. For the numerical treatment of the above problem, the structural analysis software Ruaumoko [11] is herewith used, as described in [5]. When the static case of the problem is only to be investigated, a Dynamic Relaxation approach [12] is appropriately used.

3. Numerical example

The historic industrial RC plane frame structure of Fig. 1 had been initially constructed with two more internal columns in the ground floor, which are shown as dashed lines and have been removed due to degradation caused by environmental actions. Following [16], the axial loads, which were initially undertaken by these two columns, are now shown as the two applied vertical concentrated loads of 180 kN and 220 kN.

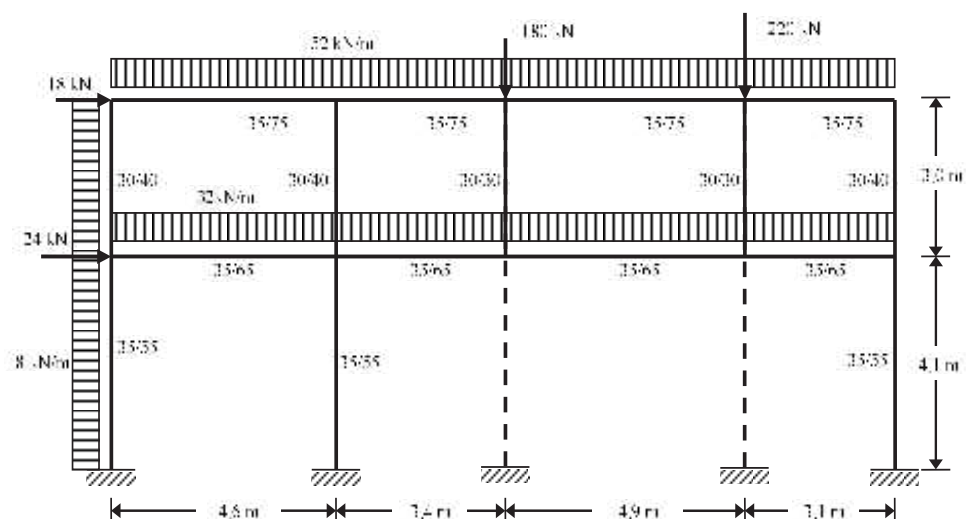


Fig. 1. The initial RC frame F0.

Due to removal of the above two columns, and after structural assessment [15] and in order to prevent a progressive collapse, the initial RC frame F0 of Fig. 1 is strengthened by ten (10) steel cables (tension-only tie-elements) as shown in Fig. 2. In the so formulated system, it is wanted to be computed which of the cables are activated and which are not, under the considered critical static loading of Fig. 1. This critical loading takes into account extreme seismic load effects according to current Greek Codes, see KANEPE [18].

Using Ruaumoko software [11], the columns and the beams of the frame are modeled by prismatic frame RC elements. The concrete class is estimated to be C12/15. The effects of cracking on columns and beams are estimated by applying the guidelines of [1-4, 13]. So, the stiffness reduction due to cracking results to effective stiffness with mean values of $0.60 I_g$ for the external columns, $0.80 I_g$ for the internal columns and $0.40 I_g$ for the beams, where I_g is the gross inertia moment of their cross-section.

Nonlinearity at the two ends of the RC frame structural elements is idealized by using one-component plastic hinge models, following the Takeda hysteresis rule [11]. Concerning the constitutive diagram of plastic hinges, a typical normalized moment-normalized rotation backbone [14] is shown in Fig. 3.

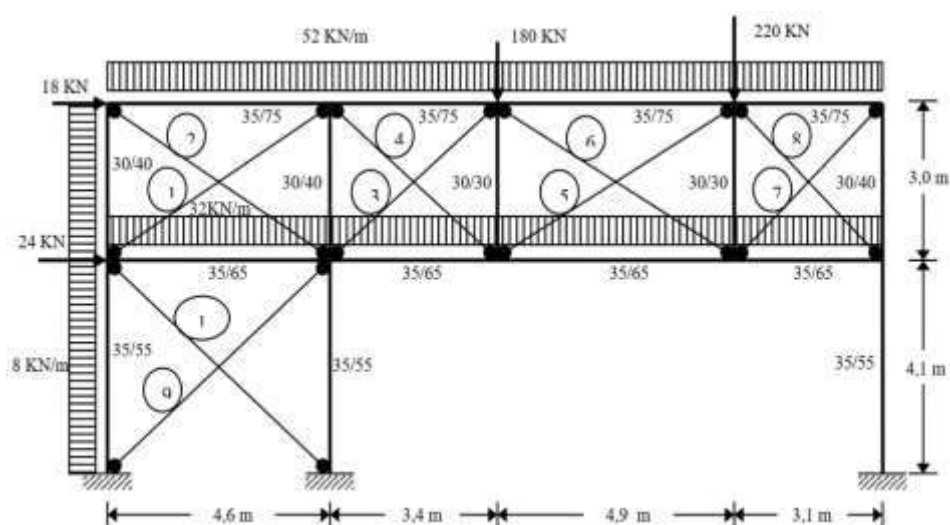


Fig. 2. The RC frame F1 (or F2) strengthened by 10 cables.

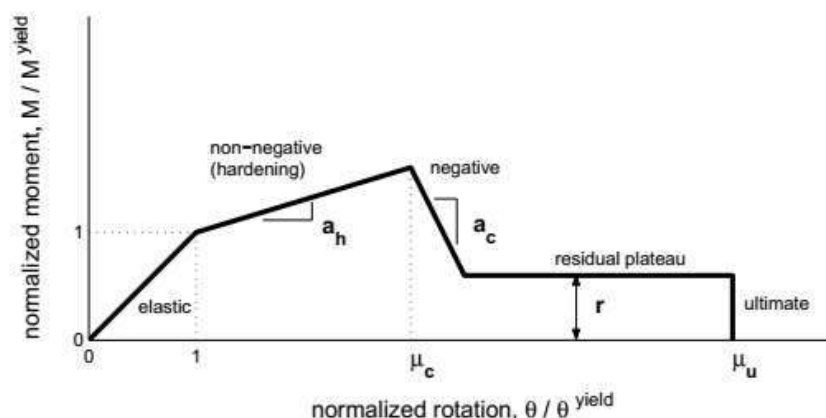


Fig. 3. Constitutive backbone diagramme for normalized moment - normalized rotation in plastic hinges [14].

The strengthening cable members have a cross-sectional area $F_r = 10 \text{ cm}^2$ and are of steel class S1400/1600 with elasticity modulus $E_s = 210 \text{ GPa}$. The cable constitutive law concerning the unilateral (slackness), hysteretic, fracturing, unloading-reloading etc. hysteretic behavior, has the diagram depicted in Fig. 4.

Considering input-parameters uncertainty, according to JCSS (Joint Committee Structural Safety), see [18-20], concrete strength and elasticity modulus follow a Normal distribution and the steel strength follows the Lognormal distribution. So the statistical characteristics of the input random variables concerning the building materials are estimated to be as shown in Table 1. By COV is denoted the coefficient of variation. The mean/median values of the random variables, computed by using upper and lower bounds-estimates, correspond to the best estimates employed in the deterministic model according to Greek code KANEPE [18].

Table 1. Statistical data for the building materials treated as random variables

	Disribution	mean	COV
Compressive strength of concrete	Normal	8.0 MPa	15%
Yield strength of steel	Lognormal	191.3 MPa	10%
Initial elasticity modulus, concrete	Normal	26.0 GPA	8%
Initial elasticity modulus, steel	Normal	200 GPA	4%

The application of the proposed numerical procedure by using a pseudo Dynamic Relaxation approach [12] and 250 Monte Carlo samples, gives the following mean-value results for the cable-elements:

cable-elements (sectional area, elasticity modulus etc.), the required upgrading of the remaining structure in order to avoid collapse can be obtained.

Table 2. Comparison of response representative mean-values of the initial frame F0 and the strengthened frames F1 and F2.

Frame	Cables cross-section	S ₄ [kN]	S ₇ [kN]	U _{y180} [cm]	U _{y220} [cm]	M ₁ [kNm]	M ₂ [kNm]
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
F0	$F_r = 0 \text{ cm}^2$	0.00	0.00	-2.46	-2.59	307.91	402.64
	COV [%]	0	0	22.1	22.1	21.8	21.8
F1	$F_r = 5 \text{ cm}^2$	272.11	268.65	-1.67	-1.70	218.22	290.98
	COV [%]	24.7	24.7	22.8	22.8	23.7	23.7
F2	$F_r = 10 \text{ cm}^2$	401.30	412.89	-1.26	-1.27	174.59	236.37
	COV [%]	22.4	22.4	23.4	23.4	20.8	20.8

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REVITALIZATION OF THE HISTORIC ROAD STEEL VIADUCT

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Abstract:

The results of the technical condition inventory related to the steel road viaduct located along the Polish national road DK88 in Silesia and used under load for several dozen years are presented and discussed in detail. The high level of wear of the basic structural elements was indicated, qualifying the tested object to an emergency condition. The numerically verified actual load capacity of the existing structure revealed in many elements significant overloads conditioned by a high degree of corrosion damage identified in situ. They resulted in above-standard deflections of the girders and also displacements of the supports. The values of such deformations and such observed imperfections clearly exceeded those that are acceptable in this type of objects. The proposed revitalization plan, which has so far been partially implemented in practice, is discussed in detail. In particular, the focus is on the design of the strengthening of the outermost beams, the refurbishment and rectification of the bearings as well as the repair of abutments and expansion joints. The proposed method of reconstruction, relief and strengthening of the existing deck is presented as well. In addition, the method of renovation of an incomplete drainage system carried out on the site is shown in detail. Separate attention is paid to the anti-corrosion protection implemented in the renovated structure. It concerned both the elements in use so far, only subject to the renovation, and also the quite new elements, inserted into the superstructure to strengthen it.

Key words: *steel viaduct, corrosion, overload, renovation, emergency condition*

1. Construction and equipment of the viaduct

The viaduct under consideration (Fig. 1) was built in 1936 so it is now the 86 years old structure. It is located in Upper Silesia region in Poland, in the city of Zabrze, along the national DK 88 road connecting Gliwice with Bytom (Fig. 2). The upper road here is a single carriageway two-way street with two lanes for each direction. The lower street that is crossed by this viaduct is also a single-carriageway and two-way road but in this case with the provincial status only (DW 921), with one lane for each direction and with a footpath on its one side. A single-track tram line is located parallel to this street [1].

The longitudinal view of the viaduct is shown in detail in Fig. 3 while its cross-section is shown in Fig. 4. The usable width of such the viaduct is 14.67 m while the overall width is 16.23 m, respectively (Fig. 4). It is made up of two mutually parallel bridges separated from each other by longitudinal dilatation. The slope measured across the axis of the whole bridge ranges from 4.5% to 5.0% (Fig. 4).

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Fig. 1 The considered viaduct seen from the side of Tarnopolska Street.



Fig. 2 Location of the viaduct shown on the map of Zabrze.

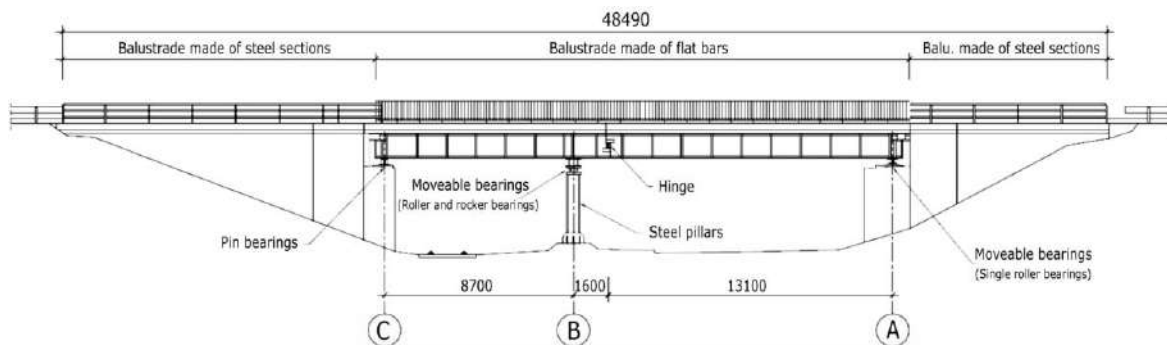


Fig. 3 Longitudinal view of the viaduct analysed by the Authors.

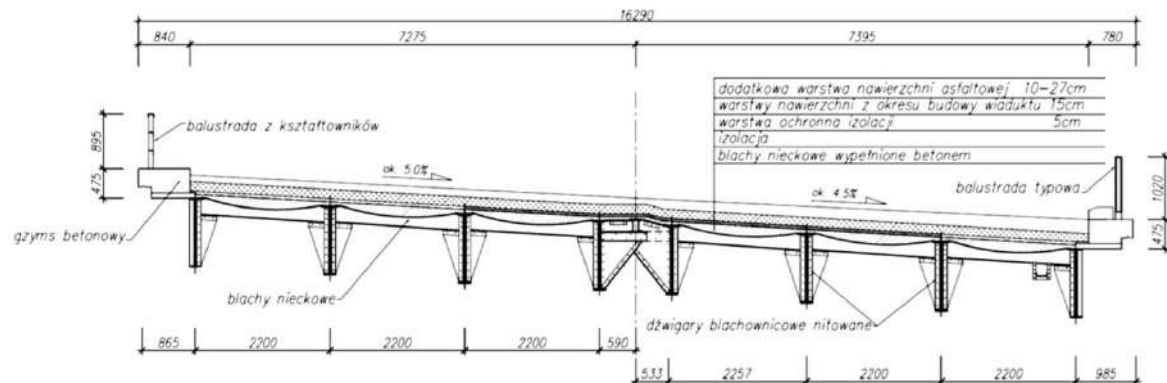


Fig. 4 The cross-section of the bridge decks.

As shown in Figs. 3 and 4, the load-bearing superstructure of the viaduct consists of two two-span steel beam grillages with span lengths of 14.70 m and 8.70 m, respectively.

The main girders are designed here as the continuous beams with hinges located in the longer spans, near the central supports (Figs. 3 and 5). The decks of both bridges are made of 8 mm thick concave steel sheets filled with concrete. Moreover, each bridge is built of four main girders with a spacing of 2.20 m (Fig. 4). The height of such girders varies in their length, which is achieved thanks to the use of thickening overlays on their lower flanges, located in the middle parts of the longer spans. The axial spacing of the girders which are adjacent to the longitudinal dilatation separating two parallel bridges is equal to 1.123 m (Fig. 4).



Fig. 5 Riveted construction of a hinge within the main girder.

The massive, non-reinforced concrete abutments with wing walls are used in the viaduct as the end supports (Fig. 3). The intermediate support is in this construction a single row of 12 separated steel columns with a closed box cross-sections (Fig. 6). Among these supports 8 columns, those located directly under the main girders, have a cross-section of 300×600 mm, while 4 additional supplementary columns, those located between the ones previously mentioned, have the cross-sections of 380×400 mm. These smaller columns were probably added to the original structure significantly later to strengthen the weaker pillar erected earlier. All the columns used in this support were finally effectively fixed in a single massive concrete foundation slab.

The main girders of the bridge deck are supported by steel bearings. In particular, the support at the abutment of the longer span is made by means of a single-shaft $\phi 170$ mm movable steel bearing, while the steel fixed bearing with a hemispherical head is placed on the abutment of the shorter span. On the intermediate support the main girders are supported by a transverse beam composed of two HEB 360 I-sections positioned parallel to each other (in accordance with the span skewness equal to 58.30°). This beam is finally supported by the steel single roller bearings (with rollers with a diameter of 200 mm) placed on the column heads located directly under the bridge deck girders and also on the steel spherical rocker bearings located on the heads of the intermediate columns.

The riveted steel plate girders with a web size of 1100×10 mm constitute the bridge deck supporting structure. The webs of these beams are stiffened with transverse ribs (Fig. 6). The hinge is located in the longer span of such beams, at a distance of 1600 mm from the axis of the bearings placed on the intermediate supports, and consists of a steel roller with a diameter of $\phi 140$ mm and also of a suitably shaped plate web, reinforced with sections of 150×100×10 angles horizontally arranged (Fig. 5). Both the lower and the upper flanges of the plate girders are made with the use of 90×90×10 mm isosceles angles, placed on both sides of the web and connected to this web by riveting. The lower flanges of the girders are strengthened, only in the middle part of the longer span, in the case of two external beams with a single while in the case of the remaining girders with a doubled overlays of the size of 200×12 mm. The upper flanges of such beams are also strengthened, in this case along their entire lengths, with a steel overlays of the size of 300×12 mm. The total height of the

girders is then accordingly: 1112 mm - on the sections without any overlays on the bottom flange, 1124 mm - on the sections with a single overlay, 1136 mm - on the sections with a doubled overlay.



Fig. 6 Construction of the intermediate support of the viaduct (The cross beam crowning the row of steel pillars is noteworthy. The ribbing of the main bridge deck girder is also visible in the foreground).

Over the supports, the main girders are connected to each other with transverse truss bracings. These bracings are shaped as K trusses made of 70×70×8 mm steel angles. The cross-bracings on the abutments are set parallel to the bearing axis (in line with the span skewness of 58.30°), while the ones above the intermediate pillars are set perpendicularly to the girders axis.

In the axes of the stiffening ribs, in the immediate vicinity of the upper flanges, the main girders are braced with secondary cross-beams made of I260 I-beams placed perpendicularly to the girders axes. Together with the plate girders, these cross-beams form the square fields with dimensions of 1350×2200 mm, in which concave steel sheets with a thickness of 8.0 mm are placed, connected by riveting to the upper flange of such a girder. Drainage holes are introduced in the central part of such the sheets.

There is a wide variety of pedestrian protection elements on the site, ranging from the oldest type of balustrades made of rolled sections with a height of 70-90 cm, through the bridge railings made of 90 cm high flat bars, ending with the modern bridge railings of the BS-3/M type, used in Poland, with a height of 110 cm.

The viaduct is drained on one side. Two different types of inlets are located here. One is near the beam hinge while the other is near the abutment. The rainwater is drained away along the road kerbs, and then collected into an open drainage trough made of C240 channel section suspended to the deck platform cross-beams at the outer beam. On the inventoried viaduct, from the side of Tarnopolska Street, no kerbs were used, while from the side of the city center, concrete kerbs were used, however loosely laid flat, not connected to the cornice.

2. Inventory of a technical condition of the analysed viaduct

At the access sections to the viaduct not only deformations and cracks in the pavement were observed but also highly advanced corrosion degradation of steel members in many places. Such cracks were noted also within the deck roadway, especially visible in the dilatation line above the abutment, as well as along the longitudinal dilatation separating adjacent decks. The cornices within both abutments revealed numerous damages, including in particular: cracks, material losses and even missing the concrete cover resulting in local

corrosion exposure of the steel reinforcement. It was also found the numerous displacements of kerbs with the lack of their proper grouting. In addition, intensive vegetation of plants was observed here, both on the cornices on the abutments and also on those located on the spans of the viaduct. There was also a lack of a pipe draining water from the C240 channel-bar and complete blockage of such the collector in the area of the sidewalk above the bridge inlet (Fig.7a). In addition, the destruction of the anti-corrosion protection was visible in many places. It resulted in numerous centres of corrosion developing mainly around the drainage holes of the concave sheets (Fig. 7b). One of the more serious damage was, however, a large material loss noted both in the road pavement and also in the road base (Fig.7a), through which rainwater flooded the abutment back wall, the abutment wing walls, the bearing pad and also the roller bearing placed under the both outer girders (Fig.7c).



Fig. 7. Deficiencies in the viaduct drainage system, including: a) the lack of the protection of the drainage pipe, b) effects of a blocked drainage system, c) rainwater running down the abutment.

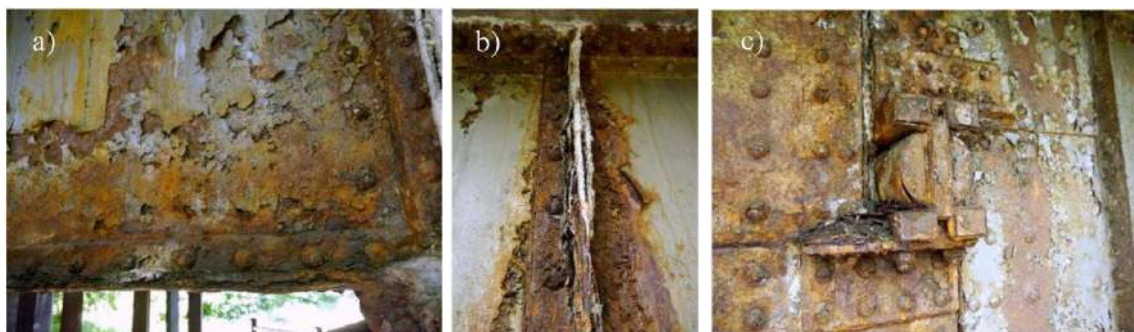


Fig. 8. Corrosion inventoried in the outer girder, related to the following places: a) the web and the lower flange of the beam, b) the stiffening rib, c) the hinge.

The most significant structural damages were diagnosed in the outer girder. In all the elements of its cross-section, such as the lower flange, the upper flange, the slender web (Fig. 8a), the stiffening ribs (Fig. 8b) and even the hinge (Fig. 8c), corrosion losses combined with locally observed delamination of the steel was inventoried. One of the most destructive effects of the material degradation of this type was blocking the real possibility of members rotation in the hinge. In addition to the corrosion, in the described girder damage to the paint coating was also observed, including its peeling and swelling, as well as a change in the colour of the protective coating. There were also visible numerous occurrences of sediments and efflorescence. This condition was probably caused by improperly made or possibly only damaged insulation of individual segments of the viaduct, or even its complete absence in the immediate vicinity of the girder. On the described bridge deck, the water from the entire road was drained away towards the outer girder, which made it most exposed to water. In

the case of the remaining girders, only slight sections were observed where surface corrosion was visible. Most often it was found in places where the concave plates were joined with the girders or, frequently, in the corners of individual trough plates. The bridge deck cross-beams also turned out to be heavily corroded, especially in the places of their connection with the main girders. In the vicinity of the abutment, a loss of a large fragment of the concrete slab was found (Fig. 9a). Moreover, some of the cantilever steel beams, made of C220 channel, supporting the reinforced concrete deck slab in its cantilever part, did not have any anti-corrosion protection due to the earlier falling off of the concrete cover. It seems to be particularly dangerous because this type of the falling off concrete pieces, from which such the cover was made, may pose a threat to users of the road and footpath under the viaduct. As shown in Fig. 9b, the lower part of the cantilevers presented here shows the steel corrosion developed on its entire surface, directly exposed to weather conditions.



Fig. 9. Damages to the deck slab identified in its cantilever part, including: a) missing a piece of the deck in the vicinity of the abutment, b) corrosion of steel beams exposed to weather conditions after the concrete cover has fallen off.



Fig. 10. Damages to the viaduct abutments (description in the text).

Both abutments of the considered viaduct were also seriously damaged. There were identified the losses of concrete on the front wall (Figs. 10b and 10c), the losses of the abutment back walls (Figs. 10a, 10c) and also the losses of concrete on the wing walls (Fig. 10c). Concrete corrosion was also found here to be significant. In addition, numerous horizontal, vertical and diagonal cracks were observed. Isolated cases of biological efflorescence and dampness (Fig. 10b), as well as plant vegetation have been reported on the wing walls cantilevers. Corrosion of the reinforcing steel was also observed, as evidenced by rust-coloured damp patches (Figs. 10a and 10b), as well as the direct exposure of the reinforcing steel at the abutment to weather conditions (Fig. 10a). Furthermore, damage of the bearing pads on both abutments under the outer girders was found (Figs. 10a and 10c) and the bearing shelf was contaminated with debris from the back wall.

The developed corrosion of the outer girder left no illusions as to the corrosion condition of the bearings placed precisely under this beam (Fig. 11). Moreover, in the abutment supporting the span located above the street unacceptable extreme displacements of all roller bearings was also found. Maximum displacement of the rollers was determined here to be 70 mm (Fig. 12), which significantly exceeded the expected value originated from the thermal displacement, which is set to be only 13 mm. Therefore, there was a concern about the possible blocked possibilities of bearing displacements, which finally could lead to a change in the static scheme of the whole structure. However, it should be noted that this type of blocking should not constitute a direct threat to the proper static behaviour of the structure, as the compensation of thermal movements may occur in the girder hinge. It should be noted that the construction of this hinge allows for the implementation of horizontal displacement in the range corresponding to the limit displacements of the girders caused by thermal influences.



Fig. 11. Bearing corrosion inventoried under the outer girder.



Fig. 12. The extreme deflections of the bearings observed at the abutment of the longer span.

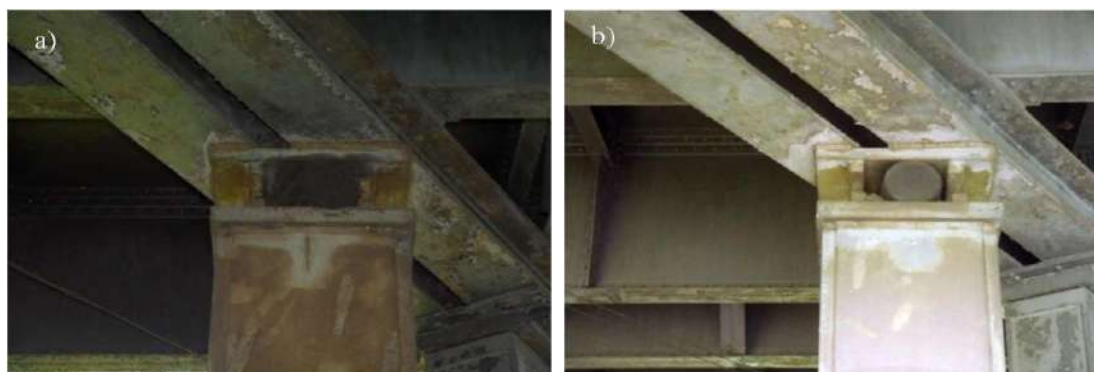


Fig. 13. The extreme deflections of the bearings observed on the intermediate pillar, inventoried as follows:
a) under the beam number 2, b) under the beam number 6.

In addition, the unacceptable position of the roller, and possible blockage of the displacement of such the roller, were found on two of the eight roller bearings located on the column heads of the intermediate pillar. The bearings in the emergency deflection were under the beam number 2 (Fig. 13a) and also under the beam number 6 (Fig. 13b) counting from the outer beam located on the side of Tarnopolska Street. There may be several reasons for excessive displacement of the bearings observed at the abutments. The most probable seem to be the following ones:

- improper setting of the structure on the bearings during the maintenance works,
- displacement of the abutment itself, whereby the rotation and/or the displacement of such the abutment in the direction of the embankment would have to be involved,
- the vehicle hitting the outer beam, which in consequence could cause a temporary lifting of the viaduct deck associated with the horizontal acting of longitudinal force.

During the performed inventory, the signs of damage to lower flange of the outer beam, seen from the abutment side of the longer span, were actually confirmed. However, their origin has not been clearly established.

In the Authors opinion, the direction of bearing displacements shown in Fig. 13 excluded the excessive load on the abutment originating from the embankment pressure as the cause of its occurrence. It seemed, however, that it would suggest possible ground movement in the foundation level. As a result of the performed measurements, no deviation of the abutment wall from the vertical direction was found. This means that if there is any movement of the abutment towards the embankment, it is only a linear displacement.

3. Assessment of the viaduct load capacity

On the basis of the chemical composition tests as well as of the strength tests performed for steel samples taken from the construction elements it was established that the cast steel of medium values of the tensile strength $R_m = 456$ MPa and of the yield strength $R_e = 319$ MPa was used for the viaduct girders. Following the above mentioned values, based on the recommendations of the standard [2], this steel was classified for conventional static analysis as a steel with the design value of random bending resistance set at the level of $R_d = 200$ MPa and with the associated design value of random shear resistance equal to $V_{R,d} = 120$ MPa. Detailed calculations were performed using the beam grillage computational model. Main girders of the grillage model was supported using pin and also moveable supports, adopted in accordance with the actual static scheme of the viaduct, taking into account the hinge within the main girders. The geometric characteristics of all the considered member cross-sections were determined on the basis of inventory measurements. As far as the main girders are concerned, the weakening of their cross-sections due to the existing rivet holes was taken into account. The self-weights of the structural elements, including the self-weights of the concave plates and the concrete filling them, were determined on the basis of the geometrical dimensions measured in situ. The thickness of the deck ballast was precisely determined by measuring the leveling of the ordinates of the roadway and by combining them with the measured ordinates of the girders. The thickness of these layers was found to be significant and their variability between the individual beams was high. In this case, the variation in thickness results from the different shape of the upper flanges of the supporting beams when considered in relation to the slope of the road surface which has been implemented in practice. The road slope is smaller, which causes the ballast thickness to increase towards the outer beam (number 8), which turned out to be the most corroded. For the beam number 1 this thickness was from 36 cm to 40 cm whereas for the outer beam (number 8) - from 49 cm to 53 cm, respectively. Such large thickness of the pavement layers was most likely due to the fact that new layers were added during subsequent road modernization works at various times. In order to remain on the side of safe estimation of the load capacity of the viaduct, it was assumed that the original pavement was made of

stone blocks on a sand bed and was not removed from the structure. Such pavement is always heavier than the asphalt pavement actually used on the viaduct in question. The cross-section of this pavement was adopted as follows, on the basis of [4]: 6 cm - concrete layer above the upper flange of the deck girders, 1 cm - insulation, 4 cm - protective concrete, 5 cm - sand bed and, finally, 10 cm - stone cubes.

The movable loads, as well as partial safety factors associated with these loads, were adopted in accordance with the recommendations of the standard [3] for the purpose of determining the standard load capacity of the viaduct both in the existing and also in the renovated condition, as well as on the basis of instructions [5] to determine its service capacity. The loads for classes A, B, C and D were taken into account, according to [3], and also the equivalent loads of categories 1/S42, 2/S32 and 3/S24, according to [5]. For movable loads, the dynamic coefficient, calculated according to [3], was used with the following values, respectively: $\phi = 1.284$ - for the main girders and $\phi = 1.325$ - for the deck cross-beams.

As a result of the calculations, it was found that the main girders do not meet the standard load capacity for any of the four considered load classes. In the case of class D, the stresses were exceeded only in the external beams, which means that for the beam number 8 these exceedances amounted to 18% was obtained while for the beam number 1 – amounted to 3% only, respectively. Exceeding the stresses in the edge beams resulted from their lower bending capacity compared to the middle beams. It is likely that, in accordance with the original design of the viaduct, these beams were located previously directly under the sidewalks, hence their weaker cross-sections. It should be noted that only one overlay was used in the span part of such outer girders instead of the two ones used in the remaining beams. At the time-point of the technical inspection, however, there were no sidewalks in the viaduct and the traffic took place very close to the edge of the viaduct deck. The greater effort of the beam number 8 in relation to the corresponding beam number 1 resulted from the fact that it was loaded more with pavement layers, the thickness of which was variable in the deck cross-section.

The shear capacity of deck girders was also verified. It was shown that such the capacity for all girders corresponded to the load of class A. Additionally, the bending resistance and also the shear resistance of the steel deck cross-beams were verified separately. It has been obtained that in the case of the bending resistance it corresponds to the load of class B, while in the case of the shear resistance - to the load of class A, according to the classification given in [3]. The standard load capacity of the viaduct was thus determined by the bending load capacity of the main girders and corresponded to the load qualified to class D. For this class load, the envelopes of bending moments in the deck girders were determined and compared with the successive envelopes typical for equivalent loads of categories 1/S42, 2/S32 and 3/S24, respectively, according to the recommendations given in [5]. Due to the fact that the bending moments lower than those associated with the load of class D has been obtained for the equivalent load of category 3/S24, the service capacity of the whole viaduct, related to the time when the evaluation was performed, was set at the level of 24 tonnes.

In the further part of inventory works, the identified deflections of the main girders were precisely measured. The obtained results are summarized in Fig. 14. In this figure they are compared with the theoretical values corresponding to these deflections, given in parentheses and calculated on the basis of conventional static analysis. These theoretical values were determined for the self-weight load estimated on the basis of the inventory measurements. As can be seen, in all the considered cases, the measured deflections turned out to be greater than those determined theoretically. Such a result may indicate a permanent deformation of the girders resulting from their overload.

The load capacity of girders corresponding to class D of the standard load allows, in accordance with the recommendations given in [3], to allow vehicles with a weight not exceeding 20 tonnes to run on the viaduct. Previously, heavier vehicles were allowed for traffic here, which could have contributed to the overload of each beam. It is also worth noting the high deflection value of the edge beams, especially those denoted by number 7 and by number 8. The greatest deformation occurred in the longer spans of such the beams, because they were most loaded with the weight of ballast unevenly distributed on the bridge deck.

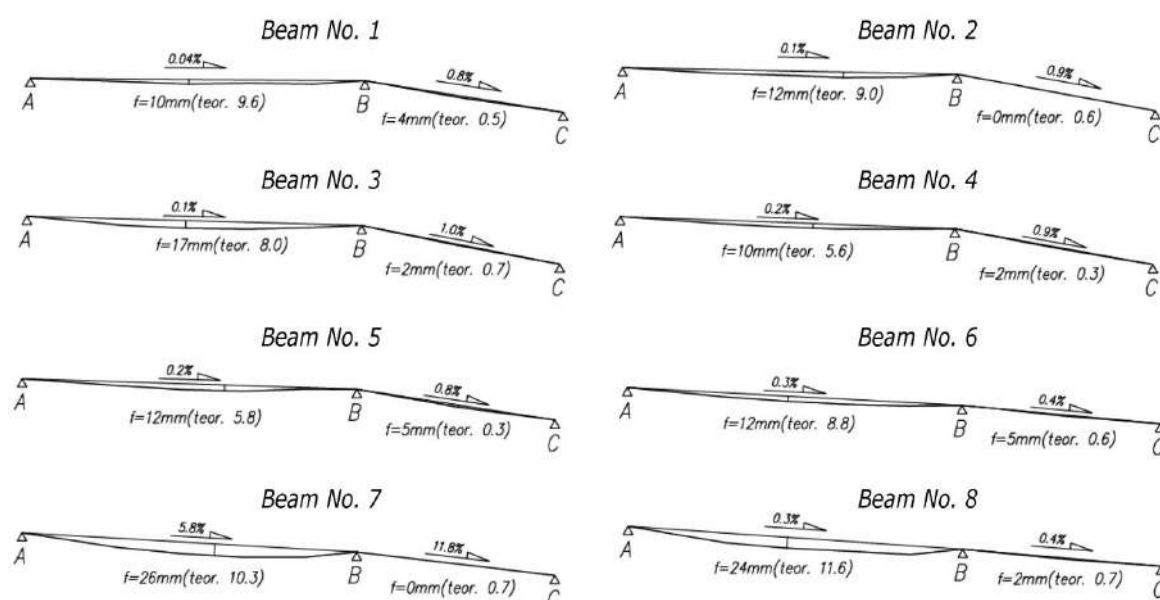


Fig. 14. Deflections of the tested beams, including respectively: values measured in situ (presented without parentheses) and values determined theoretically (presented in parentheses).

4. Renovation and revitalization of the viaduct

The structure described above was in a state of emergency and the main influence on this assessment was due to damage to the basic segments of its load-bearing structure, especially of the outer girder of the superstructure and also of the supports, located mainly from the side of the city centre. The immediate cause of such the condition was water penetrating into the area of the outer girder due to the complete failure of the drainage system, missing or partially damaged protective insulation and also numerous loses in the road pavement causing the washing out of the road base especially in the area of the abutment, as well as the corrosion of concrete in such the abutments. In addition, the inventoried damage of the back walls, the bearing shelf in the area of the external girders, as well as the corrosion degradation of the edge bearings and girders themselves resulted in the excessive stress levels in many locations, which translated into a direct risk of a structural failure.

The modernization works on this viaduct have been planned so that, after their completion, the structure has a load capacity of 37 tonnes. The scope of works to be performed included two basic parts. The first part concerned the works performed under the viaduct. The following works were included in it:

- repair of the concrete abutments,
- repair of the bearings and their rectification,
- repair and strengthening of edge steel girders,
- execution of a new anti-corrosion protection (this applies to both repaired and also newly used segments of the viaduct).

The second part of the planned works included those carried out on the viaduct, from the level of the DK88 road. They were as follows:

- reconstruction of the abutments, in particular of their back walls and of the upper parts of wing walls,
- lowering the grade line of the road, both on the viaduct itself and also on the access roads,
- reconstruction of the bridge deck slab,
- execution of a new waterproof insulation on the deck of the viaduct,
- repair of the deck drainage system,
- making new, properly working, expansion joints at both ends of the viaduct,
- execution of new kerbs along the edges of the viaduct deck,
- execution of rigid handrails on both edges of the deck,
- execution of the approach slabs behind the abutments,
- repair and strengthening of the embankment cones, especially in the area of the wing walls.

The major repairs required the abutments' bodies and also the bearing shelves, both on the A and also on the C supports, especially in the area where the girder number 8 rest on them, as well as the wing walls situated on the A support from the city centre side. These segments were repaired by adding the missing parts to concrete, after hammering off the loose concrete fragments. The rebuilt parts of the supports were reinforced, with the reinforcement anchored to these supports by fixing it on the glue in the drilled holes. Back walls of both abutments were rebuilt with their height adjusted to the new location related to the correction of the road grade line as well as the construction of the approach slabs. The existing back walls were cut at the level of the bearing footing. The upper part of the wing walls has been rebuilt with an appropriate shape that allows the installation of the sidewalk cover.

The segments of the beam number 8, such as the movable bearing on the support A, the pin bearing on the support C and also the steel hinge, were also repaired. In this case, the repair consisted in cleaning the individual bearing elements of corrosion products and then in their partial replacement. The rollers of both the movable bearing and the hinge have been processed on a lathe. After cleaning, the bearings and the hinges were protected against corrosion with a painting set, except for the rolling surfaces which were covered with graphite grease. Due to the exceeding of the limit displacement on the movable bearings on the support A their position was adjusted again. The bearings were repaired after the structure had been raised and temporarily supported with the use of hydraulic jacks (Fig. 15).



Fig. 15. The repair details of the support bearing, including: on the left - temporary lifting of the bridge deck by means of the hydraulic actuators, on the right - shifted bottom plate of the bearing with a new bearing pad.

The most serious corrosion damage occurred in the girder number 8, especially in the areas of its connection with the deck cross-beams and with concave plates. As part of the

renovation of the bearing structure the plates forming the bottom flange along the entire length of this beam were replaced, together with the angles that connected these sheets to the web. Furthermore, the new bottom flange of this beam was additionally reinforced by using a strengthening overlay in the span A-B (Figs. 16 and 17). The beam number 1 was also strengthened in the same span with a similar overlay (Fig. 18). These girders were repaired with structural steel grade S235. All connections of the existing steel elements with the new ones were made with the use of prestressing steel bolts qualified for class 10.9.

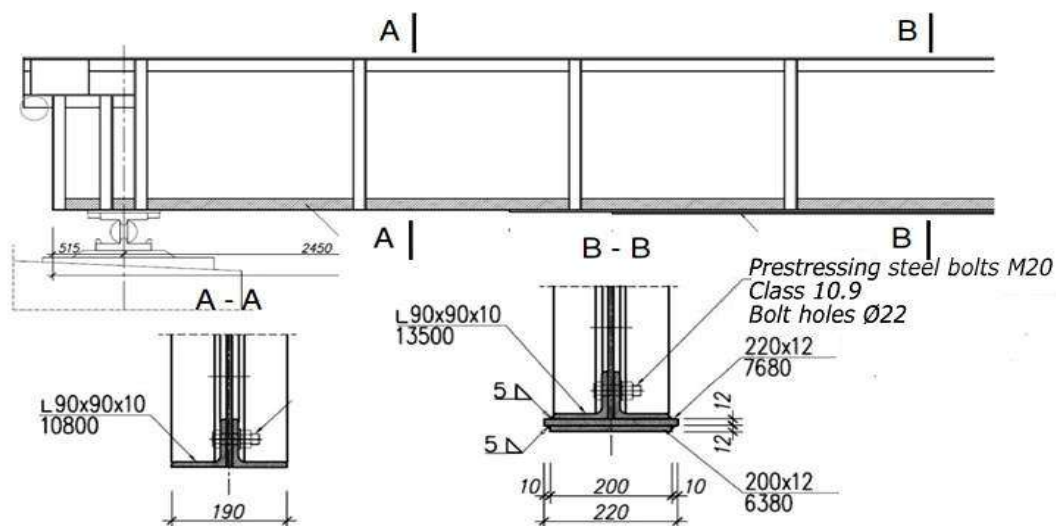


Fig. 16. The additional strengthening of the girder number 8.



Fig. 17. The girder number 8, including: on the left - condition after removing the angles and the sheets forming the bottom flange, on the right - condition after inserting the new angles and the new bottom flange.



Fig. 18. The strengthening of the bottom flange used in the girder number 1.

Anticorrosive protection of external concrete surfaces, those not in contact with the ground, was made with a 1 mm thick protective coating. A coating that is not resistant to chloride impact but has an ability to arrest the cracks developing in concrete was used for this. Gray paint coating provided additional protection for this type of surface. The anti-corrosion protection of those concrete surfaces that are in contact with the ground was made as a "cold" bitumen coating insulation. The newly dilated joints of the abutments' bodies and the back walls were secured with permanently plastic putty and with an expansion tape. The new steel members, as well as those previously corroded and existing in the current structure (in particular the beam number 8, the bearings, the columns constituting intermediate supports, and also the ends of the beams in the area of the back walls) were protected with a protective paint coating (these elements were previously cleaned to the grade Sa 2.5). An epoxy-polyurethane painting set was selected for use for this purpose, consisting of three layers. These were as follows:

- the priming layer, which was a two-component epoxy paint pigmented with zinc phosphate (100 micrometers thick),
- the intermediate layer consisting of a two-component epoxy paint containing iron gloss (130 micrometers thick),
- the top layer consisting of a two-component polyurethane topcoat (50 micrometers thick).

The total thickness of this set was thus equal to 280 micrometers.

Lowering the grade line of the road was to remove unnecessary ballast from the existing structure. It made it possible, together with the additional reinforcement of the deck girders, to increase the usable capacity of the renewed viaduct to 37 tonnes. On the viaduct, all layers of the old pavement were removed, together with the protective concrete and the existing insulation. The concrete filling the hollows of the concave sheets was also removed, while at the edges of the slab (including the longitudinal expansion joint), the entire layer of concrete was removed so that the steel structure was exposed, while only the upper layer of concrete, up to 10 cm thick, was scoured in the middle of the deck platform. The replacement of the pavement on the viaduct was associated with lowering the grade line of the road by about 20 cm and with the simultaneous need to correct its course on the access roads to the bridge. This type of grade line correction was performed on a section with a total length of 151.17 m.

The deck slab was rebuilt in the upper part and also in the outermost parts, i.e. where the new pedestrian walkways were formed. The renewed surface of the slab has a one-sided slope of 6%, in line with the position of the steel supporting structure. This plate was connected to the existing concrete by anchors made of steel rods embedded with epoxy glue in drilled holes. On the new concrete layer insulation made of weldable waterproofing membrane was laid, and on it a new asphalt concrete pavement.

The drainage of the existing structure was out of order due to its poor technical condition. The originally used number of drainage inlets turned out to be insufficient. For this reason, the old viaduct drainage devices were replaced and the number of inlets in the facility was increased to four. Two of these inlets have been placed in place of the previous ones so as to use the existing holes in the concave sheets. All inlets were connected to the drainage collecting pipe, situated in place of the old drainage gutter. In the renovated viaduct, the rainwater is drained away through the abutment to the existing drainage well located near the DK88 national road.

The modernized viaduct has been equipped with watertight expansion devices. At the ends of the structure, bitumen-mechanical expansion devices were installed, which should prevent the pavement cracking observed in the structure. The longitudinal expansion joint, at the junction of two parts of the structure, was covered at the level of the concrete slab with

a system solution in the form of tapes fixed on epoxy glue, supplemented by a cover made of stainless steel sheet.

On the edges of the deck platform, along the sidewalks, granite kerbs were placed, led outside the structure and connected with kerbs existing on the access roads. Steel safety-barriers were also installed, with the posts spacing every 1.0 m. The profile tape of the new road safety-barriers was attached to the profile tape of the safety-barriers existing at the access roads to the viaduct.

In order to improve the traffic flow at the exit from the bridge and to eliminate the possibility of transverse ruts in the pavement at this point the reinforced concrete, monolithic approach slabs, 30 cm thick and 4.0 m long, were made. These slabs were supported on the cantilevers shaped in the newly built abutment back walls.

The embankment cones were repaired in the upper part in the area of the wings. The ground losses were filled in and the slope surface was strengthened by sowing grass seeds.

5. Concluding remarks

The viaduct discussed in this article was in an emergency condition and required immediate repair works. Its technical condition posed a significant threat, in particular to the traffic of vehicles and pedestrians under the viaduct. It seems that to a large extent this state of affairs could have been avoided by adequately responding to the results of previous periodic inspections of the facility. It is difficult to imagine that during such inspections, for example, the lack of a part of the drainage collector or its complete blockage was not noticed. The progressive in time effects of insulation leaks, which increasingly degraded both the concrete and the steel of the supporting structure, and even the falling pieces of concrete, as was in the case of the abutments and of the cornices, were also not noticed. The methods applied in practice, related to the previously occurred repairs and temporary strengthening of the facility, also turned out to be controversial. For example, leveling the unevenness of the pavement by adding successive layers of asphalt over time resulted in an increasingly greater overload of the outer girders. In addition, it destroyed the effective work of the expansion joint both in the longitudinal axis of the viaduct, between the deck platforms, and at both abutments, and this resulted in an increasing number of cracks in the pavement.

The primary reasons for generating an emergency state of the viaduct can also be found in the implemented in practice, however not fully thought out and inadequate, in our opinion, adaptation of the original design of the considered viaduct, initially designed as a typical road structure with footpaths for pedestrians on both sides of the road. The purpose of such adaptation was to widen the road so that only car traffic would run on the bridge. However, it was not noticed that the load-bearing structure, placed under liquidated footpaths, was not adapted to this type of change. This is indicated by the cross-sections of the edge girders, significantly reduced in relation to the corresponding cross-sections of the remaining beams.

The analysis of the technical condition of the viaduct under consideration as well as the assessment of its ability for future use under loading applied to it was carried out on the basis of the instructions for carrying out the road inspections of engineering structures [6], of the principles of Polish construction law [7] and also of the principles of using the scoring scale in the assessment of this type [8].

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CONTRIBUTION TO DISCOURSE TO THE PRINCIPLE OF MINIMUM INTERVENTIONS IN THE CONTEXT OF ECOLOGICAL TRANSITION OF HERITAGE

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Abstract:

During its historical development, the principle of minimal conservation was primarily related to preserving the authenticity of materials, construction and historical value of architectural heritage but it changed in modern conditions. If it is not the case of the world cultural heritage and exceptional national cultural monuments, the repertoire of ideas, techniques and materials for the restoration and reconstruction of architectural heritage become wider and more creative. In the field of modern protection of architectural heritage, there is a tendency towards freer and more extensive interventions on architectural monuments. It is, to some extent, a forced process in order to reach a compromise between the existing monumental heritage and the growing need for urban and rural development of society. Principle of minimum intervention gets more structured meaning in the context of ecological transition and wide spectrum of new materials that are available in conservation nowadays. Selected Serbian heritage sites are used for case study.

Key words: *built heritage, ecological transition, minimum interventions*

1. Introduction

In the field of modern protection of architectural heritage, there are tendencies toward freer and more extensive interventions on architectural monuments. It is, to some extent, a forced process in order to reach a compromise between the existing monumental heritage and the growing need for urban and rural development of society.

If it is not about the world cultural heritage and exceptional national cultural monuments, the repertoire of ideas, techniques and materials for the restoration and reconstruction of architectural heritage is ever wider and more creative. The rapid loss of original values has led to the need to update and reconsider the principle of minimal intervention on architectural heritage, in order to test the significance of that principle, which was once more frequently applied in heritage preservation. The application of the principle of minimal intervention can be traced back to its beginning and its evolution

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followed through various examples, international charters and legislation of individual states during the 20th and the 21st c. [1].

Minimal intervention is often characterized as the least bad option. One of the strong advocates of minimal intervention and caution was Prosper Mérimée, who was the Inspector General of cultural monuments in France since 1834. He said that "in such a case, you should not pretend to do good, not to do bad is already a lot". However, he entrusted the work on great cultural monuments in France to his friend, Viollet-le-Duc, who took opposing views [2]. This principle is especially important when observed in modern conditions, when the need for preservation and sustainability is increasingly emphasized. During its historical development, the principle of minimal conservation was primarily related to preserving the authenticity of materials, construction and historical value of architectural heritage [3]. In modern conditions of exponential development of all areas of society, the choice of minimum conservation significantly expands its possibilities of protection, which refers not only to the material elements of heritage, but also interaction of values between users and other stakeholders.

The concept of an approach to all interventions on the architectural heritage that should follow the basic principles of conservation is still in force. Within these principles, minimal intervention is usually put first, which also includes minimal loss of authenticity, reversibility of all works on cultural monuments, minimal loss of building tissue and sincerity of intervention [4]. Adherence to these principles should ensure protection of aesthetic and architectural values, values of style, structure and construction techniques of historic buildings. These principles are constantly evolving, and their definition is expanding, so there is no doubt that it is necessary to reconsider significantly minimal interventions within the framework of modern doctrine of protection of architectural heritage. The review incorporates searching for the reason for the occurrence and application of minimal intervention throughout history and the way the principle is defined in various international and national documents.

Following the development of the application of this principle, it can be noticed that very often there is a deviation from today's understanding, although in modern legislative documents it is still one of the most important obligations of architectural heritage protection.

It is clear that the principle of minimal intervention, during the first decades of the 21st century, was exposed to numerous challenges of accelerated urbanization, rapid social development, economic pressure, great technical and technological possibilities, all of which led to occasional alienations or other interpretations of classical heritage doctrine. However, having in mind the modern expansion of building types, ensembles and places that are included in the architectural heritage, the principle of minimal intervention has regained its importance and possibilities of future changes should be carefully observed [5].

2. Historical reminisces of principle of minimum interventions

In order to fully understand the possibilities of applying minimal intervention in heritage protection, it is good to gain insight into previous research, primarily on identifying the origin of the term minimal intervention, into decision-making when such intervention applies and critical analysis of examples. Acquiring such knowledge is necessary because analyses of the effects of minimal intervention are not numerous, although it is one of the important principles in modern legislative protection of architectural heritage. **Through numerous texts that give a historical overview of the development of conservation doctrine, it can be extracted that the application of minimal interventions exists in some examples of conservation, but the principle is never precisely defined.** Until the middle of the twentieth century this principle did not

have an official name. However, there are examples of its applications, from which we choose the following:

- In his now famous and unavoidable study on the history of conservation, Jukka Jokilehto recognizes the initial of minimal intervention principle. This approach was applied to the protection of Trajan's triumphal pillar, which was built in Rome as early as 1162. The reason why this procedure is treated as a minimal intervention is the decision that was made at that time to prohibit any change in appearance or removal of the pillar. [6]

- In a series of numerous discussions on cultural heritage published by the French historian of architecture and urbanism Françoise Choay, she points to the changes that occurred in the ancient heritage during the Renaissance period of the respective governments by will of ecclesial rulers, the pope in particular. In line with her conclusion, Jukka Jokilehto suspects that this is exactly what happened during the process carried out by Michelangelo when he was ordered to turn part of Diocletian's terms into Basilica of St. Mary of the Angels and Martyrs [7]. His intervention was performed in the spirit of minimal intervention in space and metrics, but, unfortunately, later authors made significant changes, enriching the church space with Baroque decorations. [8]

- During the 19th century, two different approaches were taken in restoration of the Colosseum in Rome. Here stands out the first, proposed and performed by Raphael Stern in the 1920s. He constructed a brick support with which he used to support a part of the wall, and also walled it up in arches that were in that part of the wall. He preserved all the stone material in its original place, as well as all previous cracks and deformations. In Stern's approach there is no pretence that the support is part of the original structure [6], and also, the position of all stone blocks and cracks has been carefully preserved, which explains why this remains one of the interesting attempts of minimal intervention.

- In the circle of theorists who already in the nineteenth century advocated the equality of all layers of the building, some individuals in their texts and speeches clearly emphasized that the values of all layers as a whole must be respected in the building for all historical periods to be equally valuable and to be preserved. One of them was John Stevenson who, holding his incendiary lecture at the Royal Institute of British Architects (R.I.B.A.) in 1878, concluded by quoting the words of the French bishop and politician Charles-Maurice de Talleyrand-Périgord that “when in doubt, you don't have to do anything” [9]. This attitude can be understood as a true expression of minimalism, which will later be more clearly defined on similar premises.

- At the very beginning of the 20th c., in 1903, Alois Riegl published the text “The modern cult of monuments: its character and origin”. In that text, he advocates something that could be considered a minimal intervention in conservation today [10]. During the 20th c., the protection of architectural heritage, i.e. the importance of valuing buildings and choice of the appropriate technical method, gradually but continuously became more independent.

- Damage and destruction during the Second World War meant the decision to reconstruct entire buildings, even entire parts of the city. This approach was prevalent in many countries with a rich cultural heritage, the destruction of which, unfortunately, was enormous. The application of this process, the reconstruction, opened a discussion on whether the building will be reconstructed and further, after work on it, considered an architectural monument. In that international debate, it was especially questionable whether it was possible, i.e. whether the principle of minimal intervention was applied in such complex interventions. [11]. Today's understanding of the evolution of the conservation doctrine implicitly accepts that throughout history there has been the application of minimal intervention, but only if it is understood in a very broad sense.

3. Principle of minimum interventions in the documents of heritage doctrine

In all works on research, evaluation, protection and presentation, experts start from accepted theoretical principles, international and national documents and studies of already existing solutions in practice. In this paper, only those international documents were analysed in which it was possible to find elements that directly or indirectly recommend the application of minimal intervention at the level of the decision on how to perform works on architectural heritage.

From the Athens Charter, adopted in 1931 [12], in which attention was focused on modern theories of care for architectural heritage, until the latest charters and other international documents, several interesting conclusions can be drawn. Perhaps the most important statement is that in all these documents, to date, there is no clear definition of minimum intervention. Their analysis reveal that it is not possible to find a clear definition of minimum intervention in any of them. There are very few documents in which this principle is named, in more it is implied. Even if the principle is named, it is seldom more precisely explained or defined as is the case with other principles. If it is mentioned in general, then it refers to a narrower field of protection, perhaps most often in the field of archaeology.

The natural context in which the idea of minimal intervention occurs refers to interventions for adaptive, re-use of built heritage and changes that are necessary for giving a new purpose to a building. In some texts, the idea of minimal intervention can be sensed in relation with the principle of authenticity, integrity and preservation of all historical layers. The same is the case when it is in relation with archaeological remains and their protection, or the discussion about the applications of traditional or modern materials and construction techniques.

The documents related to urbanism, design and general legislation on spatial interventions advocate minimal intervention, even if declaratively. Since the beginning of this study, with the already mentioned Athens Charter of 1931, it was noticeable that the documents actually have a number of doubts or ambiguities about some terms commonly used in practice; some of them can be explained by issues of translations into various languages, but still a number of terms exist that are interpreted with varying degrees of freedom in practical application for other reasons. The Athens Charter states that the protection of heritage requires care for the building, and that a full analysis of the nature of preservation must be performed before any work can be carried out. This procedure is applied for each building on individual basis and there is no clear limit on how far one should go in the scope of the intervention.

The key documents of international heritage doctrine related to this topic in the second half of the 20th century are the following:

- In the Venice Charter of 1964, which it is considered a basic doctrinal text, the possibility of applying minimal interventions is not explicitly considered, but in certain recommendations, it can be concluded that this principle could be the best choice in some cases of work in practice. [13]

- The declaration that was well accepted, especially in Europe, was the Dresden Declaration on the Restoration of Monuments destroyed by War, adopted in 1982. [14] This declaration recommended and prescribed that the restoration of the war-destroyed architectural heritage must be done with special care so that the original parts of the building could be clearly seen. The emphasis on a clear demarcation of existing original parts and renewed ones came as result of a long-standing debate on restorations carried out in the first decades after World War II, from which some were made without demarcation. Paragraph 3.5 states that any intervention should be proportionate to the security-objectives set, thus minimizing the intervention in order to guarantee security and durability with the least damage to the value of the heritage. However, such an attitude has in practice

marginalized the principle of minimal intervention, and the line of demarcation between the original and the new material is often very freely interpreted.

- The well-known and often-applied Burra Charter from 1999 indicates that the facility is adequately protected if the existing tissue (of material) or conditions represent evidence of cultural significance or if the conservation process is impossible due to insufficient data. In a sense, this is an indicator that minimal intervention should be applied, i.e. that interventions should be reduced to a minimum when not enough is known about the facility. The same charter states that any damage to the tissue or structure of the object during the research should be minimized. It is advised to work only as much as necessary, but as little as possible in order to preserve the place as it was. The problem is, however, that while there is an increase in the requirement for all repairs or interventions to be carried out to an absolute minimum, on the other hand, expectations in terms of performance, safety and longevity have increased, all with a recommendation for minimum cost. When it comes to the new purpose of the architectural heritage, it is pointed out that such conversion, which requires adaptation, implies no or minimal impact on cultural significance and value. In this charter, all this is distributed in detail in several articles, but without a clearly formulated definition of principles.

- The text of the 1996 San Antonio Declaration refers to the views of conservative experts from a number of American countries. Considering that the service for the protection of the architectural heritage was developed later than in Europe, and that the heritage itself differs somewhat from the European one, it is interesting to interpret the way in which certain views are expressed in the declaration. The degree to which documented and missing elements are replaced as part of restoration treatment varies in North and South America, depending on the cultural characteristics of each country. Through this declaration, it is best seen that the choice of individual principles depends on the conservator's commitment to international recommendations. Some national policies suggest that what is lost from a building may be part of a memory, not a material heritage. In other areas, policies encourage the replacement of fully documented elements with facsimiles to re-establish the full significance of the place. It is emphasized, however, that only the historical tissue is considered authentic, while the interpretations achieved by the restoration are not. The restored parts can only authentically represent the meaning of the location as it is understood at the time. This insistence on preserving only the historical tissue as authentic can be subsumed under a very broad understanding of the principle of minimal intervention.

- It is only in the text "Principles for the Preservation of Historic Timber" from 1999 that the term "minimal intervention" is explicitly mentioned for the first time in the sense that this procedure is an ideal approach for construction with wooden structures. This doctrinal text of ICOMOS was improved in 2017 ("Principle for the conservation of wooden built heritage"). Again, one of the first items in the text reads "Interventions should preferably: "be the minimum necessary to ensure the physical and structural stability and the long-term survival of the structure or site as well as its cultural significance ". A similar view exists in the Charter on Built National Heritage, also adopted by ICOMOS in 1999.

- The Krakow Charter from 2000 advocates the principle of minimal intervention when it comes to the conservation of archaeological finds. Unfortunately, there is no clear definition of the principle in this charter either, but only the methodological procedure, i.e. the technique of work.

- In the same year, 2000, the Charter from Riga on authenticity and historical reconstruction of cultural heritage was adopted. The conference participants who adopted the charter at the beginning of the text pointed out that they agree that the value of cultural heritage is a testimony, tangible or intangible, of past human activities, and that

interventions of any kind that inevitably affect the quality of heritage should be at their minimum.

- In 2003, the ICOMOS Charter "Principles for Analysis, Conservation and Structural Restoration of Architectural Heritage" was adopted. The Charter explains in great detail the methodological steps in these areas, emphasizing that they should be implemented in a way that minimizes interventions in order to guarantee the safety of the building with as little damage as possible to the value of the heritage. The Paris Declaration on Heritage as a driver of tourism development from 2011 does not directly mention minimal intervention, but it is generally recommended that a minimum approach to tourism should be achieved by exchanging issues in the introduction of issues.

- One of the last doctrinal texts of ICOMOS from 2017 is the Salalah Guidelines for the management of public archaeological sites and it deals with the extensive problem of managing public archaeological sites, while insisting on sustainability in management. Sustainability in general, as well as complex management of distinctive heritage, implicitly introduces minimal interventions that will preserve the high value of archaeological sites. [15]

4. The Round Tower with Corbels of the Fortress in Bac – an example of the application of the principle of minimal intervention

Bac Fortress is a complex structure, built in several time phases. The construction began in the 14th century, and then was extended during the 15th and 16th centuries. Since the time of its construction, it has mostly been gradually devastated in its various segments, and for a long period of time it has been out of use and uninhabited. Until the second half of the 20th century, interest in the Bac Fortress was at the level of individuals, travel writers, archaeologists and amateur enthusiasts about antiquities.

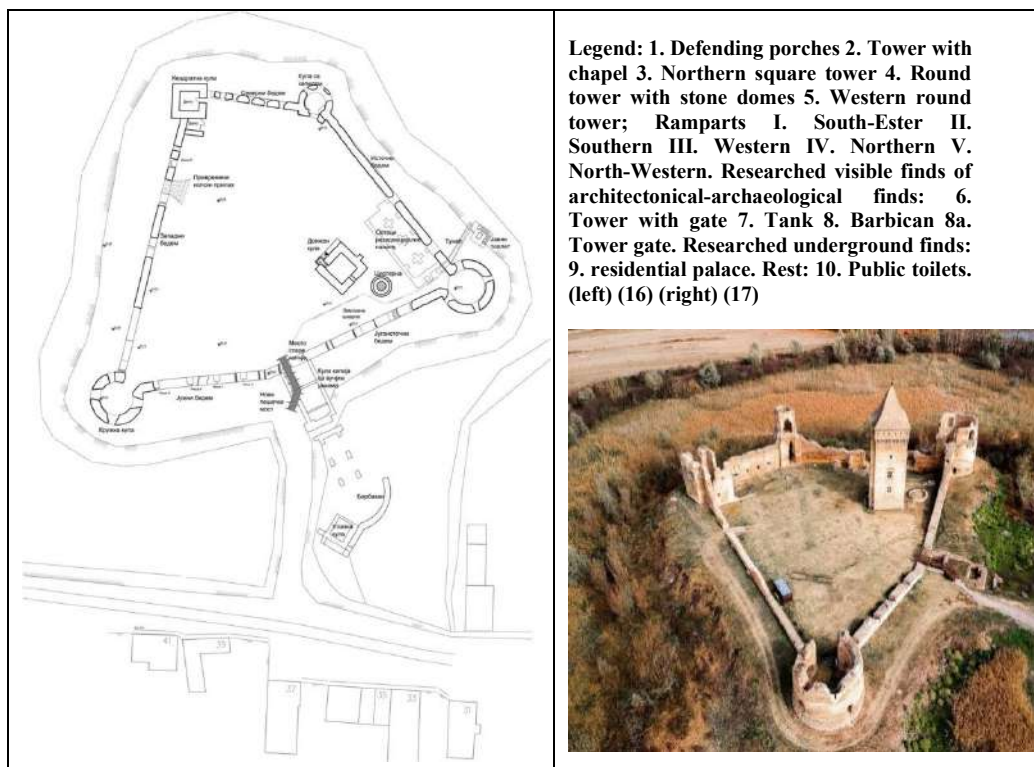


Fig. 1: The round tower with stone consoles of the Bač Fortress (photo by Nedeljko Marković)

More serious research and works on the preservation of the fortress date from the second half of the 20th century, and since 2006, through the Project "Centuries of Bac" it has been systematically researched, evaluated in segments and as a whole, has been object conservation, rehabilitation, presentation, new ways of sustainable purposes have been found, and education on the need to preserve the value of heritage have been disseminated.

Bac Fortress, which, with its suburbs, was declared a cultural asset at the national level, a spatial cultural-historical ensemble of exceptional importance [18] through the project "Centuries of Bac" [19] has been at the same time integral part of proposal for UNESCO WH List, called "Cultural landscape of Bac and its surroundings" (2019).



Fig. 2: The tower as practical example of minimum intervention before works (photo by Nedeljko Markovic)

Bac Fortress consists of several different types of buildings (Fig. 1), and they, after research and evaluation, were treated in different ways, i.e., by applying various conservation methods that leaders and experts of the multidisciplinary team considered most appropriate. Here only one building stands out as an example application of minimal intervention. The greatest attention of experts and the public was undoubtedly given to *the keep* (in French *donjon*, in Italian *dongione*) and the way it was researched, and then how the conservation and restoration works were carried out on it, as well how it was presented. However, for research on how and when the decision was made to intervene minimally on a building, it is more important to present the works on the eastern round tower with corbels as part of the defensive system of the ramparts. "The protruding position of the tower, its volume, and massive stone consoles are unavoidable contents of the silhouette of the Bac castle. This tower has preserved numerous elements of the original architecture, as well as data that allow reading the phases of construction. Data on construction techniques and materials, shapes, communications and the functioning of the Defense System have been preserved, including traces of fighting on its plastered facades and the harmful effects of Fire on the face of the circular wall. The large degree of damaged material found in the upper zone, in addition to the loss of historical structures, also posed a danger to the safety of visitors." [20] It is clear from this citation that the experts had a large amount of data on the basis of which a complete reconstruction of the entire monumental tower was possible, but the conservator's decision was to just repair interior parts of tower walls, as well as arches above the openings. Comprehensive research has shown its full significance and justification in the circumstances of insufficient archival material and a high degree of deterioration and degradation of materials useful in the construction of the Bac Fortress. "However, it has to be emphasised that the newly acquired knowledge did not affect the approach to the mediaeval structures conservation and presentation, which through history

had lost more than 60% of their original body, thus justifying the museum presentation of the cultural property." [21]

This is undoubtedly an example of the application of the principle of minimal intervention because the decision of the experts defines the way in which the integral of the original shape of the tower is preserved, which allows observer to imagine the appearance. This approach is completely in line with the initial decision of the experts to have the museum presentation of the Bač Fortress (Fig. 2, 3).



Fig. 3: The tower of Bač as practical example of minimum intervention after works (photo by Nedeljko Markovic)

5. Minimum intervention in the Context of Ecological Transition

The experts in all fields, built heritage including, recently became “aware” of ecological footprint. It was only the question of time when these problems would come into public focus, not if they existed.

The “discovery” of the first led to development of new economically-sustainable models and boundless efforts on improving heritage management. In the case of ecological transition, the principles and models seem to already exist but - they are needed to be seen in a new light. In both cases, interesting development of thought could have been found in early phase of movement among British researchers (for the ecological transition see [22]).

The humankind returns in this process to the genuine meaning of sustainability and its ecological essence as the highest priority as well as a challenge. Furthermore, ecological transition reflects reaction of experts to our overall reality: from radical urbanity of demographic trends to climate change. Focus moves from questioning if there are means for extensive intervention or presentation of certain kind towards inspecting if such presentation is really needed or the goal can be achieved with much less effort. Scope of means for achieving those goals is much wider due to new technologies. In the past, reasons for minimal interventions were prevalingly ethical, while nowadays they are highly practical and existential: without future for humankind there will not be a future of our heritage either. [26]

Principle of minimum interventions is one of the key principles for leading built heritage preservation in line with the trend of ecological transition. For example, it implies:

- minimal use of material, meaning minimal use of unrenovable resources and energy in a process of production which may follow
- reduction of need for transportation,
- minimal or optimal use of clean water (as highly limited resource)
- optimal if not minimal use of energy in the process of construction

- optimal use of human resources
- reduction of debris etc.

all of which contribute reducing greenhouse effect and minimize the ecological footprint.

Minimum intervention principle is not a magical solution. For example, when, at one point, the problem of the relation between architectural heritage and climate change was opened, the application of minimal intervention was considered insufficiently effective. That is, protection against climate change disclosed that more significant interventions are required on the fabric of the building in order to permanently protect it from temperature changes, pollution and the increasing frequency of natural disasters caused by numerous factors. It further proves that minimal intervention, as a principle of protection of architectural heritage, still remains the subject of theoretical consideration in terms of defining meaning of the term and its limitations.

The principle of minimum intervention may easily be generalized and further applied to any architectural activity, which, as a rule of good practice, gets confirmation in choice of laureates for 2021 Pritzker Prize [23]. Therefore, there should be no question why Bac, with its clear application of principle of minimum intervention got attention of jury in the name of EU and EUROPA NOSTRA and won grand prix in 2018 in the field of conservation.

6. Concluding remarks and conclusion

The principle of minimal intervention has been, undoubtedly from the beginning of its application, related to the idea of authenticity and the effort to preserve that authenticity as much as possible. This implies maximum preservation of historical tissue in order for the building to represent an original historical document.

Through the history of various treatments of cultural monuments, one can see examples in which the widely understood principle of minimal intervention has been applied. However, the idea that it is necessary to intervene as little as possible in order for the building to be a sincere testimony to a number of values, from historical to aesthetic, becomes much clearer and more explicit in the works of authors who began to develop their views in the mid-19th century. Their works represent the philosophical basis for the development of the principle of minimal intervention. It took more than a century for a number of factors, especially significant scientific achievements, to accelerate application of minimum intervention, yet even that was not enough to reach precise definition, such as was given for other principles. [27]

In some international and national documents that have been singled out and presented in this paper, it can be seen that even quite-extensive explanations of how and when to apply minimum interventions exist, but that there is no clear definition of the principle. Through the mentioned documents, this principle is most often binding when it comes to the restoration of the old or the introduction of a new function in an architectural monument (adaptive reuse). This restrains the desire for more significant changes, so minimal intervention is usually associated with another principle, namely reversibility. [24]

It is also noticeable that the number of recommendations for the application of the principle of minimal intervention increases with the degree of value, i.e., when the integrity, authenticity and preservation of all historical layers are extremely important factors for the categorization of cultural monuments. In these cases, minimal access to interventions is strongly recommended. Similar recommendations are given in cases when dealing with remains during work in urban planning or design in historic areas where there is a tendency to apply the laws of modern construction and when there is a tendency to incorporate new buildings into the historical fabric. The application of minimal interventions is very difficult to implement in cases of very "active" architectural heritage, i.e., when it comes to very populated and functionally dynamic ensembles.

When considering the application of minimal interventions in technical conservation, which concerns the direct work on the protection of original building materials and structural elements, there are great possibilities. Today's development of technology and technique enables the application of materials of exceptional potential with minimal destruction or complete preservation of the original parts of cultural monuments. This is very encouraging for preserving the authenticity and integrity of the heritage. [25]

Research and analysis of structures and all building materials has become much safer and less destructive with the use of new devices that can observe in depth without any consequences and detect damages which cannot be discovered other ways. These damages can also be stabilized with minimal intervention, i.e., by using materials that little, practically none at all, destroy the original elements of the heritage.

In recent years of the rapid development, new materials of exceptional performance emerged, applications of which can preserve the original material, authenticity and integrity of heritage. That enabled that even the application of minimal interventions provides big opportunities for preservation and presentation of built heritage. However, it should always be taken into account that a certain minimum of intervention is not a universal category, but depends on the case. [28]

Like other forms of architectural heritage protection, the concept of minimal intervention is relative and depends on several factors. The type of heritage is one of these factors, the second is the place where such an intervention should be performed and the third is the broader context within which the minimum intervention should be done. The choice of procedure undoubtedly differs depending on whether it is a building ruin or a historical landscape. Then, there is the difference in the type of ruin, i.e., whether it is a sacral or some other type of building. In this context, the choice of minimum intervention is the subject of serious consideration and weighing of all influential factors.

In addition to the elements that are directly related to the physical structure of the building, we should also take into account economic values, which have recently become increasingly relevant, often binding. Then there are the requirements of social, health, climate and other protections that need to be provided by the right decision when choosing a minimum intervention as an option. [26] The significance and scope of the practical application of the minimum intervention are very extensible and largely depend on the assessment of the conservation team working on a particular cultural monument.

Successful application of the principle of minimal intervention is achieved by taking into account all the values of architectural heritage, all other influential factors, the final decision of whether a cultural monument will be in use or just presented. Finally, it is very important to assess how the choice of minimal interventions in heritage conservation will affect social affiliation, quality of life, sustainability of the spatial unit within which it will take place and will continue.

From the facts summed in this paper, it can be concluded that minimal intervention is insufficiently defined as a technical method, especially in terms of scope and type of work, but that its importance is widely accepted in a cultural context.

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STYLE DIFFERENCES IN THE ARCHITECTURE OF BANOVINA BUILDINGS OF THE KINGDOM OF YUGOSLAVIA

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Abstract: *In 1929, the Law on the Name and Division of the Kingdom of Serbs, Croats and Slovenes into Administrative Areas was passed. The Kingdom of Serbs, Croats and Slovenes was renamed the Kingdom of Yugoslavia, with the idea of a political integral project, according to which the peoples were supposed to merge into a single, Yugoslav nation. In the second paragraph of this law, banovinas were established as the largest administrative units, with the aim of more efficient work of the administration and the creation of natural economic units. Banovine were named after the most famous rivers that flowed through them. Banovine existed from 1929 to 1941. At the beginning, the Drava, Sava, Vrbaska, Zeta, Vardar, Primorska and Danube banovinas were founded. The administrative buildings were very different in style. Older administrative buildings or residences were used for certain administrations, and some were new and built with great architectural ambitions. The paper analyzes the stylistic differences of these buildings. The research shows the evolutionary development of pre-war architecture in the Kingdom of Yugoslavia, which ranges from the adaptation of baroque-classical buildings to creations that represent the very top of modernist architecture.*

Key words: *Kingdom of Yugoslavia, Banovina buildings, Historical styles, Modern architecture, Differences.*

1. Introduction

The decree on the creation of nine "self-governing" units, whose territories were conditionally correlated with the basins of some of the most important rivers in the territory of the Kingdom of Yugoslavia, was passed on 3rd of October, 1929. These administrative units were called banovinas, and they were administered by the ban with power of authority. The capital Belgrade and its surroundings were a separate administrative unit. Banovinas were abolished in 1941 [23].

The reason for naming the Banovina after the names of big rivers came from the idea of national unitarism. During the formation of banovinas in accordance with the ideology of integral Yugoslavia, the earlier historical state-forming territories were deliberately broken up and their names avoided. The borders were drawn according to the geographical and economic principles, and the banovinas were named after the names of

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ivers, namely: Drava (with the center in Ljubljana), Savska (with center in Zagreb), Vrbaska (with center in Banja Luka), Drinska (with center in Sarajevo), Dunavska (with center in Novi Sad), Moravska (with center in Niš), Vardarska (with center in Skopje) and Zetska with the center in Cetinje). The ninth banovina (based in Split) was called Primorska (Coastal), considering that it covered a large part of the coast along the Adriatic Sea. In 1939, a new administrative unit, Banovina Hrvatska, was created by connecting the Savska and Primorska Banovinas (Fig. 1) [24].



Fig. 1. Banovinas of the Kingdom of Yugoslavia. A/ 1929 -1939. B/ 1939 – 1941.

The meaning of the word ban (banus) is very old and, even in the Middle Ages, it meant the highest civil servant, and the area under his administration is called banovina. There are several interpretations of the origin of the word ban, and the most common one is that it comes from the Mongolian-Turkish word bayan, meaning rich, powerful. Among the Avars, from whom it most likely entered the Croatian language, and then spread to other territories, medieval Bosnia and the border areas of Hungary, ban denoted a military commander. In some areas, bans ruled until 1921. In the Kingdom of Yugoslavia, since 1929, bans had the highest political and administrative power in the banovina, and were appointed, on the proposal of the president of the Council of Ministers, by a royal decree. It is interesting that not even the king could change the ban's decisions in the area of ban's jurisdiction, but only the courts.

2. Architecture of banovina buildings of the Kingdom of Yugoslavia

The creation of these administrative units meant that appropriate buildings should be provided in which the administration of numerous offices would be housed, as well as a residence for the head of the administration, ban. In each banovina, suitable buildings were sought in which the banovina administration could be located. A smaller number of ban offices could be accommodated in the existing buildings that had been previously built for the administrative and representative purposes of administrative bodies that had operated before the decision on the establishment of ban offices was made. As a rule, these were the buildings of districts, county, or other types of administrative units into which the areas that became part of the Kingdom of Yugoslavia were previously divided. Others were purpose-built and reflected the complex, almost conflicting situation in the architecture of the thirties of the 20th century, ranging from very late academicism and eclecticism to examples of Modernism in its most valuable form. The architecture of Banovina buildings is shown and analyzed according to the time of creation.

2.1 Royal administration building of Savska Banovina in Zagreb

The intricate structure of the complex known as Banski dvori (Ban's Palace) was created successively and is the oldest complex in which the Banovina administration of the Kingdom of Yugoslavia worked [4]. At the same time, the institution of bans in Croatia existed much earlier, back in the era when a part of today's Croatia was under the rule of the Austrian and then the Austro-Hungarian monarchy. The history of construction dates back to the 18th century, when the old building of the then Ban Kulmer was purchased on the west side of Marko's Square in Upper Town, Zagreb, with the intention of accommodating the parliament, the city county, the seat of the courts, archives of public records and parliamentary minutes, and Ban's apartment.

This building was not spacious enough to accommodate all the mentioned functions, and parliamentary sessions could not be held either, so the first solution was to make it a residence for the ban. As it became the most famous function of the building, it was originally called "ban palace" and then "banski dvori", a name that has remained in use to this day. The bans of Croatia, then part of the Austrian, later Austro-Hungarian monarchy, ruled there from 1809 to 1918, and from 1929 the Royal Banovina Administration of the Sava Banovina was located there until 1939, when it was disbanded by reform and became a part of the Croatian banovina. Already in the first half of the 19th century, functional expansion was necessary, so in 1839 the Zagreb County acquired neighboring houses on both corners of the square. The unification of the thus created urban block was entrusted to the architect Nikola Brdarić, who replaced the existing building structure with a large two-story building with internal courtyards. Due to various internal modifications, the first floor of the building housed the Ban's apartment, the second floor housed the courts, and the 83 rooms still house various administrative offices of the Government of the Republic of Croatia [1]. Although this complex later underwent more changes, the vestibule and the assembly hall mostly retained the recognizable original structure. The exterior architecture was designed in the spirit of baroque romanticism, but reduced to basic forms, without excessive decorative elements, and was embellished with later interventions, the last time after the rocket attack in the war of the nineties of the 20th century, during which the Federal Federal Republic of Yugoslavia disintegrated [16]. The Parliament moved to a building on the eastern side of Marko's Square, and today the seat of the Government of the Republic of Croatia is located in this complex. Banski dvori of the former Savska banovina in Zagreb are on the list of protected cultural property (Figure 2).





Fig. 2. Royal administration building of Savska Banovina in Zagreb

2.2 Royal administration building of Drinska Banovina in Sarajevo

The administration of the Royal Administration of Drinska Banovina was placed into an already existing building, which was used for the needs of the Austro-Hungarian administration, that is, the Land Government for Bosnia and Herzegovina. It was designed by the distinguished architect Josip Vancaš (1859 – 1932), who worked in Sarajevo for a long time [2]. This building, built very quickly, in the period from 1884 to 1886, of large dimensions, 70 m long and 65 m wide, originally had a ground floor and two floors. The number of rooms, 203, was needed, because it was functionally necessary, in order to accommodate the administration of numerous departments, the cadastral archive, the national bank, the supreme court, inspectorates and representative salons, as well as a number of auxiliary services (guards, couriers, janitors, etc.). In accordance with the spirit of the times, the architecture of historical styles was the only choice for such a significant and representative state building. Architect Vancaš opted for the external appearance of the building in the Neo-Renaissance style, and the arrangement of stylistic elements was done in accordance with the academic forms of the Florentine early Renaissance. The main reason for this choice was economic; such finish of facades was the cheapest, because there are no pillars and other ornate decorative elements. The main staircase, salons and offices of senior officials were decorated somewhat more ornately. The original form of the building was changed in 1911, when the third floor was added. The Bosnian Parliament was founded in 1909, so additional space was needed for this new function. The Royal Administration of Drinska Banovina was moved into such an expanded building where it worked until 1941. In the exterior architecture, the addition of the third floor is clearly indicated by the difference in color. Considering the chosen stylistic expression, the building in which the Royal Administration of Drinska Banovina was located fits into the academic type of buildings created during the 19th century, adaptable to different functions, from administrative to educational to museum. Similar forms were preserved in many cities of the former Austro-Hungarian Empire. Today, the Presidency of Bosnia and Herzegovina is located in that building (Figure 3) [5].



Fig. 3 Royal Administration building of Drinska Banovina in Sarajevo

2.3 Royal administration building of Dravska banovina in Ljubljana

The building that housed the Royal Administration of Dravska Banovina still exists. The first conceptual design was done in 1886 by the famous Viennese architect E. Ritter von Förster, but even in this case, the construction was accompanied by unpleasant events such as the choice of poor terrain for construction, and design changes. The building was redesigned and overseen till completion, until obtaining the occupancy permit on November 11, 1898, by the engineer Rudolf Bauer, who often worked together with von Förster. This building occupies a large area of the block at the joint of three streets, Prešern, Erjavc and Gregorčič, and was undoubtedly one of the largest buildings in Ljubljana at the time of construction. The three street facades have three floors, while the fourth, the courtyard facade, is a floor lower. Until 1918, the building was the seat of the Administration of Kranj, then of the People's Government, of the Prefect of the Ljubljana Region and, until World War II, the Royal Administration of Dravska Banovina. Given its original purpose, which was administrative, this building in everything followed the characteristic spatial organization and appearance of representative palaces in the architecture of historical styles. The exterior appearance was designed following the principles of the Florentine Renaissance, so historians of architecture have long ago

noticed the conceptual closeness to the Drinska banovina building, with the distinction that the sculptural decoration of the Dravska banovina palace is more ornate, featuring statues by the sculptor Josip Beyer that have a symbolic meaning. Two seated statues representing power and justice are located next to the main entrance from Prešern Street. The largest room in the palace, the Crystal Hall, today the reception hall, was originally a chapel [18]. Both buildings, Drava and Drina banovinas, were modeled after the administrative buildings in Vienna, the capital of the Austro-Hungarian monarchy. The palace is a cultural monument and is one of the representative examples of Slovenian architecture created at the end of the 19th century. Over the past 110 years, the Neo-Renaissance look of the building has remained largely intact both externally and internally. The only major change in its external shape is the oval wall that encloses the courtyard terrace [17]. Today, it houses the two most important state institutions of the Republic of Slovenia, the Office of the President of the Republic and the Office of the Prime Minister. The building of the former Dravska banovina is a cultural monument of national importance (Figure 4).

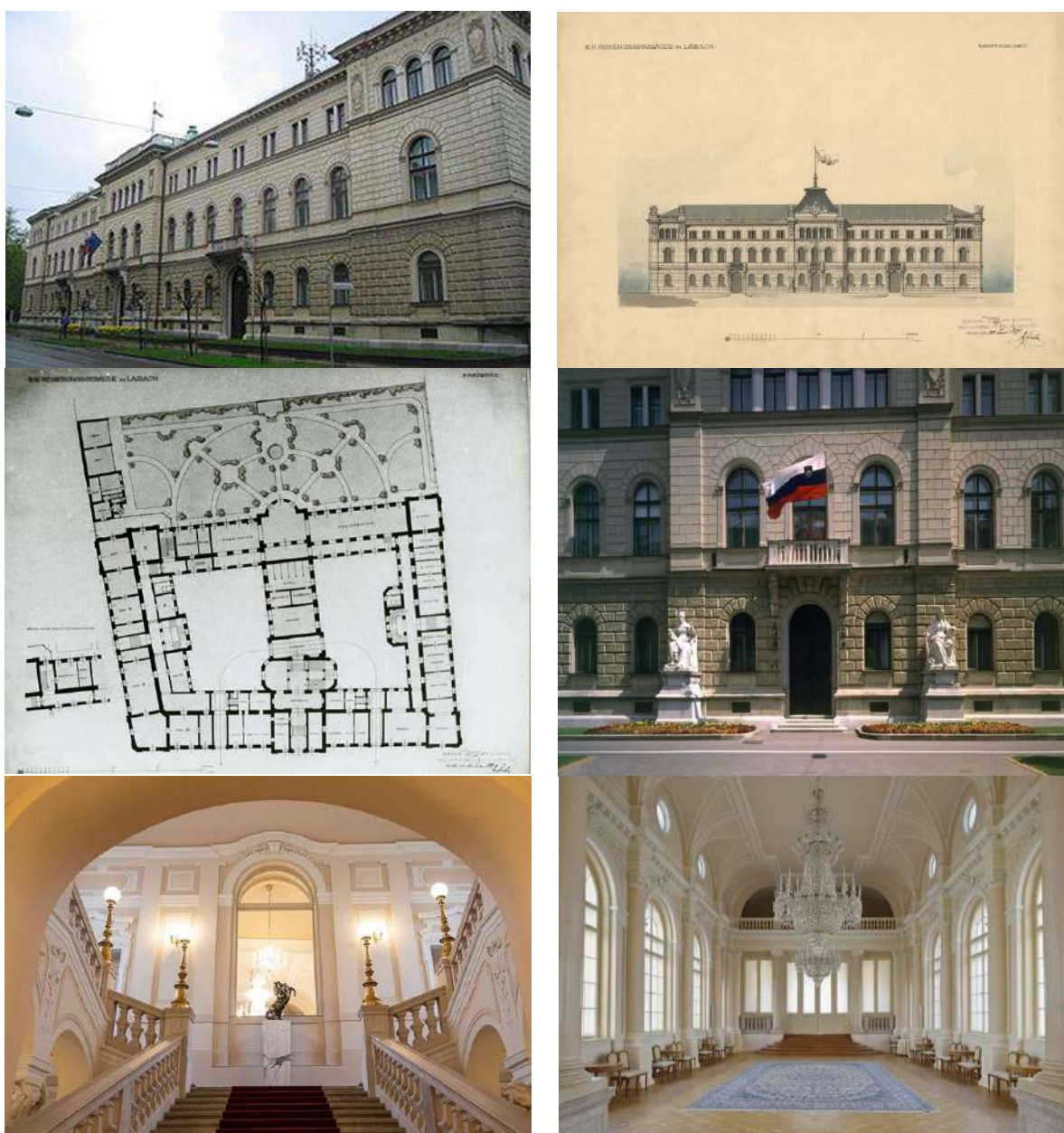


Fig. 4. Royal administration building of Dravska banovina in Ljubljana

2.4 Royal administration building of Moravska banovina in Niš

Another old building, which was not originally intended for the Banovina administration, was built in Niš in 1886. At that time, it housed the Administration of the Nis District of the Kingdom of Serbia, the court and the police administration. It was built in the center of Niš, on the quay along the bank of the river Nišava and near the spacious Turkish Fortress, built on multi-layered cultural foundations. The building's original form already represented the most monumental and representative public building in the territory of the Niš district with its position, volume, structural composition and stylistic expression. In the first phase of construction, two tracts were formed with a ground floor and a first floor, with an entrance avant-corps that divided the length of the facade along the river into two symmetrical parts. The second floor was added in 1930, and the building attained its present appearance when it was decided to house the administration of Moravska Banovina which was completed as late as in 1935. According to the desing of the engineer Petar Gačić, the wings that closed the urban block in the form of an elongated trapezoid, with a rounded corner avant-corps, which represents the front, were completed. There is a stately entrance with a staircase and a porch. There is an internal courtyard in the middle of the block. The functional needs were met by adding floors and expanding, and the exterior of the building became even more striking in contrast to the immediate environment dominated by low-rise family residential buildings. The building is academically composed in the architecture of historical styles, dominated by Neo-Renaissance, which is reflected in the application of horizontal gradation of elements on the bearing wall (from rustic treatment in the lower zones to finely smoothed surfaces in the upper zone, rows of pillars, tympanums, end wreaths, portals, balconies, semicircular entrance porch). The interiors, especially the large meeting hall, were richly decorated and luxuriously furnished [22]. Today, the building houses the Rectorate of the University of Niš and the Institute for the Protection of Cultural Monuments in Niš (Figure 5).



Fig. 5. Royal administration building of Moravska banovina in Niš

There were no suitable buildings in the other seats of banovina administrations, which could embody the importance of the newly created state institution and be spacious enough to accommodate the numerous functions that needed to operate through the banovina administration. Thus, for almost every banovina administration, immediately after the decision on their formation was made, a competition for the construction of the building was organized. This was completely in the spirit of the then practice of always announcing "tender" for representative public buildings, where young, not yet established architects often won. During 1930, first, on April 7, a tender was announced for the conceptual design of the Banska palata in Novi Sad for Dunavska Banovina, then, on April 12, a call for tenders was announced for Banska Palace and Banski Dvori in Banja Luka, for Vrbaska Banovina, and on May 26 for the administrative palace in Skopje, for the administration of Vardarska banovina. In the middle of 1930, a competition was announced for the seat of the Royal Administration of Primorska Banovina in Split. The course of construction of each of banovina administration buildings, which had to be built based on the winning tender designs, was different, but they have in common that not one of them was realized according to the original tender designs.

2.5 Royal administration building of Zetska banovina in Cetinje

Zetska banovina building in Cetinje, built between 1930 and 1932, was the fastest built one among the new palaces of the royal banovina administrations [8]. Its designer was an esteemed Russian émigré, architect Nikolay Krasnov (1864-1939), employed for many years in the Ministry of Construction of the Kingdom of Yugoslavia. Most of his buildings were designed so that their architecture, in one of the recognizable historical styles, seems representative, monumental, but also anachronistic for the period in which it was created. There were several reasons for this, but there is no doubt that his way of working suited the investors, who, as a rule, represented state institutions. At the time when he designed the building of Zetska banovina, he was the favorite architect of King Alexander I Karađorđević, who was also the creator of the banovina as an administrative system. In accordance with the rich experience he had, designing representative buildings in Imperial Russia, and then in the Kingdom of Serbs, Croats and Slovenes, i.e. later the Kingdom of Yugoslavia, Krasnov, looking more at the past than the future of architectural ideas, designed the administration building of Zetska banovina as a combination of the monumental volume and simple decorative elements, which are reminiscent of the architecture of some public, educational and administrative buildings that were built in the coastal cities of Italy and Dalmatia for the needs of the former Austro-Hungarian monarchy. Considering the variety and number of functions that had to be accommodated in the building, its dimensions undoubtedly had to be strikingly larger than all the buildings that had been erected in Cetinje up to that time, including numerous embassies that, as a rule, were erected in historical architectural styles [12]. The character of its purpose thus received an appropriate architectural envelope, harmonized to a large extent with the environment, in which there was a mixture of heterogeneous forms, from traditional residential houses, through neostylistically designed offices, to simultaneously created buildings devoid of any architectural ambition. The building, developed according to the depth of the plot, has a basement, the windows of which are completely visible above the ground level, an elevated ground floor and two floors, except in the central part where also the third floor was built. The frontal section of the palace of the royal administration of Zetska banovina has an accentuated central part and two shallow corner avant-corps, which are rustically finished with stone blocks vertically, as is the entire ground floor along the length of the building. The entrance part is highlighted by a three-arched porch, on which rests a narrow terrace with a fence made of simple balusters. All other parts of the building are devoid of the plastic decoration. The more pronounced eave hides the shallow roof

slope, which, together with the rustic treatment of parts of the building and the porticoes of the porch and several similarly finished windows, indicates the architect's desire to evoke something of the Renaissance expression of the entire facade in an eclectic way [7]. The building is a cultural monument of national importance and today the offices of the municipality of Cetinje are located in this building (Figure 6).

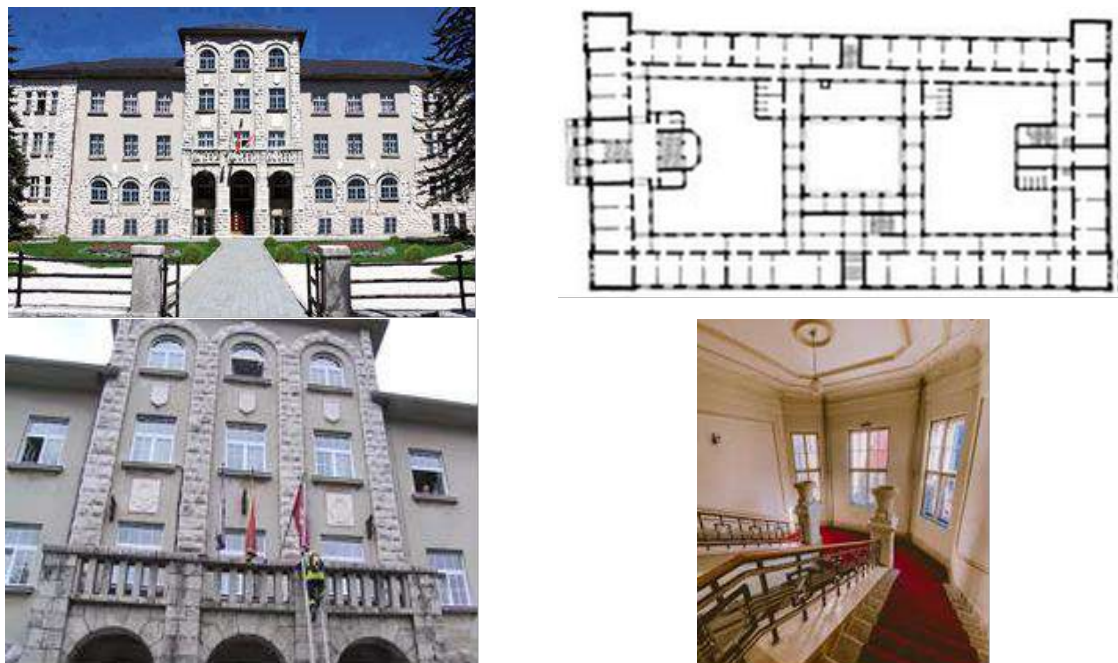


Fig. 6. Royal administration building of Zetska banovina in Cetinje

2.6 Royal administration building of Vrbaska banovina in Banja Luka

In the competition for the Ban Palace and the Banski dvori of Vrbaska Banovina in Banja Luka, held in 1930, the team of architects from Belgrade, Jovan Ranković (1902 – 1992), Jovanka Bončić-Katerinić (1887 – 1966) and Anđelija Pavlović, won. In a city that experienced a transformation of the urban design at the end of the 19th and the beginning of the 20th century, when the oriental urban matrix was gradually replaced by the European one, it was not realistic to expect that state administrative and representative buildings would introduce major architectural innovations in the sense of more contemporary, i.e., modern design. The offered competition design solution was not only architectural, but also town-planning. By defining the position of two buildings, one for the banovina administration, the other for the ban residence, a symmetrical solution of the square in the center of the city was implicitly encouraged [15]. Both buildings are very massive and appear monumental. In the external architecture, one cannot recognize a clearly defined stylistic expression, but one can see the authors' desire to connect the basic body of the building, formed in the architecture of historical styles, with decorative motifs and elements of Serbian medieval architecture [6]. The entrance to the ban's residence, above the portico, is additionally decorated with two free-standing female figures, personifications of Yugoslavia, sculpted by the famous sculptor Đorđe Jovanović. This is the only example, except the complex of the Royal Administration of Dunavska Banovina, that the administration building and the ban's residence are separated into two separate buildings. In the banovina administration, 150 rooms are planned for the entire administrative apparatus of the banovina [19]. In the lower zone of the Ban's residence, meeting rooms, reception halls, and a counseling hall were formed, and in the second zone, were located the ban's apartment and an apartment for distinguished guests. The

infrastructure was also located in this building, i.e., rooms for central heating, kitchens, sanitary facilities, wardrobes and other auxiliary rooms [11]. The dispositions of these buildings are solved with a block system, with skylights and an accentuated centrally placed staircase, well-conceived and creative, satisfying all the functions. Both buildings were built according to the street block system, they include the corners of the streets, where, coming out with facades on three sides, closed units are formed. The distance between these two palaces is dimensionally so large that a representative square is formed between them. In the center of the square is the monumental church of the Holy Trinity. Such an architectural solution, which contains a complex architectural expression of unified classicism and elements of medieval Serbian-Byzantine architecture, with extended and accentuated pilasters, and with decorative capitals and archivolts that frame semicircular openings, produced an architectural creation of special value [3]. The Banja Luka municipality is located in the building of the former banovina administration, and the cultural center is located in the ban's residence (Figure 7). Both buildings are cultural monuments of national importance. The way in which these two buildings with different functions, and almost the same volume and exterior architecture, are designed gives the impression that they are two almost identical city palaces, and not a single complex, as was successfully correlate in the case of the Dunavska banovina in Novi Sad. The reasons should be sought in the different needs that the two banovinas were supposed to meet and the way in which the first bans in power, at the beginning of the fourth decade of the 20th century, imagined that they should represent the royal authority.

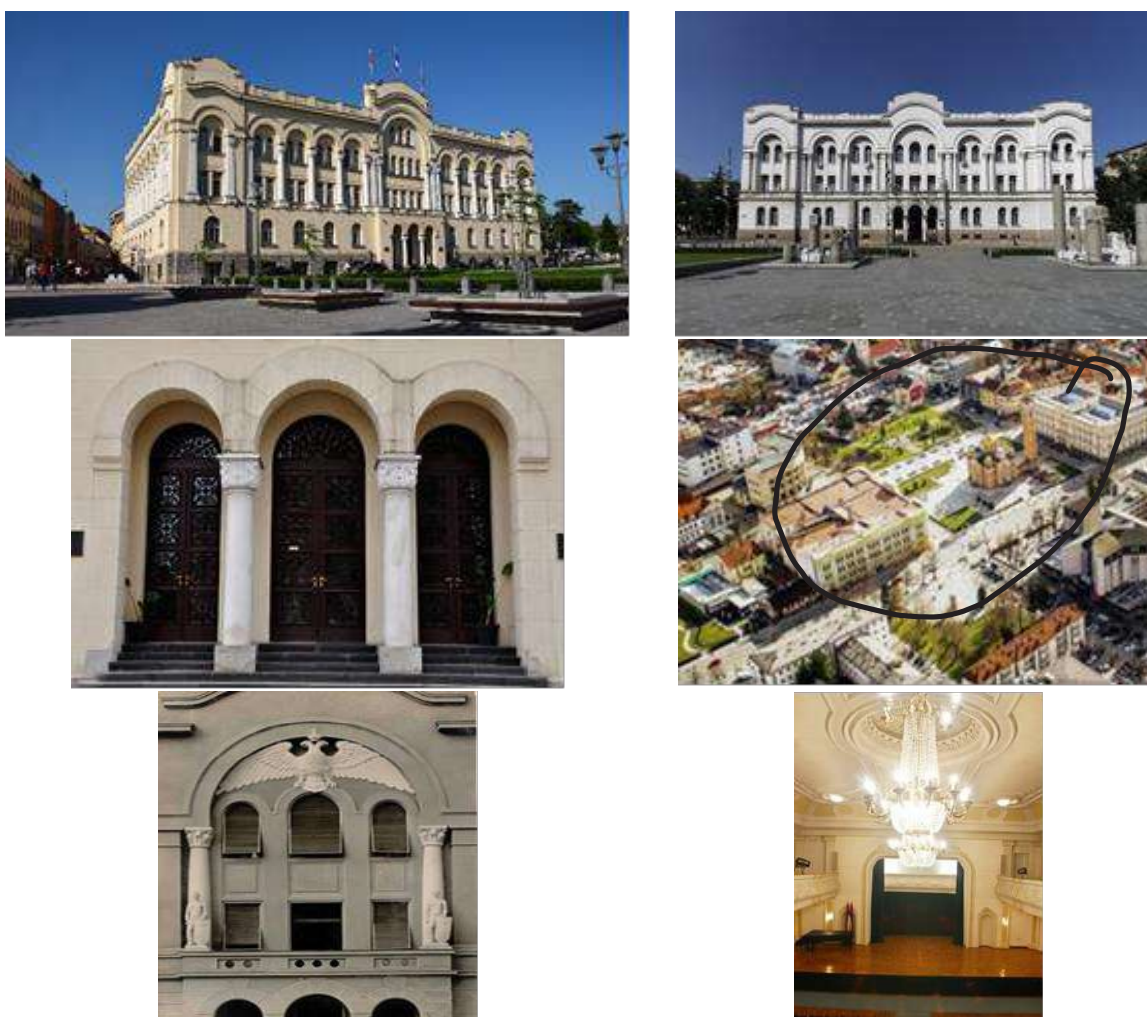


Fig. 7. Royal administration building of Vrbaska banovina

2.7. Royal administration building of Vardarska building

The competition for the administrative building of Vardarska banovina in Skopje ended with the victory of the Belgrade architects, Russian emigrants Viktor Lukomski and Ivan Rik. The proposed design was very pretentiously conceived for the environment in which it was to be built and with certain parts of the building that could not evoke associations with the administrative and governing functions of the state. The fate of this competitive work was similar to others which won in competitions for other banovina administration buildings, that is, it was decided not to implement it. Czech architect Viktor Hudak was the author of the design according to which the building was constructed between 1936 and 1939. This design is still today one of the most monumental buildings in Macedonia. The very massive building has an inner courtyard and a large hall. The external appearance is defined with, for that time and environment, a modern architectural language, with refined vertical dividers and emphasized wide segments containing geometrized decorative schemes, shown in separate parts, which is especially emphasized in the central avant-corps. In relation to the then existing urban fabric of Skopje, this building undoubtedly seemed contemporary and representative, regardless of the wishes of the then ban, who had made the decision that "all public buildings in Skopje should be built exclusively in the national style". The interior was much more ornately design and with hints of national artistic peculiarities, with cabinets, salons and halls decorated with wood carvings, which were designed by Dragutin Inkiostri Medenjak, known for the revival of national decorative art. In contrast to the buildings of the Dunavska banovina, where the architect Dragiša Brašovan, as the author, stylistically united the exterior and interior decoration, in the Vardarska banovina building there is a certain discrepancy in the stylistic expression of the exterior and interior [21].

The Banovina Administration of Vardarska Banovina governed here until the collapse of the Kingdom of Yugoslavia, and after World War II, various state functions were housed in it. As a result, the interior of the building was gradually but constantly remodeled, and one of the reconstructions followed the earthquake in 1963. A significant intervention was carried out in 1996, when a number of completely new rooms were built through remodeling. In 2008, the implementation of the most extensive renovation of the upper part of the building began. In an effort to emphasize the independence of the newly created state, Macedonian politicians resorted to specific imitation of some monumental buildings of other countries.

In order for the Macedonian parliament to resemble the German Reichstag, the dome of which was restored, but in the new, glass version by architect Norman Foster, domes were built over the lateral avant-corps and a large glass dome over the great hall for the parliament session [22]. At the same time, the entire building was added one floor. The majority of society opposed these changes and criticized them, but they were carried out anyway. It is believed that one of the key reasons is actually the erasure of the past that the building is associated with, because it was the seat of Vardarska banovina before World War II, that is, the territory was a part of the Kingdom of Yugoslavia, which is reluctantly mentioned. Today, this building houses the seat of the Assembly of the Republic of Macedonia, as well as a number of other important state functions. The cabinet of the President of the Republic, the President of the Assembly and numerous other state offices are located there (Figure 8).



Fig. 8. Royal administration building of Vardarska banovina in Skopje

2.8 Royal administration building of Primorska banovina

The process of building the administrative building of Primorska banovina was very slow and turbulent at the same time. A public tender for the Design of the new Banovina administration was announced in 1930, with the indication that the Banovina palace should be designed "...in a free monumental style, but its general character and shape should contain a classical rhythm, taking into account the stone building material that will harmonize with a coastal landscape". The original location was chosen very close to the extensive walls of Diocletian's palace, and then, after the competition terminated without results, the place for the construction of the banovina was changed several times.

In all further designs, the building was always monumental, with elements of architecture of historical styles. Such construction was opposed by many architects from Split, and fierce reactions in the public were frequent. Therefore, in 1936, the Banovina authority decided to prepare the ground for the construction of an administrative building at a location far from Diocletian's Palace, on the road that led along the coast to the urban part of the city with luxurious family villas [9]. The building of Primorska banovina was realized according to the modified design of the first-prized project from the competition from 1937, which is attributed to the architect Vladimir Turina and collaborators Nikola Despot and Vid Vrbanić. At the time it was built and long after World War II, this building stood alone on the seashore, so its dimensions and geometrized cubic mass were strikingly out of harmony with the background. However, its architectural envelope was thoughtfully adapted to the traditional construction of stone blocks, which were perfectly dressed and connected. The minimalist expression of the facades and a certain rigidity of all applied elements are undoubtedly the result of several influences that met in the coastal region, modernism, architecture that developed especially in Italy during the reign of Mussolini and the local tradition of building unornamented stone buildings for multi-family housing [10]. After World War II, in addition to the building of Primorska banovina, which became a sub-banovina with the formation of Croatian banovina in 1939, various administrative institutions were housed in this building, and nowadays the city administration of Split is located there (Figure 9).



Fig. 9. Royal administration building of Primorska banovina

2.9 Royal Banovina administration building of Dunavska banovina

Among the Banovina buildings, the Dunavska Banovina Complex represents a complete novelty in terms of its boldness, functional rationality and almost timeless architectural expression, which surpasses the usual qualifications of modernism to which architecture is most often attributed.

The tender for Dunavska Banovina was Yugoslav in its character, with the exception that Russian creators, who lived within the borders of the Kingdom of Yugoslavia, could participate in it on an equal basis. Ban Daka Popović, a prominent builder from Novi Sad and the first ban, was most responsible for organizing the tender. The first prize was not

awarded, and the investor, Dunavska banovina, did not accept any solution, instead opting for the architect Dragiša Brašovan (1883-1965). Dragiša Brašovan, by the indisputable consensus of all theoreticians, is ranked at the very top of the interwar architectural creativity of Serbia. The Dunavska Banovina complex was built within the town plan, which envisaged that numerous buildings related to the banovina administration would be built as part of a pre-designed single entity. This was a procedure that was provided for other newly designed banovina complexes, but it could not always be consistently implemented. In Novi Sad, due to a well-chosen location, outside the densely built fabric of the city, it succeeded in the best possible way. The homogeneity of the built forms has been preserved to this day, although the modern way of life, the density of traffic, the way buildings are maintained have somewhat influenced that they are often not perceived that way. In a broader sense, the building for Banovina officials built in 1930 and the model school "Kraljević Andrej" (today the "Isidora Sekulić" grammar school) constituted the single entity with the main Banovina buildings, while the Palace of the Police Administration was not realized. The designer of these supporting buildings was civil engineer Slavko Kosirović [13]. The complex of the Royal Banovina Administration of Dunavska Banovina is very well laid out in terms of urban planning. It is located in the already defined urban matrix of the newly commissioned Boulevard that opened onto the Danube. Considering the importance of state administrative buildings and their implied monumentality, as a rule, they are placed as independent structures, with free areas around them so that they can be observed well as city landmarks. In setting the outlines of the Banovina complex, the architect Brašovan moved the building line from the boulevard street line, which resulted in a more representative access area, with a roadway for stopping cars right in front of the central entrance, emphasized by columns and a deep porch. This setback also enabled a better view of the tower, which remained along the street line and which dominated the entire area with its height for a long time. The setback from the regulation line was also carried out on the western side, where the building is rounded, so a plateau was created for public events, which was covered with greenery over time. The massive roundness of the western facade has become a recognizable motif that marks the intersection of the boulevard and the street that leads to the strictest center of the city, to the square where the town hall is located. Between the administrative building and the town hall, a narrow street of local character, parallel to the boulevard, was built for the entrance to the town hall building, i.e. the ban's apartment, as well as for the supply of both buildings. The design turned out to be extremely well conceived in terms of access control and the speed of internal communication between employees. Most administrative buildings in other banovina centers represent a single body, more or less developed, and contain most of the functions provided by the program.

The exceptions are the Vrbaska and Dunavska banovina complexes, but they are conceptually very different. The buildings of the Banovina administration and the Ban's residence in Banja Luka were placed so that they form two sides of the square, on which the Orthodox Church of the Holy Trinity was placed. There is no good visual connection between the buildings, as the volume of the church hinders this. In the case of Dunavska banovina, two complementary functions, the banovina administration and the town hall with the ban's residence, are also separated into two buildings, but so that they are closely connected and clearly represent a homogenous architectural unit.

The design that Brašovan opted for came precisely from the concept of good and simple spatial organization, which has long been applied to buildings of judicial, administrative, governing and business functions. The administrative building of Banovina was conceived in such a way as to define one building block with several interior atriums. The basic dimensions of the building are imposing, 180 m x 54 m, and it was organized to have a basement, ground floor, two floors and an attic. In a structural sense, the building

consists of two distinctly elongated longitudinal parallel tracts, connected at the ends, on one side by a vertically placed tract, and on the other by a semicircular one, between which there are four atriums and a central hall. The atriums are separated by short transverse tracts that also house communications and offices. A slender tower with a height of 40 m on a square base was constructed at the northeast corner of the building. This compositional scheme gives the building an extremely elegant silhouette. As for the smaller building with the hall and the ban's apartment, the functions are intertwined in the middle, and in the wings the design was simple, along the corridors there were rooms facing the local street that separated it from the Banovina. This extremely attractive and complex solution was modified several times later. The complex of apartment buildings is one of the few that is monochromatic, lined exclusively with white marble and with decorative reliefs and profiles that are also white and made of the same material. Varying the size and arrangement of the window openings is what compositionally breaks the monotony of the facades and makes them very dynamic. What is transposed into the physical is the inner, spiritual content, and this can be clearly visible through the form of the structure, but also given only as a hint. Thus, for example, considerations of the basic form of the Administrative Building were brought into connection with the association that many observers of the building spontaneously expressed through the nickname "boat on the Danube" [14].

In many texts, the Dunavska Banovina complex is considered a sculptural, timeless expression of a skillfully chosen architectural theme. It is a testimony to the harmonious agreement of form, function, art and human needs. The complex of buildings of the Royal Banovina Administration building of Dunavska Banovina is not only the greatest work of the architect Dragiša Brašovan, but also one of the most successful examples of architecture between the two world wars in the former Kingdom of Yugoslavia. Today, both buildings house the Provincial Government and the Assembly of the Autonomous Province of Vojvodina, and the entire complex is protected as a cultural monument of exceptional importance (Figure 10).





Fig. 10. Royal Banovina administration building of Dunavska banovina

3. Conclusion notes

Some buildings, just like people, are destined to always be in the center of attention, to be noticed immediately, to reach the most subtle emotions and to be remembered for a long time. Evidence for this claim are the Banovina buildings of, the administrative centers of the territorial units (Banovinas) formed in 1929 within the Kingdom of Yugoslavia. All the Banovina buildings still exist today, they are highly valued and protected as national cultural monuments.

What makes them interesting for comprehensive research is their architecture. The buildings are very different in terms of time of construction, style and structure, their status ended during the World War II, but today they still house important state functions.

Comparing the architectural solutions of nine Banovina administrations, especially their outer envelope, as the main reflection of aesthetic design, is very interesting. A relatively small number of buildings, only nine examples, reflect the dichotomy of architectural creativity in the Kingdom of Yugoslavia between the two world wars. The fact that the bans, as high state officials, had a great influence when deciding on the appearance of administrative buildings speaks in favor of the relatively significant influence of politics. At the same time, under their influence, the way they manage finances during construction, as well as a certain type of rational reconstruction and use of already existing buildings (Savska, Dravska, Drinska banovina) comes to the fore. Other examples reflect the desire to give great importance to the ban's administration and status as a manager by building new, monumental buildings.

Another conclusion that can be drawn is related to the choice of architectural style, i.e. the direction. The entire era of the reign of Aleksandar I Karađorđević, then the viceroy, Prince Pavle Karađorđević, was marked by the dynamic construction of state, military and public buildings. The king's fondness for the architecture of historical styles, which was forced by the majority of architects, emigrants from Russia after the October Revolution, is obvious, because the largest number of these buildings were built in the spirit of a kind of belated academicism, eclecticism and other unusual designs in which art deco and Modern elements were awkwardly inserted. However, over time, as shown by the examples of Vardarska and Dunavska Banovina, that attitude changed and a modern expression was accepted in the design of state buildings. Modern art, in general, can be treated as a kind of instrument with which society develops progressively. Modern architecture makes a departure from most of the attitudes of stylistic architecture in its most retrograde form, thus in a certain way it unifies the uniqueness of the principles of architectural creativity on the territory of the kingdom. Modern is not tied to the historical styles applied in Europe, and present in the Austro-Hungarian monarchy, to which some parts of Yugoslavia belonged, nor to the sentimental application of the Serbian-Byzantine revival, popular in other parts of the new state. All the aforementioned dilemmas were resolved in the last constructed building of Dunavska banovina. Neither on the Banovina administration

building nor on the council building is the modern direction completely authentic, puristic, because numerous details and elements are woven, thus sensibly associating with various other architectural directions.

Finally, all the administrative buildings and residences of banovinas have survived to this day without significant alterations that would have caused them to lose their identity. The fact that most of these buildings are still called banovina by the inhabitants, even though their functions have long since changed, best speaks of the fact that their monumental, albeit diverse, stylistic forms are deeply imprinted in the consciousness of a society that has long since disintegrated into parts, of which, after the First World War, it was ambitiously composed

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HISTORICAL NOTES AND ARCHITECTONIC SPECIFICS OF KADY (KADIN) BRIDGE CONSTRUCTED 1469/70, OVER STRUMA RIVER NEAR KYUSTENDIL

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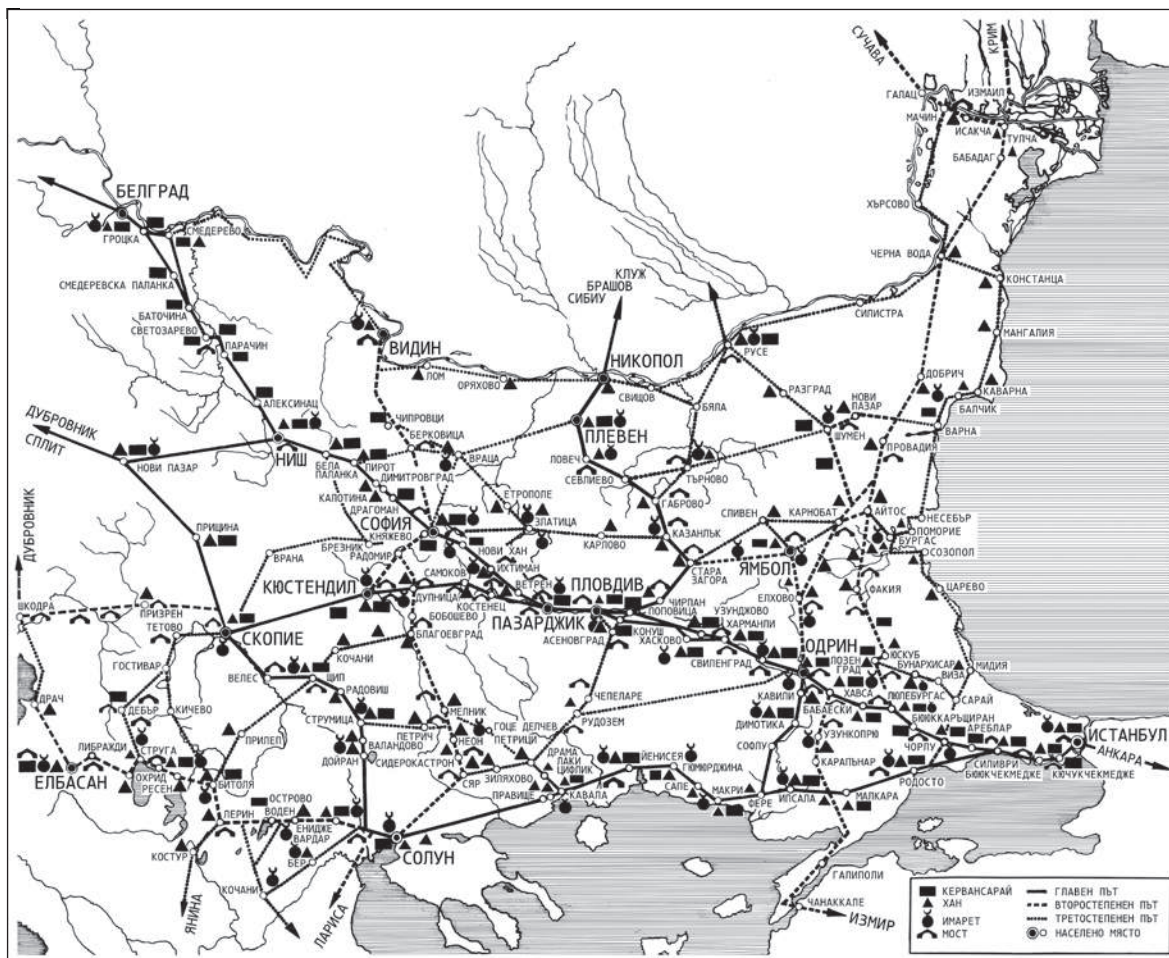
Abstract: *During the 14th and 15th centuries the expanding Ottoman Empire conquered new lands in South-Eastern Europe. As they needed a working road infrastructure for their military plans, they continued taking care of the restoration and construction of structures along the routes of utmost importance. Among these is the Kady (Kadin) Bridge over the Struma River built in 1469-1472. It is believed to have been built by the Islamized Christian Ishak Pasha, although the authorship issue has not yet been resolved. Architecturally, the bridge utilizes several different techniques uncharacteristic of the artisans of the time. The origin of the bridge has yet to be discovered.*

Key words: *old bridges, stone bridges, Ottoman bridges, history of architecture, bridges of the Balkan Peninsula, Bulgaria, Turkey, Iran*

Upon the adoption of the then-current road network from the late 14th and especially the turn of the 15th century, the Ottoman administration continued taking care of the restoration and construction of bridges along the routes of utmost importance. At that time the most important route for the Empire's military plans is the track of the antique Via Militaris leading directly west. But the section of the route between Sofia and Belgrade is unsafe because of military activities. Therefore, in 15th century, merchants and ambassadorial delegations mainly used the old Adriatic (Bosnian; Toplicki for the Serbs) route of the Dubrovnik merchants, splitting to the left from Via Militaris (also called Tsarigradski; Stambul Yolu [Turkish]), before entering Trajan Pass leading to Momin Pass – Sestrimo and via Rila to Kostenets – Dolna Banya – Samokovsko Pole – Cherven Breg (later redirected to Dupnitsa) – Kjustendil – Skopje – Pristina – Novi Pazar – Priepole – Plevlya – Foca – Trebine and on to the Adriatic, and through another branch – Sarajevo to Central Europe.[1]

A direct duplicating, secondary derivative of the Adriatic Road is also in operation, passing east of Sofia, near the settlement of Slatina (present-day district of Sofia). It derives from the main route in the North-East near Pristina, leading to Vranja – Surdulitsa – Stressimirovtsi – Znepole – Graovsko plain and Breznik, and connects to the Tsarigradski route. Many embassies, missions, merchants, military intelligence officers use this road during 15th century, enroute to Istanbul, but mainly its antique medieval path

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Road and infrastructure map – bridges, caravanserais, inns and imarets in the Eastern Balkans from the Ottoman Empire era, 15th – 19th centuries. Tuleshkov (1994)

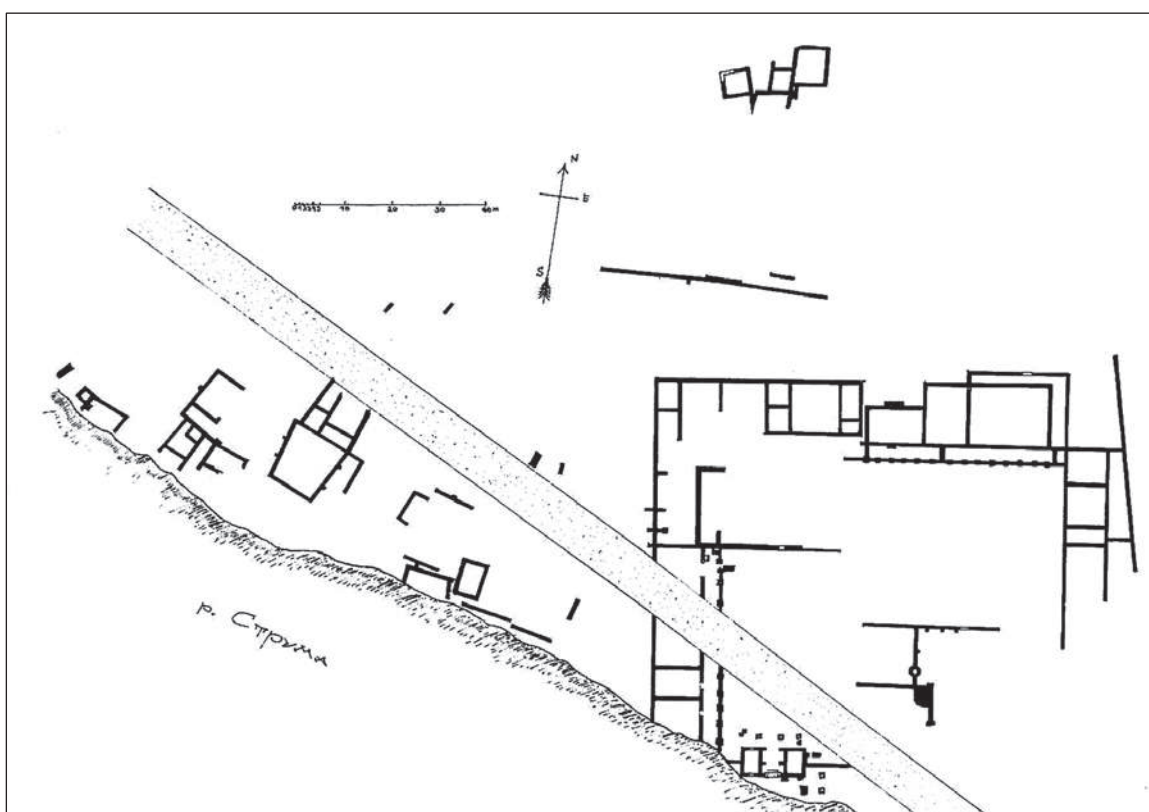
through Kjustendil, and much more rarely the derivative through Vranja and Sofia. Therefore along the main of these two routes several infrastructures are being repaired and rebuilt – the antique medieval seven-vaulted Arakchiiski bridge near Dupnitsa (referred to as “old”, and without a commemorative Ottoman inscription in 1530), the long-gone five-vault bridge over Razmentanitsa river near Dupnitsa, Kadin bridge near Kjustendil, the large antique medieval bridge over the Pchinya river near Sredno Konyare (reconstructed on the ancient foundations and vaults in 15th century) in Kumanovo region, the ancient Kamen Bridge in Skopje (reconstructed in 1421/2, 1451 – 1469), both in North Macedonia, the medieval Babin Bridge near Pristina, the extended bridge to Vuchitryn, both in Kosovo, etc.

Among these facilities, the one of utmost importance for Ottoman bridge-building in 15th century is the newly-built Kadin bridge in the modern-day village of Nevestino near Kjustendil, built as a substitute to the mostly ruined Roman bridge situated upstream, before Kjustendil.

Kadin bridge is constructed over the Struma River, according to his kitab, by the Islamized Christian Ishak Pasha (Turkish - Inegöllü İshak Paşa) during his first mandate as Grand Vizier to Sultan Murad II in 1468/9 – 1471/72. [2] This bridge is the first of his charitable public works in Europe. During excavations that lasted from 1904 to 1906 in the neighboring region of Moshteni, Yordan Ivanov discovered the remains of an ancient stone bridge connecting neighbourhoods of a large settlement on the banks of the river. One of the two parallel routes of the road from Constantinople - through Plovdiv, Belovo, Kostenets, Cherven breg and Kyustendil to the Adriatic, passed through here. [3]

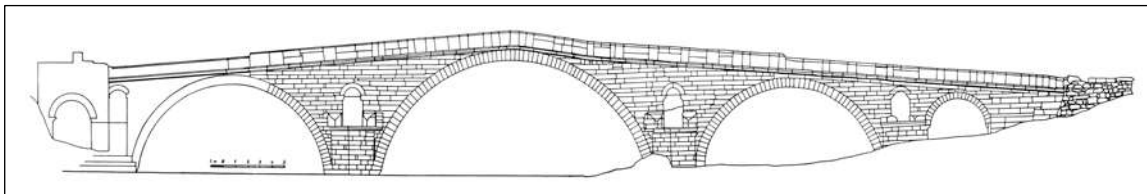


Kadiyski (Kadin) Bridge over the Struma River in Nevestino settlement, Kyustendil region, 1469/70. General view and details from the tarich and original road pavement



Partial plan of the 1902 – 1904 excavation of the ancient settlement and villa in Moshteni near Kady (Kadin) Bridge connecting the left and right river banks

The new bridge has a length of 89.5 m, and a hull width of 6.49 m. It is a five-vault, humpback bridge, with a central arch opening of 21.65 m, and side arch openings of 14.75 m and 13.70 m. There are two smaller vaultings on the river banks, and four relief openings above the foundations in the river. The upper sections of the low breakwaters on either side of the central relief openings are raised above the triangular platforms, with pointed faces made of rock – a solution unique to the bridge. This prevents obstructions to the outflow by objects carried by the inflow. This bridge also has the earliest, at least for the Balkans, layout of relief openings with a narrowing of the vaulting relative to the width of the opening below it. In these two features, it has similarities only with the Lüleburgaz bridge.

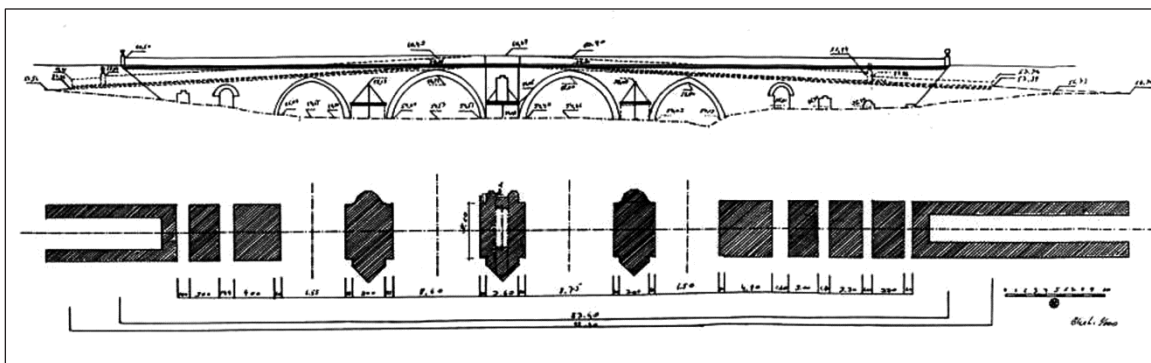


Photogrammetric façade plan of Kadiyski (Kadin) Bridge – NINCH archive; view from the bridge during restoration activities in 2020



Fragmentary view from the central part of the hull of Kady Bridge with the specific vaultings and clogging protection of the relief openings

The humpback bridge over the Küprüaltı river at Lüleburgaz is indicated by some authors to be Sinan's work of 1569/70, the reference being based on the dating of the culvert next to it, which he built for the grand vizier Mehmed Pasha Sokolu. However, from the travelogue of Benedikt Kurpešić, it is clear that this one of the two "long brick bridges" he saw whilst traveling from Lüleburgaz to Karštiran already existed in 1530, which means that it has nothing to do with the construction of the Grand Vizier's tower, completed according to the kitab of the mosque in 1569/70.



Sokullu Mehmet Paşa Bridge (15th century) in Luleburgaz, Kırklareli province (Bulgarian name – Lozensgrad). Photographic views from the end of 19th century on a marked superstructure plan by Bozkurt (1950), and current condition

The Lüleburgaz bridge, according to Orhan Bozkurt, is 92.60m long and 6.05m wide, with 4 large arches, the central ones being 8.70m and 8.60m wide. However, it also has six small side lower shore breaks and one relief opening above the central breakwater. Of these, four small vaults and the relief opening architecturally belong to the typology of the relief openings from Kadin Bridge. In addition, at the Lüleburgaz bridge, in an identical way, on both sides of this opening, above the breakwater, similar to the bridge over the Struma, stone pyramidal sides, directing the water outflow, step over the breakwater. (6c) Thus, all these identical elements lead to the conclusion that the bridge at Lüleburgaz (nowadays in the centre of the grown city) was most likely built by the same artisans who constructed the facility over the Struma River near Kyustendil. Therefore, the Lüleburg Bridge can be dated like it to the second half of the 15th century.



Sokullu Mehmet Paşa Bridge (15th century) in Lüleburgaz, Kırklareli province (Bulgarian name – Lozengrad). Photographic views from the end of 19th century on a marked superstructure plan by Bozkurt (1950), and current condition

The issue with the authorship of the Kadin bridge, like the Lüleburgaz bridge, has not been resolved yet. A suggestion has been made of artisans from the Adriatic coast, but it does not rest on architectural analysis. Indeed, the Adriatic artisans, who are mainly Italian, have been working all around the western Balkans since the 9th - 10th centuries, and in the 15th century the first Croats trained by them appeared, so theoretically there is some, albeit historically unsubstantiated, possibility that they reached Kjustendil through the route coming from the Adriatic coast. However, although these builders worked well with cut stone, it should be noted that until the 16th-17th centuries they did not apply relief openings to bridges, as well as overhanging hull linings over the facades of the vaulted structures and all types of vaults are made up of not very large trapezoidal squares, levelled in a common surface with the other linings. At the same time, the idea of a narrow vaulting of the rectangular opening by means of cantilever superstructures on both sides was carried out by Dubrovnik artisans during the construction of the arcade in Sarajevo in 1540, but the same is also the case with bridges on the Edirne - Istanbul route dating back to the second half of the 15th century. So maybe the idea came to Bosnia most likely from the Eastern Balkans.

Therefore, at Kadin bridge there are relief openings, which are uncharacteristic of the Adriatic artisans of that time, the cladding of the hull is overhung in one step above the faces of the vaults in the traditional Armenian manner and the relief openings end

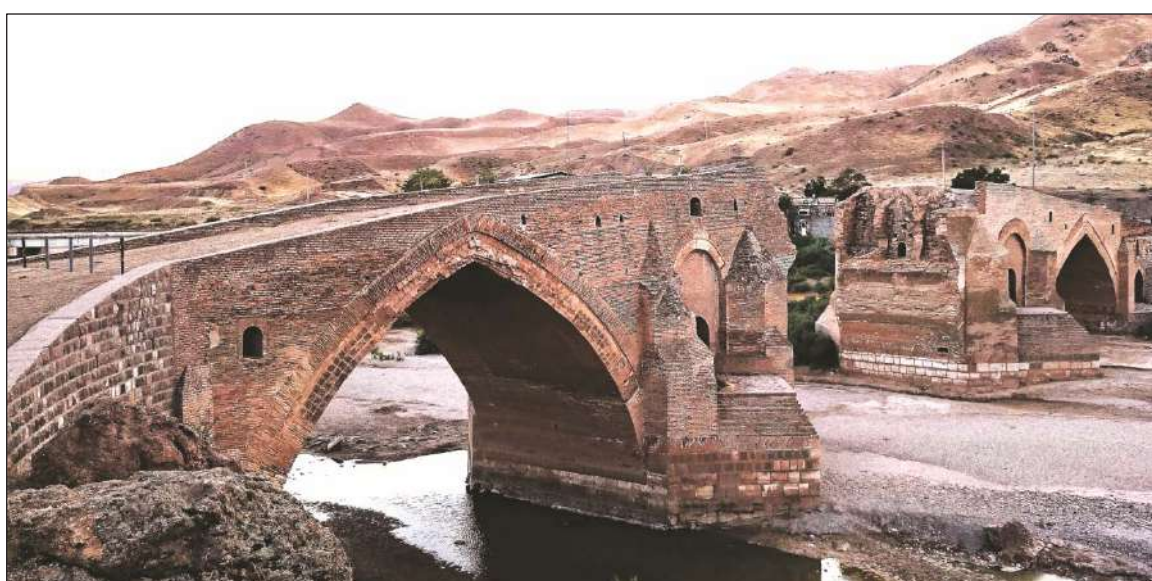
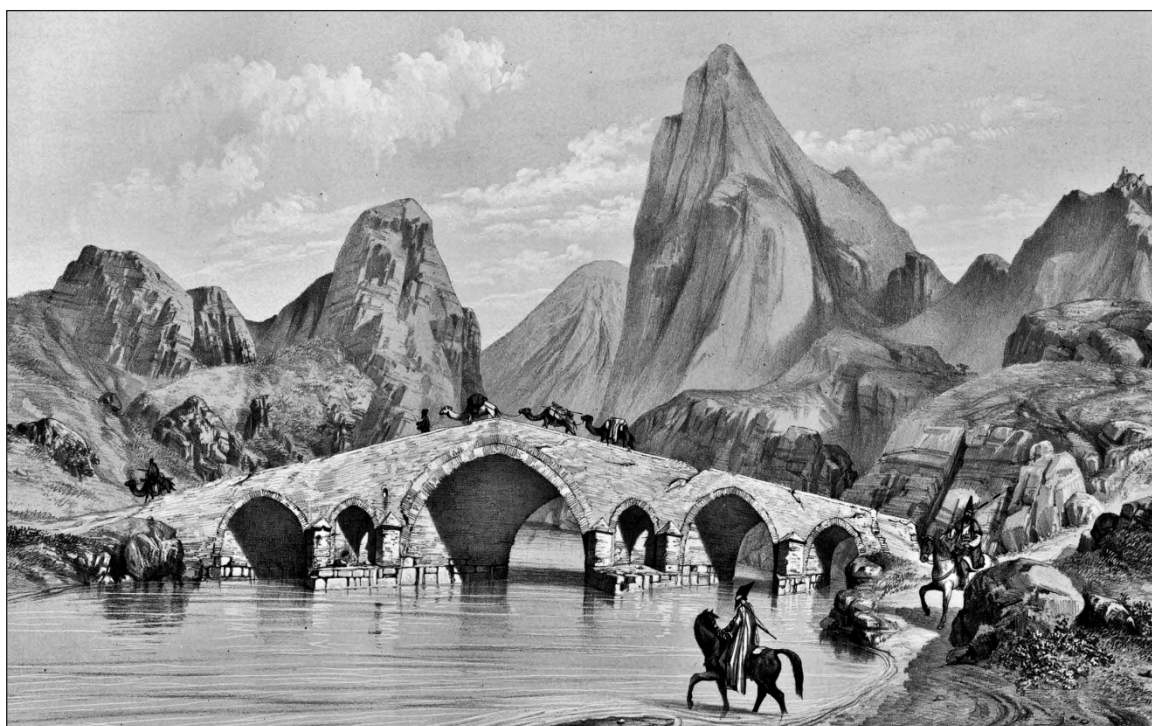
on the facade with carved stone arches of one whole, or of two segments connected at the zeniths. These techniques are not observed in Italian artisans of the time including those from the Adriatic coast region.

At the same time, in Armenian architecture since the 5th - 6th centuries, the practice of installing precisely carved stone forms on the facades, made of one or two segments connected at the zenith on the smaller vaults, was present in the same way as it is at Kadin Bridge.

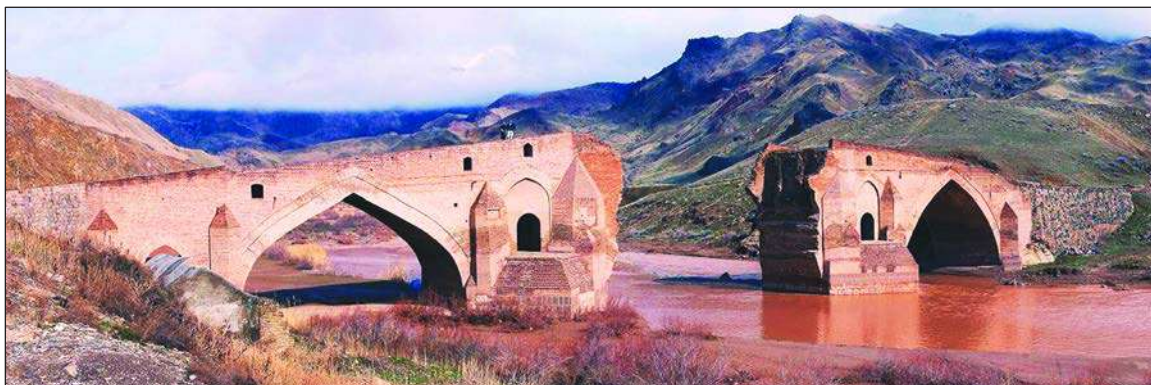


Armenian vaultings constructed with large curved elements – St. Joan church in Mastara village, Argatsotn province (5th century); the White church in Ashtarak, Argatsotn province (VI century); the Red church in Ashtarak, Argatsotn province (7th century); Artavasik church near the village of Burakan, Argatsotn province (730 – 740)

An analysis of medieval European bridges reveals that it was in Byzantium that the construction of bridges with relief openings was resumed and, as it seems, Armenian builders were directly involved in it. They also introduced the overhangs of the hull linings over the facades of the vaulted structures. It should be added that the sharp pointed protective elements on both sides of the relief openings, which are strange for the era and the place, are known not only from the bridge in Luleburgaz, but also from the one near Miyane (Turk. Kizil Hauzen, Iran. Pol-e Dokhtar, East Azerbaijan, now in Iran) over the Kizil Hauzen River, constructed in the 12th - beginning of the 13th century and rebuilt in 1475/6-1483/4. Territorially, it looks like the work of Azerbaijani builders, but in these territories until the 15th-16th century mainly worked Armenian artisans. [4] At that time, local Azeris, trained by them and by Central Asian builders, establish their own



The Iranian brick bridge Pol-e Dokhtar (Pol-e Kizil-Hauzen, 12th – turn of 13th century, general reconstruction 1475/6 – 1483/4) over the Kizil-Hauzen River near Miyane, Eastern Azerbaijan, nowadays Iran



academy, in which, however, Central Asian and Persian architectural features predominate. This is why the bridge at Miane is lined with bricks, unlike the stone structure of Kadin bridge. The protection of relief openings with forward retaining elements on both sides is also present in other Azeri and Iranian bridges from the 15th - 17th centuries. Significant examples are the Tadjan Bridge (Pol-e Tadjan, or Pol-e Kheshte, 16th century) over the Shim Rud River in the village of Tajan Guke, Gilan Province, East Azerbaijan, Iran, as well as the Red Bridge (Azerbaijani Qırmızı Köprü, Georgian Tsiteli Khidi, 12th century, major reconstruction in 1647) over the Aran River near the Armenian town of Julfa, Lower Kartli, Azerbaijan. (9+10) A hypothesis can be proposed that the bridge over the Struma River was built by an Asia Minor architect - an Armenian, or an Azeri, perhaps even from the territory of nowadays Azerbaijan, where, however, in the 15th century, the builders were both Azeri and local Armenian artisans. This is the case, for example, in 1514 when the Ottoman



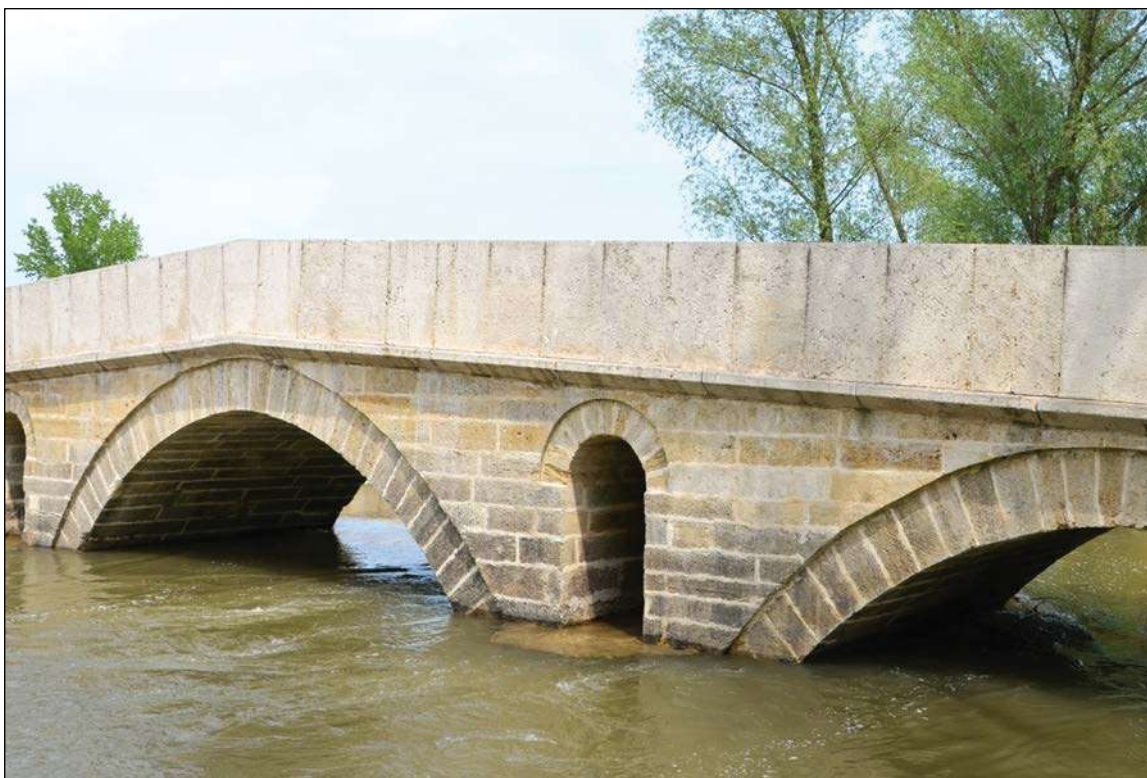
The Iranian brick bridge Pol-e Tamijan (1354/5) over the Tamijan river near Rudsar village, Gilan province, Iran



The Azerbaijani Red bridge (Qırmızı Köprü, Sınıq köprü – The broken bridge, 12th century, reconstructed in 1647) over the Efram river (Georgian – Hrami) near Julfa, Lower Kartli (Azerbaijani – Nahicevan)

Empire captured the great cultural and artisanal Safavid centre of Tabriz, from where several thousand artisans were moved to western Asia Minor and eastern European territories, including many builders, decorators, carpenters, both Azeris and Armenians. Builders from this territory already worked for the sultans in the previous century. Such is, for example, Hajja Ivaz Pasha from the province of Tokat, who designed and built the most significant religious and public buildings in Bursa in the first decades of the 14th century. [5] And in 1514, when the city of Tabriz was captured, hundreds of master builders and decorators, artists and carvers, both Azeris and Armenians, were resettled from there to the west of the Empire. It was them who introduced painted interior ceramics to palace and cult architecture.

Kadin Bridge has other similarities with early Ottoman bridges from Asia Minor and the eastern Balkans besides the overhangs of the linings over the faces of the arches and the presence of relief openings. One of these other architectural details is the specific, chamfered at about 45° , molding passing between the main bridge hull and the stone parapets stepping above it. Such moldings are found on the earliest Ottoman bridges - the triarch one with relief openings Fatih bridge over the Tundzha River (1451/2) in Edirne, the large Koyunbaba köprüsü in Osmanjik (1484/5 - 1488/9) and even became the most preferred for bridge construction from the Golden Century.

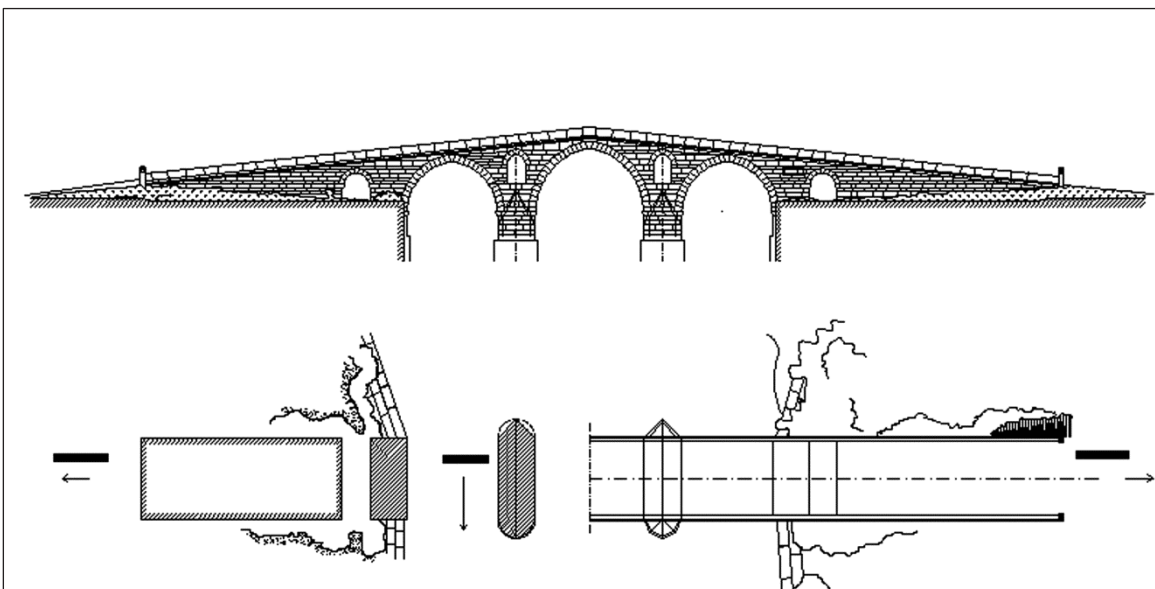


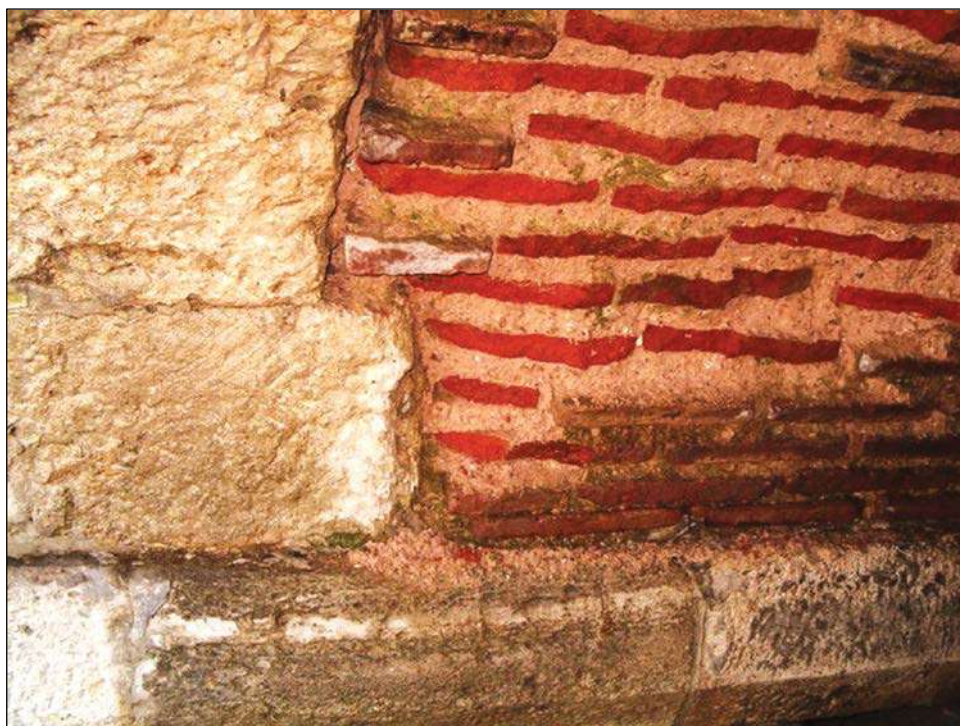
Mehmed the Conqueror bridge (Turkish - Fatih Köprüsü, Bönce Köprüsü, 1451/2) over the Tundzha river in Edirne, Turkey

The same molding is later placed on the Byzantine bridge over Harâmidere (Harâmidere Köprüsü) later known as Kapı Ağası Köprüsü (or Yakup Ağa Köprüsü) on the Constantinople Road between the Küçük- (6th century, between 867 and 886, 1455, 1498/9 etc.) and Buyukcekmedze bridges. The bridge being of Byzantine origin is evident from the brickwork of the oldest arches, so it is wrongly assumed that this is a newly built Ottoman bridge. In this case, there is a renovation by the Ottoman administration in the time before the construction of the two big bridges (one of which – Büyükçekmece, is the work of Sinan), i.e., it was carried out before 1568. [6] However, its architectural features suggest that it was done much earlier, most likely in the second half of the 15th century,



The Haramidere bridge (Harâmidere Köprüsü, Kapı Ağası Köprüsü, Yakup Ağa Köprüsü, 6th – 7th centuries, renovated circa 1455 and reconstructed circa 1560 by mimar Sinan) over the Haramidere river, Beylikdüzü district, Istanbul





The Haramidere bridge (Harâmidere Köprüsü, Kapı Ağası Köprüsü, Yakup Ağa Köprüsü, 6th – 7th centuries, renovated circa 1455 and reconstructed circa 1560 by mimar Sinan) over the Haramidere river, Beylikdüzü district, Istanbul, view from the early Byzantine construction

after the capture of Constantinople in 1453. It was most likely renovated together with Kyucukcekmeje in 1455. The bridge over Haramidere, however, is recorded by Mustafa Çelebi in Sinan's biography as his work, which means that in the 60s of the 16th century, he undoubtedly carried out a major repair of it, with partial reconstructions on its old abutments. This is established by the differences in the stonework between the old lower and newer upper parts of the hull and the central vault, by the mixture of semi-circular and pointed vaults, by the presence of large antique formwork imposts in the end vaults, etc. But in the end, a deeper study of this structure with the use of laboratory technologies will clarify not only the periodicities in the construction, but also how far the reconstruction of Sinan extends, compared to the older repairs.

In conclusion, it should be noted that the formation of Ottoman architecture, and bridge-building in particular, during the first two or three centuries of the Empire's existence, is a process of mixing architectonics and technologies, the work of artisans of different ethnic and religious origins, among whom the Armenian, the Syrian, the Greek and the Bulgarian predominate, and from the 16th century also the Azeri.

NOTES

[1] Tuleshkov, N. Roadside architecture of the late Middle Ages. – In: Bulgarian Architectural Heritage, book 1, 1994, 29-41

[2] Direct translation of the Kitab: "By the high order of Ishaq Pasha, one of the greatest viziers and noble chiefs, who does good deeds purely from the soul's impulse, in order to receive from the creator above the corresponding retribution, he ordered that this sacred bridge be built in this country for the free passage of travellers. May the almighty God fill his life with joy and gladness. In the year 874 (1469/70)"

[3] Ivanov, J. Kadın Köprüsü. – In: Bulletin de l'Institut archéologique bulgare, I, Sofia, 1910, pp. 197 – 201, rs. 100

[4] In the 15th - 16th centuries, an Azeri Muslim academy of construction was formed in the Azerbaijani ethnic land, uniting the architectonics of Armenian stonework techniques and Central Asian brickwork. However, local Armenian Christian craftsmen continue to work in a number of cities of this territory. They are among the main builders, and other craftsman profiles in Iran as well. For example, quite significant buildings, including palaces and bridges in Tehran and its region, were the work of Armenian architects and craftsmen from Julfa (now a district of the city), who moved here in the 16th century from the large city of the same name in Lower Kartli, Azerbaijan

[5] Tuleshkov, N. The Bridges of the Ancient and Medieval World. Part II, C., 2022, 669 – 677

[6] KantarAtlas, kantaratlas.blogspot.qa/2017/12/?view=timeslide. Although there is no record of the time of its construction, several Turkish authors refer to it as the work of Sinan (for example: Orhan Bozkurt, Koca Sinan'ın Köprüleri, İstanbul 1952, pp. 12-19, lv. 1-5; Cevdet Çulpan, Türk Taş Köprüleri, Ankara 1975, p. 148, nr. 88; Gülgün Tunç, Taş Köprülerimiz, Ankara 1978, pp. 93-95, etc.). The reason for this is given by the mimar (architect) Sinan himself, who pointed it out to his biographer Mustafa Çelebi, for his work (Sâî Mustafa Çelebi. Tezkiretü'l-ebniye)

SURVEY OF SOME OLD RIVETING BRIDGES, WITHIN URBAN HERITAGE PROTECTION AREAS IN CZECH REPUBLIC

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Abstract: Bridges, which belong to cultural heritage and are with high significance, perform an essential part of the socio - cultural and physical phenomena belong to the tourism trends. They represent the identity of the local population and at the same time fascinate and attract visitors. The value of cultural heritage is constantly increasing due to the naturally conditioned service life, as well as the unstable environment in terms of new technologies and changes of life style. Thus, care about the preservation of cultural heritage in the XXI century is not only declarative obligation of the authorities, but also the moral role of the whole modern society, which creates the conditions for preservation of cultural heritage in the future by recognizing its importance.

Key words: assessment, steel bridge, damages, repairing measures.

1. Introduction

Bridges are part of every route, whether for rail, highway, cycle or pedestrian traffic in Czech Republic[20]. They are large structures that complement the landscape or urban character of cities[1-9]. The second half of the 19th century brought a boom in railway transport. It was at this time that many railway bridges were built. They were stone or brick bridges with small spans. For larger spans, steel bridges were made. Some of them are still in use today[10-19]. Mostly they are found on local lines or less frequented routes. Approximately 400 bridges are protected, four bridges are national cultural monuments and others within urban heritage protection areas[20]. However, age has left its mark on many bridges, particularly in the form of corrosion or the development of cracks. Many valuable metal bridges are not listed and, as a result of their rapidly declining physical integrity, an important footprint of our nineteenth and early to mid-twentieth century industrial heritage is in danger of being irretrievably lost. These bridges have fulfilled their purpose in terms of their required service life, but by no means all of them can be replaced by new bridges. From the point of view of preserving our industrial heritage, it is, on the other hand, desirable to conserve at least some of them, and to maintain them in a worthy condition(Fig.1).

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Fig. 1. Railway bridge under Vyšehrad[20]



Fig. 2. Suspension bridge in Stádlec[20]

In many cases, these are beautiful and unique structures that are no longer commonly encountered in the contemporary world. They are the pride of our bridge engineering from the Austro-Hungarian period and from Czechoslovakia before WWII. The high level of expertise of the engineers of that time, the extent of developed industry, and the quality of work in the production and construction of steel bridges were the prerequisites for the fact that some of these bridges have survived until now. It is, therefore, our duty to preserve this industrial and cultural heritage to an appropriate extent for future generations. However, this means that we must take care of selected representatives of old steel bridges so that they can serve their main purpose – carrying traffic, albeit with some restrictions in usage, for several decades to come. One option, for example, is to transform an inoperative road or railway bridge into a bridge carrying footpaths and cycle tracks. Steel bridges that came into service 100 years ago and earlier do have a different structural design compared to modern bridges. The main characteristic of old steel bridges is the practice of riveting. Individual members that could not be made by rolling were at that time made by riveting. Even the joints in the main structure were riveted. It was not until the second half of the 20th century that welding technology became customarily used in the construction of steel bridges(Fig.2). Another specific feature of the old steel truss bridges is the fact that each member has a different cross-section, because at that time maximum savings were made on the amount of steel material used, regardless of labour requirements. The design of steel truss bridges used simplified structural models that were compatible with the computational means of the time. Approximate static models were used to deal with statically indeterminate structures, which contained certain reserves that can still be used effectively today. Similarly, we can better describe the interaction of the individual load-bearing elements of a steel structure, which was impossible to do until recently. In order to maintain the functionality, safety, and protection of these structures in the face of increasing traffic volumes, it is necessary to develop new methods of survey, assessment, and strengthening of these structures. Therefore, from 2018 to 2022, the Ministry of Culture of the Czech Republic supported project DG18P02OVV033: The Methods for Achieving the Sustainability of Industrial Heritage Steel Bridges. In cooperation with the National Heritage Institute and the operators of railway and road infrastructure, the project developed methods and technologies for the surveying and assessment of historic cast iron and steel structures, with a focus on failures due to corrosion and fatigue. Non-invasive and reversible strengthening procedures for elements and connectors, new technologies for surface treatment of historic bridges, and innovative corrosion protection systems using nanotechnologies were developed. Technical inspections of steel bridges were carried out as part of the project, following the results of the comparative mapping of industrial heritage. Subsequently, the results of the project were applied in pilot case studies of typical existing bridges(Fig.3).



Fig.3. Details of riveted bridges – Mokropsy[20]

Fig. 4. Bridge in Skochovice[20]

The multidisciplinary project is based on the cooperation of leading experts from the Faculty of Civil Engineering, Faculty of Architecture (including the Industrial Heritage Research Centre), Faculty of Mechanical Engineering, and the Klokner Institute of the Czech Technical University with leading experts from the National Conservation Agency and representatives of operators (ŘSD ČR, SŽ and TSK Praha). The achieved findings are being applied not only in construction practice, but also in university lectures and in the lifelong education of civil engineers. The results are verified on existing steel bridges. The target group includes experts in the field of heritage structure protection, designers, construction companies, employees of the responsible authorities, and students at secondary schools and universities. The critical catalogue herein presented is the result of this five-year period of research, and recapitulates both its results and the state and outlook of historic riveted bridges in the Czech Republic(Fig.4).

2. On the history of metal bridge construction in the Czech Republic

Metal bridges began to be built in the Czech lands in the middle of the 19th century, a process that was related to, and in conjunction with, the development of the railway network. The oldest metal bridges in the Czech Republic date back to the construction of the Ferdinand Northern Railway. Several cast-iron bridges were built in Moravia between Brno, Přerov and Olomouc in order to carry road traffic over the railway tracks. One of them was the arched bridge near Nemilany, which dates from 1868 (Fig. 5a). It was a three-hinge arch bridge with a span of 12.88 m. The bridge had 4 main girders on which brick arches were placed as a supporting deck for the road. The main girder was of modular construction and consisted of individual members connected at the abutments by bolts. When the railway line was electrified, it was necessary to increase the underpass height beneath the bridge. Therefore, it was taken down in 1964 and replaced with a new steel girder bridge. The bridge at Hostěrádky–Řešov, dating from 1869 (Fig. 5b), had a similar history. Again, it was a cast-iron bridge of the same construction. The span of this bridge was 13.5 m and it was demolished around the mid-1980s.

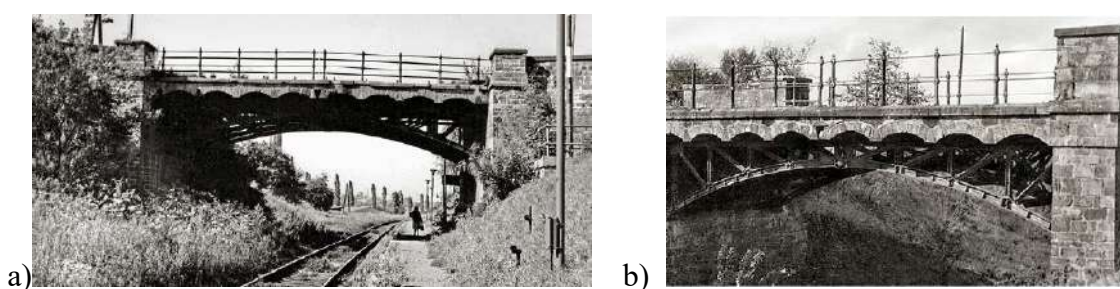


Fig. 5. Cast iron bridge near Nemilany (a) and Hoštěrádky–Řešov (b)[20]

In the production of steel from pig iron, oxidation removes undesirable elements, especially C, Si, P and S. In 1784, the production of **wrought steel** was introduced by the Englishman Cort. Production was carried out in a puddling furnace, in which pig iron and scrap are processed in a doughy state with air and constant stirring (puddling). Wrought steel is heavily saturated with slag, which is stretched by rolling to length, and is therefore layered. Welding steel has a low carbon content and is therefore mild. In the rolling direction it has a strength of 330 to 400 MPa and a ductility of 12 to 25 %. In the transverse direction the strength and ductility are considerably lower. Wrought steel is ductile and forge-weldable. Later, wrought steel was completely replaced by **mild steel**. This is the collective name for all steels made in the liquid state. Mild steel has a minimum tensile and compressive strength of 370 MPa, a minimum ductility of 20%, high toughness and almost the same properties in all

stress directions. According to the method of production, a distinction is made between converter steel (invented by the Englishmen Henry Bessemer in 1855, and Sidney Thomas in 1878), Martin steel or Siemens-Martin steel (invented by Pierre-Émile Martin in 1864) and electrical steel, produced by melting raw steel in electric furnaces, resulting in carbon or alloy high-grade structural steels. The microstructure of the steel can be examined on a smoothly ground and polished sample surface. After possible etching, a fine relief of crystalline components is formed on the surface, which can be examined under a microscope. Figure 6 shows a microphotograph of a cut-out of steels of different carbon content:

- (a) 0,1 % C steel; larger ferrite crystals and smaller dark perlite crystals
- (b) 0,15 % C steel; higher perlite content, less pure steel, refinements stretched in the rolling direction
- (c) steel with 0,2 % C; perlite content noticeably higher than (a), slightly marked row structure
- (d) 0,4 % C steel; large perlite crystals
- (e) 0,2 % C steel; distinct row structure.

Comparison of Figures 6(a) to 6(d) shows how the structure of the steel changes with carbon content. In Figure 6(e), the linear structure characteristic of wrought steel is evident. In addition, wrought steel is characterised by a lower carbon content than mild steel.

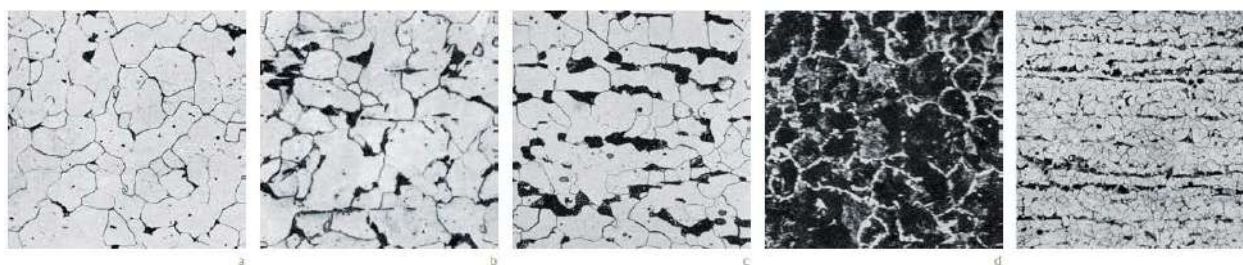


Fig. 6. Microphotographs of the cut of steels of different strengths[20]

Wrought steel was first used for the production of the steel structure. Later, from the beginning of the 20th century, mild steel was used. The first regulation for the design of steel bridges was the “Bridge Regulations of the Ministry of Railways” of 1887. Later, the bridge regulations were replaced by state standards. Until the middle of the 20th century, riveting was used exclusively for joining the individual elements. Riveted steel bridge structures consist of a large number of subtle elements and are therefore airy and have a specific appearance. There are a large number of riveted steel bridges still in use in the Czech Republic. These include railway bridges, road bridges and footbridges. In some cases, these are bridges with large spans over deep valleys that dominate the landscape, e.g. the railway bridge over the Vltava River near Červená, and they have a historic quality, e.g. the Čech Bridge in Prague; or they are large bridges that enhance the views in the city, e.g. the railway bridge below Vyšehrad in Prague. These are mostly riveted truss bridges that have fulfilled their purpose during their lifetime. Where these bridges comply with operational and safety requirements, they need to be properly maintained, i.e. regular maintenance accompanied by specific repairs respecting their heritage value. It is well-known that, under these conditions, steel riveted bridges can serve us for well for over 100 years.

Riveting is the oldest method of joining steel elements, both to create the required cross-section and to assemble the whole bridge structure. The starting material for the manufacture of steel bridge structures is hot-rolled steel material. The basic metallurgical material is sheet metal, wide steel, shaped bars (angles, I-sections and U-sections), or Zorés profiles, gutters and buckle plates (specially shaped sections formerly often used on road and railway bridge decks). The shapes and dimensions of the rolled products were mostly standardised. Rolling mills issued so-called rolling programmes, which contained individual cross-sections, their dimensions and static quantities. Riveting was used from the second half of the 19th century until the end of the 1950s. Today, riveting is used in the repair and reconstruction of old

riveted bridges to preserve the historic value and appearance of the structure. A rivet is a cylindrical body with a ball head or countersunk head on one side (Fig.7). Rivets are made from round profiles by hot pressing. Rivets with diameters $d_l = 20$ and 24 mm were most commonly used for steel bridges. The diameter of the rivet hole (drilling) is 1 mm larger.

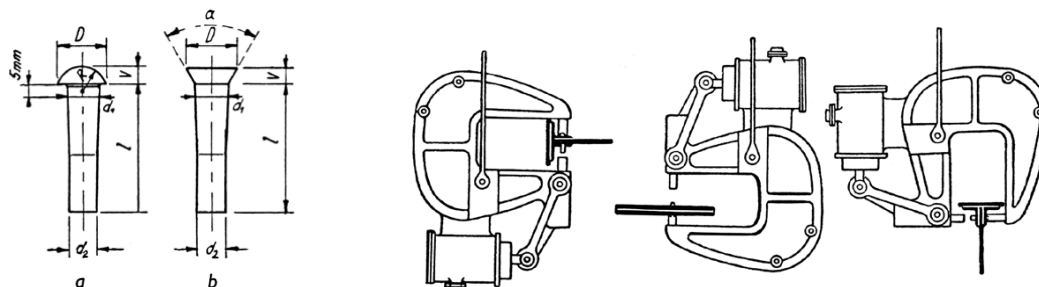


Fig. 7. Ball head rivet and countersunk head rivet[20]

Fig. 8. Different positions of the pneumatic riveting press[20]

Rivets are heated either in a coke oven or by electrical resistance. Rivets heated to white heat, i.e. to a temperature of 1000 to 1100°C , are inserted into the rivet hole. First the shank is pushed into the hole to ensure a tight fit, then the rivet head is forged, either by a pneumatic riveter (Fig. 8) or by hand hammering. When hand-forging, a header is used to form the ball head (Fig. 5). After riveting, the rivet cools and shrinks to tightly clamp the individual sheets that are to be joined. This is why riveting technology is suitable for bridge structures that are exposed to aggressive environmental conditions.

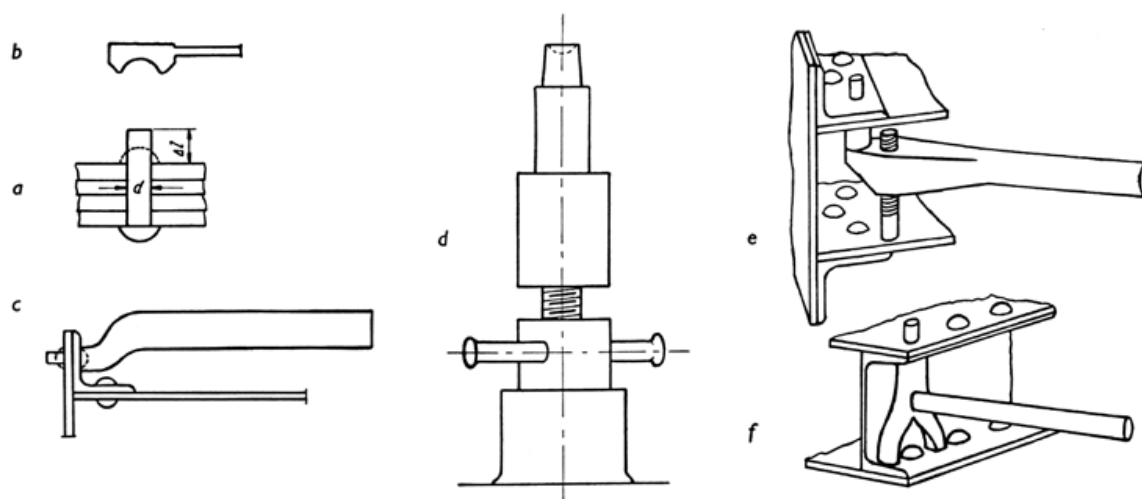


Fig. 9. Riveting and support of the riveting head[20]

Examples of plate riveted girders that have been used either as main girders for small span road and railway bridges, or as crossbeams and stringers for the bridge deck of railway and road bridges, are shown in Fig.10.

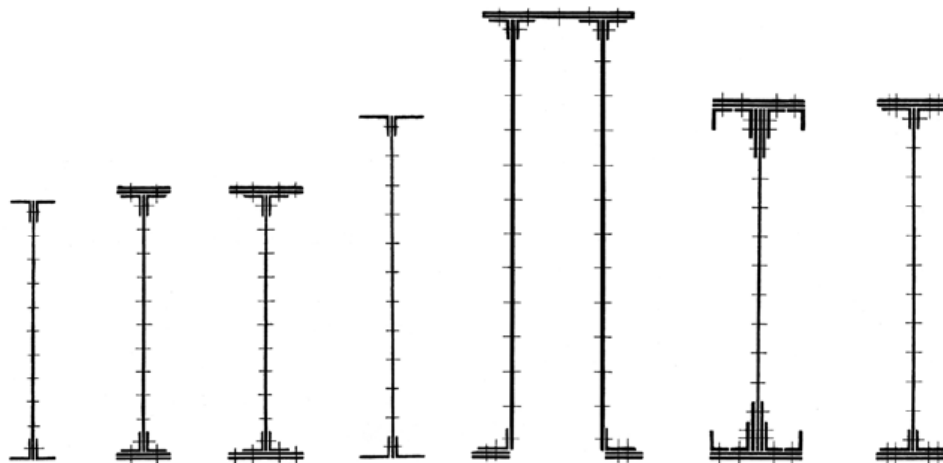


Fig. 10. Plate riveted girders[20]

Main truss girders may have straight chords, or one or both chords may be curved. Examples of simple girder systems with parallel chords are shown in Fig. 11. These are systems that are statically determined. The systems in Figures 11e and 11f were used for bridges with a half-through bridge deck, while the systems in Figures 11g and 11h were used for deck-type bridges.

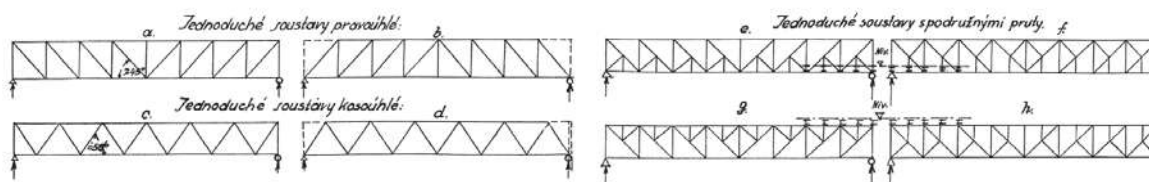


Fig. 11. Simple girder systems with straight chords[20]

Figure 12 shows complex girder systems with parallel flanges. The complex systems are statically indeterminate. For old railway bridges, the most common systems are those shown in Figs. 12n., 12p. and 12s.

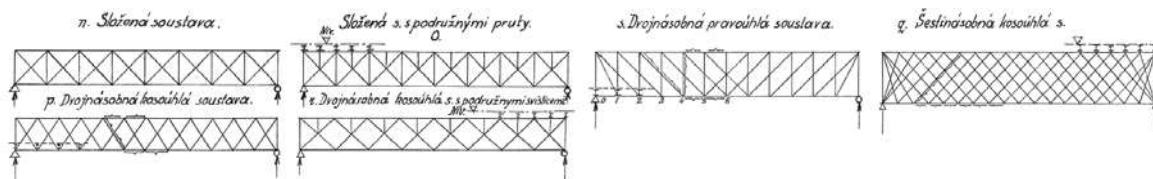


Fig. 12. Complex girder systems with parallel chords[20]

Systems with curved chords were based on the bending moments on a simple beam. In the middle of the span, at the point of the highest bending moment, the beams achieved maximum height. For half-through bridges, a beam with a curved upper chord was predetermined, whereas a beam with a curved lower chord was predetermined for deck-type bridges. Depending on the shape of the curved chord, a semi-parabolic chord (Fig. 13a, Fig. 13a') or a parabolic chord (Fig. 13d, 13d') can be defined.

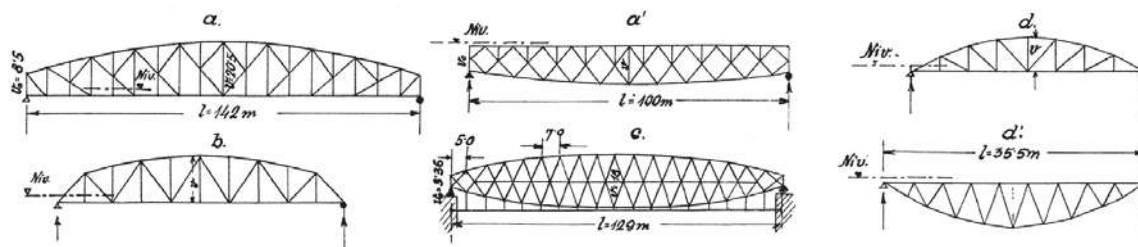


Fig. 13. Girders with curved chord[20]

The chords can be single-walled (Fig. 14) or double-walled (Fig. 15). Single-walled chord members are found in older bridges and are simpler in construction, but are less stiff. Double-walled web members are stiffer and structurally more complex. The lower flange of a double-wall member has a longitudinal gap between the members due to drainage requirements

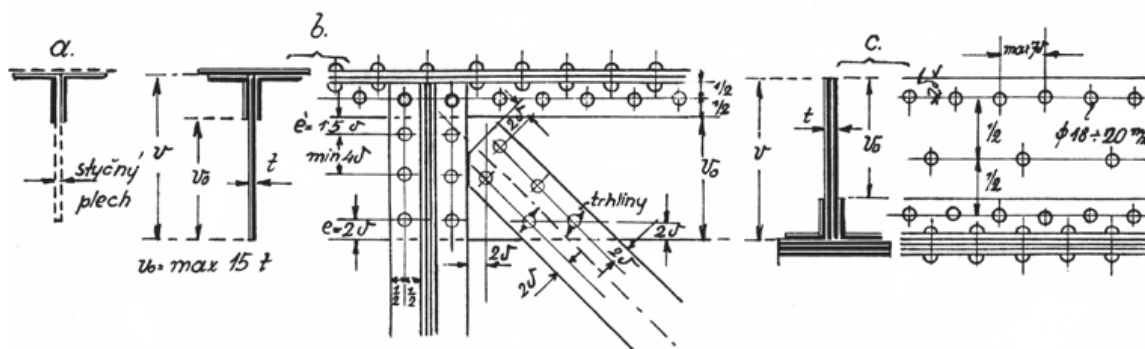


Fig. 14. Single-walled chord members[20]

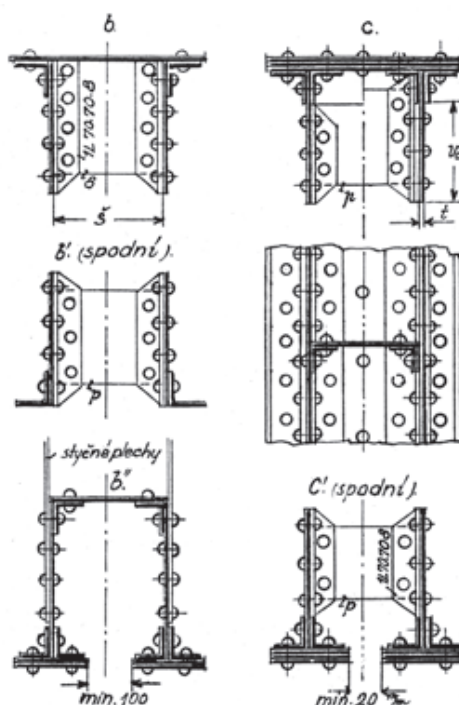


Fig. 15. Double-walled chords[20]

Built-up members composed of two or four angles (L-profiles) may have only a contact plate gap between the members (Fig. 16 and Fig.17). They are then connected to each other by lacings or battens.

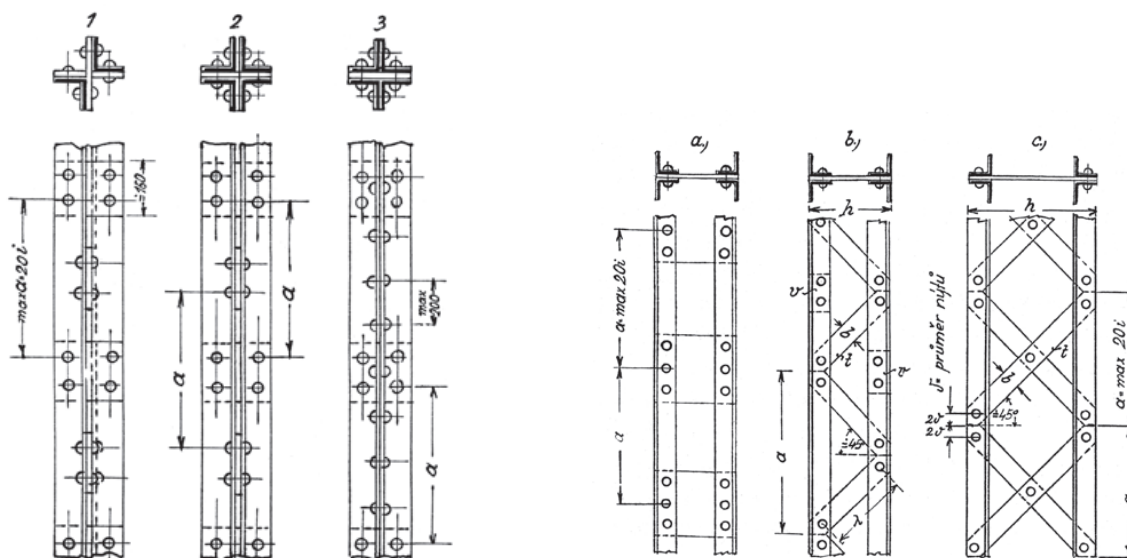


Fig. 16. Built-up members with a gap for the connecting plate[20]

Fig. 17. Built-up members with battens (a) or lacings (b), (c)[20]

Fig. 18 shows an example of a joint with a single-walled lower chord. Figure 19 is a photo of a member joint with a double-walled lower chord. An examination of the structural design of riveted truss bridges shows that there is a large amount of variability in the shapes of the girders and the structural design of the individual members and joints. Riveted truss bridges were characterised by their lightness, but also, on the other hand, by the intensive labour needed for manufacturing and assembly. The structural design of early riveted truss bridges contains many details that are difficult to maintain and to restore corrosion protection in the long term perspective.

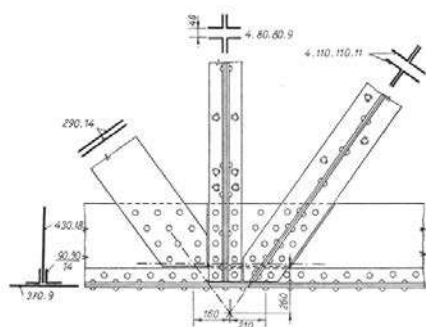


Fig.18. Example of a riveted truss joints[20]



Fig. 19. Truss joint with double-walled lower chord[20]

The oldest steel riveted bridges in Czechia date back to the second half of the 19th century, the period of railway development. These are mainly railway-carrying truss bridges. An impressive example was the Ivančice viaduct dating from 1870, which was a part of the Ferdinand railway on the route Vienna – Hrušovany nad Jevišovkou – Střelice – Brno over the Jihlava river valley (Fig. 20). After more than 100 years of operation of the riveted viaduct, a new parallel welded bridge was built in 1978. The original bridge was dismantled, leaving only one span as a cultural monument.



Fig. 20. Ivančice Viaduct[20]

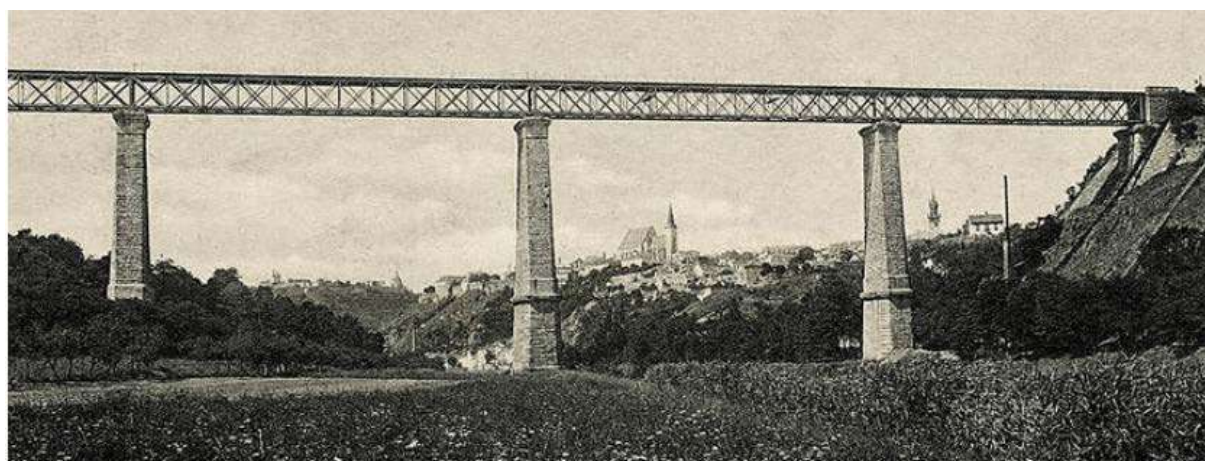


Fig. 21. Znojmo Viaduct[20]

The second large viaduct from that time was the Znojmo Viaduct (1871), which crossed the valley of the river Dyje on the Vienna–Retz–Znojmo line (Fig. 21). This structure served until 1992, when it was replaced by the temporary bridge structure ŽM 16. In 2009, the temporary bridge was replaced by a permanent one.

3. Typology of bridges

Since the dawn of time, bridges have been built on pathways and roads to overcome obstacles. In the Middle Ages, bridges were built with timber, or were arch bridges made of brick and stone. The development of the railways at the end of the first half of the 19th century was a major stimulus for bridge engineering and bridge construction. At that time, it was already possible to use metals for bridge-building purposes. First it was cast iron, and later steel. It was not until the turn of the 19th and 20th centuries that reinforced concrete began to be used for bridge construction (Fig. 22-Fig. 31). Steel bridges are distinguished by the main girders, which create the distinctive look of the bridge and which transfer all the load to the supports. These are **girder bridges** with one or more spans, with plate or truss main girders, with constant or variable girder height, deck-type or half-through and semi-through bridges.



Fig. 22. Plate girder bridge with half-through bridge deck (footbridge in Prague Hostivař from 1937)[20]

Fig. 23. Plate girder deck-type bridge (Křídlovická railway bridge in Brno from 1895)[20]



Fig. 24. Truss half-through bridge with straight chords (railway bridge in Prague under Vyšehrad from 1871)[20]

Fig. 25. Truss bridge with lower bridge deck with upper parabolic chord (road bridge in Prague-Bohdalec from 1915)[20]



Fig. 26. Truss half-through bridge with upper parabolic chord (road bridge to Císařská louka from 1901)[20]

Fig. 27. Truss deck-type bridge with straight chords (railway bridge in Znojmo from 1871 and after reconstruction in 2009)[20]



Fig. 28. A truss semi-through bridge with variable height of main girders (Gerber beam). (Tyrš road bridge over the River Labe in Litoměřice dating from 1910)[20]

Fig. 29. Truss bridge with semi-through bridge deck (Gerber beam) (railway bridge at Červená nad Vltavou from 1889)[20]

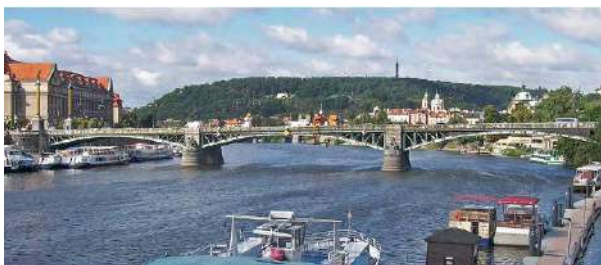


Fig. 30. Arch deck-type bridge. Two-hinged truss arches. (Čechův most in Prague from 1908)[20]

Fig. 31. Arch deck-type bridge. Two-hinged plate arches. (Žďákov Bridge over the Vltava River from 1967)[20]

From the point of view of rail transport, the 1871 railway bridge below Vyšehrad, which connected the Prague railway stations on both banks of the Vltava River, was important. This truss bridge with parallel chords later became an important Prague landmark. However, the single-track bridge gradually ceased to meet the increased traffic demands, and in 1901 it was replaced by a new double-track truss bridge with an upper parabolic chord. Other important railway bridges from the last quarter of the 19th century include Prostřední Žleb, a bridge over the River Labedating from 1874, which was part of the Austrian Northwestern Railway between the stations of Děčín and Prostřední Žleb. Then there is the 1889 bridge over the Orlík reservoir near Červená nad Vltavou, and the bridge over the River Vltava near Skochovice, which dates from 1897. The oldest steel riveted road bridges were built in the Czech Republic at the end of the 19th and the beginning of the 20th century, e.g.: the bridge over the Vltava in Týn nad Vltavou from 1892, the bridge to Císařská louka over the Vltava arm in Prague from 1901, the Čech Bridge over the Vltava in Prague from 1908 and the bridge over the railway tracks in Prague below Bohdalec from 1915. All these examples of steel riveted bridges, both railway and road, are proof of increasing technical sophistication of the Czech lands during the 19th century.

4. Case Study

4.1. Chrást u Plzně – railway bridge

Railway bridge over the Klabava valley. Steel semi-through truss bridge with parabolic lower chord. The bridge has three spans of about 35 m (Fig.32). The truss structure is vertical with crossed diagonals. There is a longitudinal horizontal truss bracing at the level of the bottom chord and under the stringers. There is a vertical transverse bracing in each joint. The chords of the truss are single-web, the upper chord is a T-section, and the lower chord is an inverted T-section. The crossbeams are trussed and connected to the verticals of the main beam. The stringers are I-sections and are recessed into the crossbeams. All members and connections are riveted. The superstructure is supported by steel bearings, the movable bearings have 5 cylinders. The original bridge (of Schifkorn-type) was replaced by the present steel structure in 1892.



Fig. 32. Railway bridge over the Klabava valley. Steel semi-through truss bridge with parabolic lower chord from 1892[20]

4.2. Brno – railway truss arch bridge over Hybešova street

The railway bridge over Hybešova Street has one span of 15.2 m (Fig.33). The bridge consists of four structures lying side by side. It is an arch deck-type bridge. The main girder has 11 sections, the outermost three spans have a trussed vertical structure, and the five sections in the middle have a plated structure. The number of main girders varies from 5 to 6, according to the width of the bridge structure. The distance between the girders is 2.80 m. All the trusses are the same. The height of the girder is 2.70 m in the bearing, 0.50 m in the middle of the span. The main beams are riveted single web. The upper chord has a T cross-section, the lower chord an inverted T cross-section. The diagonals are of segmented members with frame connections. The crossbeams are at an axial distance of 1.5 m, and are plated riveted of cross-section I. In one case, the crossbeams are trussed. The curved metal plates (called “puklovka”) forming the bridge deck are placed on the crossbeams. The transverse truss bracings are located between the three end verticals. For two structures, there is longitudinal horizontal bracing at the upper chords of the crossbeams. The bearing of the superstructures is transversely movable on steel table bearings. The substructure is made of stone masonry. The bridge was constructed in 1897.



Fig. 33. The railway bridge over Hybešova Street with one span of 15.2 m, from 1897[20]

4.3. Strachovice – railway truss girder railway bridge

The railway bridge over Javornice brook near the village of Čistá, an integral part of the local railway Rakovník–Mladotice, has three spans of 12.5+57.0+12.5 m (Fig.34). In the outermost spans there are stone arch structures. In the mid-span there is a steel beam truss bridge with a lower parabolic chord. The 12-span truss is of the Howe truss-type. The axial distance of the main girders is 3.6 m. The upper chords are of a Π - section, the lower chords are of two inverted T-sections. The verticals and diagonals are made of segmental members with truss connectors. The bridge has two levels of horizontal bracing at the upper and lower chords. There is a transverse truss stiffening at each vertical. The crossbeams are riveted plate girders, the stringers are welded plate girders and are mounted on top of the crossbeams. The stringers have vertical bracing placed in the mid-span. The superstructure is supported by steel fixed and roller bearings. The substructure is from stone masonry. The bridge was built in 1898 and underwent reconstruction in 2010.



Fig. 34. The railway bridge near the village of Čistá; in the mid-span there is a steel beam truss bridge with a lower parabolic chord, span 57.0 m, from 1898 [20]

4.4. České Budějovice- „Golden bridge “ - road arch bridge

A road bridge over the River Malše with one span of approximately 40 m (Fig.35). The main load-bearing structure is a steel-tied arch. Currently it serves as a footbridge. There is a paved roadway between the arch-beams. Outside the tied arch, there are cantilevered sidewalks with a bridge deck made of timber. The main beam, upper arch chord, and vertical ties are riveted I-sections. There are three vertical frame portals in the mid-spans. The bridge deck consists of riveted I-section crossbeams, rolled I-section stringers and a bridge deck slab supported by trapezoidal plates. The bridge is supported by steel bearings. The bridge acquired its name during the Middle Ages, when a bridge stood in front of the Rožnov Gate, which was used by merchants when entering the town. The current bridge dates from 1915 and was built by the Prášil Brothers company of Prague-Libeň. The bridge was repaired in 2001.



Fig. 35. A road bridge over the River Malše with one span of approximately 40 m., from 1915[20]

4.5. Ostrava – road arch bridge of Miloš Sýkora

The Miloš Sýkora Bridge over the River Ostravice in Ostrava is a three-span bridge of the Gerber beam-type, featuring a load-bearing middle span and two suspended approach spans with span lengths of 16+60+16 m (Fig.36). The main span consists of a semi-through truss arch and lateral bracing at the level of the top chords. The approach spans are quadrangular trusses of a constant height with verticals. The bridge deck is formed by crossbeams, stringers, and a slab with a steel plate of lost formwork. The structure is riveted, with bolted

and embedded hinges. On the site of the current bridge, there used to be a timber structure, which was built in the 1830s. Due to its poor condition, it was replaced after eleven years by a chain bridge, manufactured by the Klein Brothers' Ironworks in Kladno. Even this bridge did not last very long. After 30 years, it collapsed on the 15th of September 1886, when a squadron of Hulan cavalry troops attempted to cross the bridge. Less than a year later, in 1887, a new bridge was in place, but its condition rapidly deteriorated too. It was replaced by the present bridge in 1914. The bridge was named after Miloš Sýkora, the legendary 24-year-old resistance fighter, who died while preventing the Nazis from blowing up the structure, thus enabling the Red Army's rapid advance during the battle for Ostrava.



Fig. 36. A road bridge over the River Ostravice with a three-span bridge of the Gerber beam-type with span lengths of 16+60+16 m, from 1914[20]

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EDUCATION OF STRUCTURAL ENGINEERS AND CULTURAL HERITAGE

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Abstract: *Education of structural engineers at Faculty of Civil Engineering, Slovak University of Technology in Bratislava. Education includes not only terminology, completion of structural calculations and drawings but also environmental issues, reconstruction, rehabilitation and strengthening of buildings and other structures. Special focus is devoted to preservation of various types of cultural heritage. The paper deals primary with education of foreign and Slovak students in the frame of subject English language focused on several aspects of cultural heritage.*

Key words: *cultural heritage, education of structural engineers, preservation*

1. Introduction

Department of Languages, Faculty of Civil Engineering, Slovak University of Technology in Bratislava provides teaching of English and German language in Bachelor study within the first four semesters [1, 2, 3]. The courses are primary focused on various terminologies of 20 different departments including Department of Architecture; Department of Building Construction and Department of Building Technology. During every of 4 semesters the terminologies of 5 departments are analysed. It is extremely difficult for students to understand meaning of special terms, which are not in the basic dictionaries and applied them in solving their task also in other subjects. A teacher may have altogether up to 200 students in semester. The participants of the courses consist of Slovak and foreign regular students studying normally 5 years and also Erasmus foreign students studying in Slovakia only 1-2 semesters. Foreign students are from various countries, e.g. from: Brazil, Greece, Hungary, Kazakhstan, Kenya, Kuwait, Lithuania, Mali, Palestine, Poland, Russia, Spain, Syria, UK, etc.

For the students it is important to understand the design of structures according to set of Eurocodes EN 1990-EN1999 and accompanying Technical Specifications and Technical Reports, e.g. Technical Specification CEN/TS 17440:2020 Assessment of Existing Structures [4]. The relevant ISO standards [5] and the environmental issues [6] are also important. Big importance has knowledge of students about cultural heritage and its preservation.

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2. Education of Structural Engineers

Why does a student want to become a professional engineer? The answer may begin on a personal level (money, power, prestige) or from a more service-oriented mindset (create a better world for future generations). To the latter reason may help also better knowledge about civil engineering history and cultural heritage. To be trained to think like an engineer is also an important aspect of education. An engineer must have the ability to retain knowledge through an association with some sort of practical purpose. It is far easier for an engineer to recall an important bit of knowledge when it has an application. Before an application can be linked to factual information, useable data must be properly understood. Sometimes a dictionary or thesaurus might need to be consulted if there is a question as to meaning [7].

Henrik Ibsen first said "A thousand words leave not the same deep impression as does a single deed." This quote was plagiarized and para-phrased several times. "One Picture Worth Ten Thousand Words" is Chinese proverb, which became popularly when attributed to Confucius. Computer scientist Edsger Dijkstra once remarked, "A picture may be worth a thousand words, a formula is worth a thousand pictures," which may be interesting for structural engineers.

That is why I prepare for students the following tables to help them easier understand engineering terminology.

Table 1. Equivalent terms in seven languages: English, German Spanish, Italian, French, Slovak and Czech language. English, Spanish and Slovak languages contains also definitions of term used Eurocodes.

English [German]	Spanish [Italian] (French)	Slovak [Czech]
construction works [Bauwerk] everything that is constructed or results from construction operations. NOTE. – This definition accords with ISO 6707-1. The term covers both building and civil engineering works. It refers to the complete construction works comprising structural, non-structural and geotechnical elements.	obras de construcción <i>alebo</i> construcción [costruzione], (construction) todo lo que es construido o resultado de operaciones de construcción. NOTA . – Esta definición está de acuerdo con la Norma ISO 6707-1. El término abarca tanto los edificios como las obras de ingeniería civil. Se refiere a los trabajos de construcción completos comprendiendo elementos estructurales, no estructurales y geotécnicos.	stavba [stavební objekt] všetko, čo je postavené alebo je výsledkom stavebnej činnosti POZNÁMKA. – Táto definícia je v súlade s ISO 677-1. Termín zahŕňa budovy a inžinierske stavby. Vztahuje sa na kompletnú stavbu vrátane nosných, nenosných a geotechnických prvkov.
Execution [(Bau-) Ausführung] all activities carried out for the physical completion of the work including procurement, the inspection and documentation thereof NOTE. – The term covers work on site; it may also signify the fabrication of components off site and their subsequent erection on site.	Ejecución [esecuzione] (exécution) todas las actividades llevadas a cabo para completar físicamente la obra, incluidas las tareas administrativas, la inspección y la documentación derivadas de las mismas. NOTA . – El término comprende el trabajo in situ; puede también significar la fabricación de componentes	zhotovovanie [provádění] všetky činnosti, vykonávané na fyzické dokončenie stavby vrátane dodávok, kontroly a príslušnej dokumentácie POZNÁMKA. – Tento názov zahŕňa práce na stavenisku; môže znamenať aj výrobu prvkov mimo staveniska a ich následné zmontovanie na stavenisku.

	de la obra y su posterior colocación en la obra.	NÁRODNÁ POZNÁMKA. – Termín „construction“ sa môže v angličtine používať v slovných kombináciách, ktoré nie sú dvojznačné (napríklad „during construction“ - počas zhotovovania).
Structure [Tragwerk] organised combination of connected parts designed to carry loads and provide adequate rigidity	Estructura [struttura] (structure) organización combinada de partes conectadas proyectadas para soportar cargas y suministrar la rigidez adecuada	konštrukcia [konstrukce] usporiadaná zostava spojených prvkov navrhnutá tak, aby prenášala zaťaženia a mala primeranú tuhosť NÁRODNÁ POZNÁMKA. – Anglický a francúzsky názov „structure“ označuje v eurokódach nosnú časť
type of building or civil engineering works [Art des Bauwerks] type of construction works designating its intended purpose, e.g. dwelling house, retaining wall, industrial building, road bridge	tipo de edificio o de obras de ingeniería civil alebo naturaleza de la construcción [tipo di costruzione] (nature de construction) tipos de obras de construcción con designación de su función prevista, por ejemplo, vivienda, muro de contención, edificio industrial, puente de carretera.	druh budovy alebo inžinierskej stavby [druh pozemního nebo inženýrského objektu] druh stavby označujúci jej zamýšľaný účel, napríklad obytný dom, oporný múr, priemyselná budova, cestný most
form of structure [Art des Tragwerks] arrangement of structural members NOTE. – Forms of structure are, for example, frames, suspension bridges.	forma de estructura alebo tipo de estructura [tipo di struttura] (type de structure) disposición de los elementos estructurales. NOTA. – Una forma de estructura es, por ejemplo, un armazón, puentes suspendidos	typ konštrukcie [typ konstrukce] usporiadanie konštrukčných prvkov POZNÁMKA. – Typy konštrukcie sú napríklad rámy, visuté mosty.
construction material [Baustoff] [Werkstoff] material used in construction work, e.g. concrete, steel, timber, masonry	material de construcción [material da costruzione] (matériau de construction) material utilizado en obras de construcción, por ejemplo, hormigón, acero, madera, fábrica de albañilería.	stavebný materiál [stavební materiál] materiál použitý na stavbu, napríklad betón, oceľ, drevo, murivo
type of construction [Bauweise] indication of the principal structural material, e.g. reinforced concrete construction, steel construction, timber construction, masonry construction, steel and concrete composite construction	tipo de construcción alebo modo de construcción [sistema costruttivo] (mode de construction) indicación del material estructural principal, por ejemplo, construcción de hormigón armado, construcción de acero, construcción de madera, construcción de fábrica de	druh stavby [druh konstrukce] označenie stavby podľa hlavného stavebného materiálu, napríklad železobetónová, oceľová, drevená, murovaná, spriahnutá oceľovo-betónová stavba

	albañilería, construcción mixta de acero y hormigón.	
method of construction [Bauverfahren] manner in which the execution will be carried out, <i>e.g.</i> cast in place, prefabricated, cantilevered	método constructivo alebo procedimiento de ejecución [procedimento executivo] (proceed d'exécution) manera en que se va a llevar a cabo la ejecución, por ejemplo, hormigonada in situ, prefabricada, en ménsula.	spôsob výstavby [stavební postup] spôsob, akým sa stavba zhotoví, napríklad betonážou na stavenisku, montážou z prefabrikátov, letmou betonážou
structural system [Tragsystem] load-bearing members of a building or civil engineering works and the way in which these members function together	sistema estructural [sistema strutturale] (système structural) elementos portantes de un edificio o una obra de ingeniería civil y la forma en que estos elementos funcionan juntos.	nosný systém [nosná soustava] nosné prvky budovy alebo inžinierskej stavby a spôsob, akým tieto prvky spolupôsobia

ISO 6707-1:2017 [8] contains only definitions of English terms. The author added Slovak and Spanish terms accompanied by photographs. The short samples which were prepared for the students are shown in the following Tables 2-4. The complete set of special terms, but without Spanish equivalents was published in [9].

Table 2.

stavba	construction works	US: construction
všetko, čo sa postavilo alebo je výsledkom stavebnej činnosti	everything that is constructed or results from construction operations	
	obras de construcción	
	todo lo que se construye o resulta de operaciones (tarefas, trabajos) de construcción	
		

Table 3.

inžinierska stavba	civil engineering works	US: civil engineering project
stavba znamenajúca konštrukciu, akou je priehrada, most, pozemná komunikácia, železnica, vzletová a pristávacia dráha, inžinierska sieť, potrubie alebo stoková sieť, alebo je to výsledok činností, akými sú bagrovanie, zemné práce, geotechnické procesy, ale nepatrí sem pozemná stavba a s ňou súvisiaca stavebná činnosť na stavenisku	construction works comprising a structure, such as a dam, bridge, road, railway, runway, utilities, pipeline, or sewerage system, or the result of operations such as dredging, earthwork, geotechnical processes, but excluding a building and its associated site works	

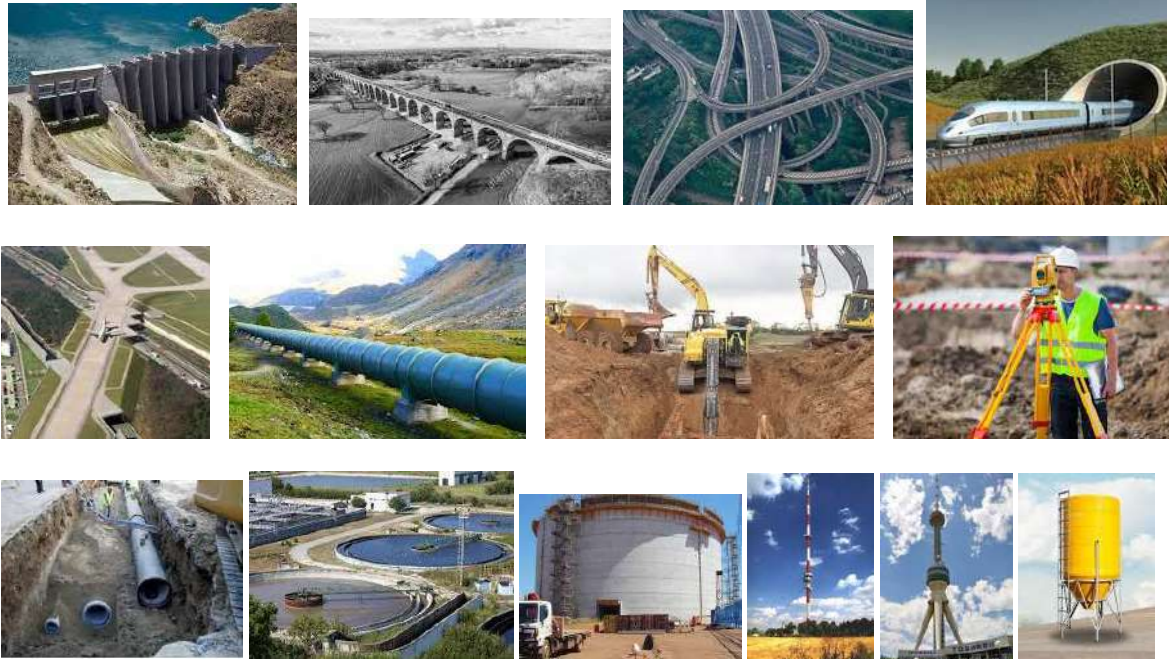
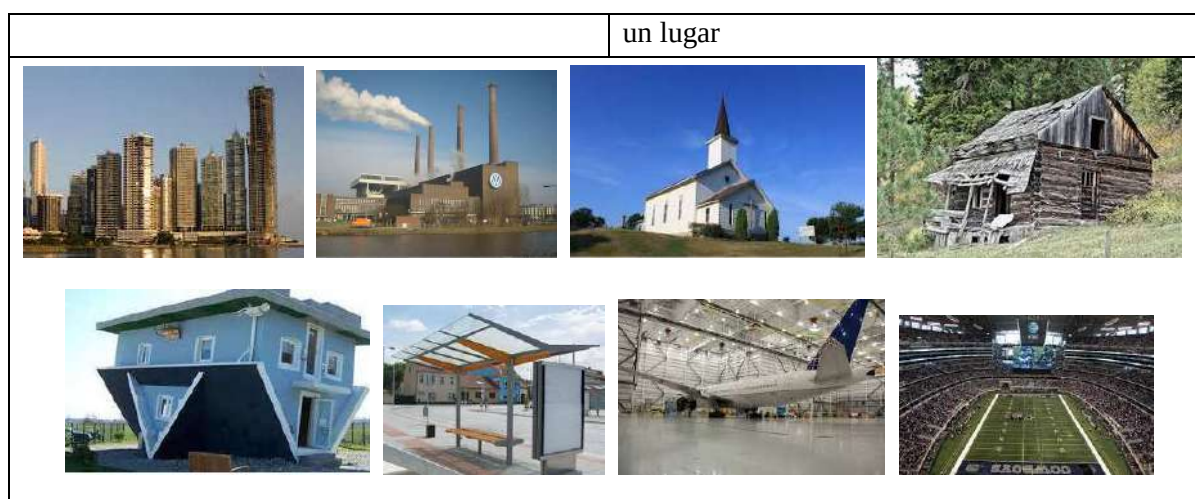
POZNÁMKA: Súvisiaca stavebná činnosť na stavenisku patrí v USA k inžinierskym stavbám	NOTE: Associated siteworks are included in US civil engineering projects
	obras de ingeniería civil
	obras de construcción que comprenden (incluyen) una estructura, como una presa, puente, carretera, vía férrea, pista (autopista), servicios públicos, tubería o sistema de alcantarillado, o el resultado de operaciones (tareas, trabajos) como dragado, movimiento de tierras, procesos geotécnicos, pero excluyendo un edificio y sus obras asociadas en el sitio
	
POZNÁMKA: patrí sem aj kanalizácia (sewerage), čistiareň odpadových vôd (sewage treatment plant), nádrž (tank), kotvený stožiar (guyed mast), tower (veža), silo (silo), chimney (komín), etc.	

Table 4.

pozemná stavba, budova	building
stavba, ktorá má prístrešok pre jej obyvateľov alebo obsah v nej, čo je jedným z jej hlavných účelov; obyčajne je čiastočne alebo úplne uzatvorená a navrhuje sa tak, aby trvale stála na jednom mieste	construction works that has the provision of shelter for its occupants or contents as one of its main purposes; usually partially or totally enclosed and designed to stand permanently in one place
porovnaj: výstavba	cf. building
	obras de edificación
	las obras de construcción que tengan como uno de sus fines principales la provisión de alojamiento para sus ocupantes o contenidos; por lo general parcial o totalmente cerrado y diseñado para permanecer permanentemente en



The curiosity and creativity are two psychological tools which may substantially increase the interest of many students. That is why I asked them to prepare presentations about various types of cultural heritage based on their own investigations of publications including daily press. Some results of their investigation are given in the following paragraph.

3. Cultural heritage in Slovakia

The photographs by Czechoslovak photographer Karol Plicka surely belong to Slovak cultural heritage. His work was topic of the master thesis [10].



a)



b)

Fig. 2. a) Ruin of Lietava castle; b) ruin of Spiš castle. (Karol Plicka, 1925 [11]).



a)



b)

Fig. 3. a) Zemplín; b) Hruštín. (Karol Plicka, postcards 1925-27).



a)



b)

Fig. 4. a) Horný Tisovník; b) Goral house in Polhora. (Karol Plicka, postcards 1925-27).

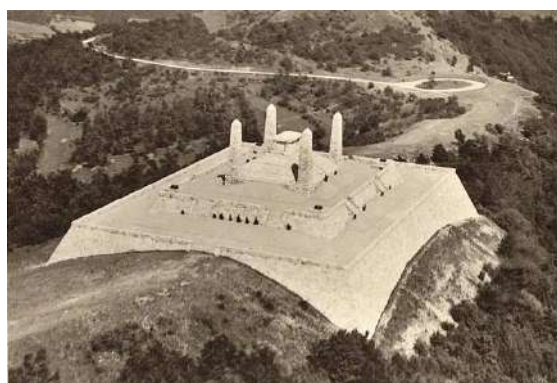


a)

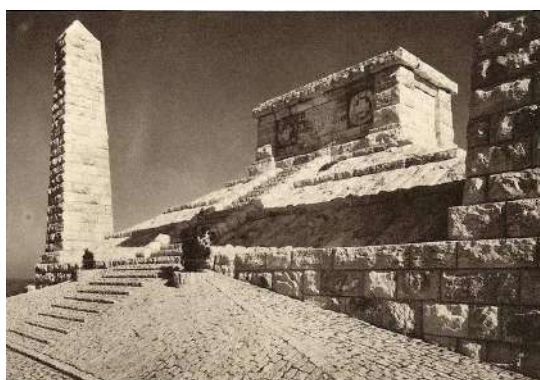


b)

Fig. 5. a) Women in the front of church. (Karol Plicka, 1925 [11]); b) Timber church in Zboj. (Karol Plicka, postcard 1925-27).



a)



b)

Fig. 6. a) Bradlo. Memorial of Gen. M. R. Štefánik; b) Detail of memorial (Karol Plicka, 1925 [11]).

Bradlo – Memorial of Gen. M. R. Štefánik was designed by Dušan Samo Jurkovič (23. August 1868, Turá Lúka – 21. December 1947, Bratislava). He was a Slovak architect, furniture designer, artist and ethnographer. One of the best-known promoters of Slovak art in 20th century Czechoslovakia. He is remembered mostly due to his projects of numerous World War I cemeteries in Galicia and thanks to his wooden works of spa complex in Luhačovice and mountain cottage hotel Maměnka and canteen Libušín in Pustevny. Thanks to his artistic work with wood, he is referred to as "the poet of timber". His architectonic style was a unique fusion of folk architecture and then-popular architectonic styles, mostly associated with Art Nouveau. Jurkovič repeatedly stressed: "The work of art is rooted in the time. I also have always cautiously listened to its voice."



a)



b)

Fig. 7. Libušín in Pustevny Ski Resort, Beskydy Mountains, Czech Republic. (1897-99).
Design: Dušan Jurkovič.



a)



b)

Fig. 8. Hotel Maměnka, Pustevny, Beskydy Mountains, Czech Republic. (1897-99).
Design: Dušan Jurkovič.



a)



b)

Fig. 9. a) Spa house in Luhačovice, Czech Republic (1902); b) bridge by Jurkovič. Nové Město nad Metují, Czech Republic (1910). Design: Dušan Jurkovič.

Dušan Jurkovič was the first to whom the title National artist (1946) was awarded. He designed much more structures as they are mentioned above and his work would surely deserve the books not a part of short paper.



a)



b)

Fig. 10. a) Alois Jirásek cottage in Dobrošov, Náchod, Czech Republic (1923); b) Culture house in Skalica, Slovak Republic (1905). Design: Dušan Jurkovič.



a)



b)



Fig. 11. a) Calvary at Bardejov. Beginning of building 1863, finished in 1868, sainted in 1869 (courtesy of bardejov.sk, J. Ličková, Korzár Prešov, K. Gécziová); b) Dron photography (courtesy of Filip Liskay).

Calvary (Figure 11) is located not far from Bardejov centre. It consists from the church Exaltation of Holy Cross and 14 chapels. Twice a year the calvary is the site of large regional pilgrimages. Calvary lacks an adequate road, new benches and lighting. The chapels also need renovation. It needs urgently renovation. However, after years there is a plan to improve this place.



Fig. 12. Church of the Seven Sorrowful Virgin Mary in Horná Lehota built with his own help in the era of the toughest communist normalization (courtesy of MY Orava, Dorota Mikulášová, source: LP).

300-year-old Church of the Holy Trinity on the hill above the village Horná Lehota was too small for 600 inhabitants. That is why the new church (Figure 12) was built in 1969–1972 with money donated by emigrants to the USA. In the interior there is a Pieta statue by A. Grom.



Fig. 13. Classicist mansion in Krásna near Košice (courtesy Korzár / Judita Čermáková).

Thanks to the European Capital of Culture 2013 project, about 20 objects in Košice have been improved, reconstructed and made operational. One of the most beautiful Slovak classicist mansions in Krásna (Figure 13) was reconstructed from ruins for 1,2 million euro.



Fig. 14. Secondary Industrial School of Mechanical Engineering in Košice from 1872. (courtesy Korzár Košice / Katarína Gécziová, source: FB / SPŠ Strojnícka).

This year, the school (Figure 14) is celebrating 150 years since its establishment, which makes it one of the oldest of its kind in Central Europe. At the same time, it was the first ever industrial school in Austria-Hungary. Teaching in it began on October 9, 1872 in the form of a three-year Higher Engineering School. At first, 19 students sat in its desks, during the first school year this number increased to 31. Two years later, it already had three classes and its own professors. In the six months after the exams, three were dismissed for failure, 21 students successfully completed the first year, four failed and three attended the school as guests.



Fig. 14. Istropolis, the House of Trade Unions on the Trnava Mýto in Bratislava

The famous Slovak architects Ferdinand Konček (1929), Il'ja Skoček (1930-2022), Ľubomír Titl (1929-2018) won in 1956 competition and designed the building of the House of Trade Unions which was opened in 1981. The building of the House of Trade Unions on the Trnava Mýto in Bratislava, also known as Istropolis, was built in the 1980s. It served as a conference palace for the leaders of the communist regime. After 1989, it was the property of the trade unions. The municipality tried to buy it, but no agreement was reached. The union sold Istropolis to a private investor in 2017. After longer resistance and hesitation the municipality decided in January 2022 that Istropolis may be removed. Developer plans to demolish the building and build the New Istropolis project there. Titl and Skoček died without knowing that their work will be destroyed.



Fig. 14. New Istropolis in Bratislava (visualization: developer Immocap).

Not only building and civil engineering structures belong to the national heritage. Karel Gott (14 July 1939 – 1 October 2019) was a Czech singer, considered the most successful male singer in Czechoslovakia and the Czech Republic. He was voted the country's best male singer in the annual Český slavík (Czech Nightingale) national music award 42 times, most recently in 2017. Over the course of his career he released over 100 albums and 100 compilation albums, and sold an estimated 50–100 million records worldwide, 23 million of them in the German-speaking market.

Miroslav "Miro" Žbirka (21 October 1952 – 10 November 2021), called Meky, was a Slovak pop and rock singer and songwriter, widely popular in 1980s Czechoslovakia. Born in Bratislava to a Slovak father and an English mother, he sang in Slovak, English, and Czech. He sometimes recorded in London, but lived in Slovakia and since early 1990s in Prague, Czech Republic, where he died. Together with the artist's Czech wife Kateřina, the National Museum in Prague decided to commemorate the personality of Miro Žbirka through an exhibition "Miro Žbirka – years and days" in the Historical Building of the National Museum. 28 panels and, above all, a number of important and at first glance insignificant things will present the singer's life and work not in dry data, but in stories, not in nostalgic memories, but in celebration of musical enthusiasm, talent and empathy.

Conclusion

Education of structural engineers and importance of cultural heritage in process of teaching them. Experiences of members of Department of Languages at Faculty of Civil Engineering, Slovak University of Technology in Bratislava, Slovakia.

Acknowledgement

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RELIGIOUS CULTURAL HERITAGE IN THE SLOVAK REPUBLIC IN THE EDUCATION OF CIVIL ENGINEERING STUDENTS

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Abstract: *Wooden and brick, Gothic and Baroque sacred buildings and monuments in eastern Slovakia. Eleven churches including the highest one with details of the top of their towers. The highest Slovak church (87 m) in Spišská Nová Ves compared with the second highest one (85 m) in Bratislava. Twelve postage stamps Cultural heritage in Slovakia issued in 2nd January 2009 and postage stamp 1150th anniversary of the recognition of the Slavic liturgical language issued together with Bulgaria, the Czech Republic and Vatican City State. Importance of topic Cultural heritage in the education of civil engineers.*

Key words: *cultural heritage, education of structural engineers, postage stamps*

1. Introduction

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 (677 km), Poland (444 km), Czech Republic (252 km), Ukraine (97 km), and Austria (91 km). The total area of Slovakia is 49 035 km². The population is about 5,447 million people. Slovakia is a member state of the United Nations (1993), European Union (2004), and the European Economic and Monetary Union (2009), among others. The official language is Slovak, very similar to the Czech language, both members of the Indo-European Slavic language family.

Three localities from Slovakia were inscribed to the UNESCO World Heritage List already in 1993: Castle of Spiš and its environs, Banská Štiavnica and Vlkolínec. In 2000, the historic town Bardejov was added, in 2008 wooden churches of the Slovak part of Carpathian Mountain Area and in 2009 town Levoča. The paper presents only small number of churches, chapels and rotundas belonging to cultural heritage of Slovakia.

Educators at Faculty of Civil Engineering, Slovak University of Technology in Bratislava take part in investigation, assessment, reconstruction and strengthening of historical structures including those belonging to cultural heritage. Some bachelor, master and PhD final thesis are devoted to such problems too.

Information about existing cultural heritage in Slovakia is important part of education. Slovak and foreign students are informed about it also in the frame of subject English language by members of Department of Languages.

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2. Towers of eleven churches in eastern Slovakia plus St. Martin's Cathedral



Fig. 1. Dome of St. Elizabeth in Košice is the easternmost Gothic cathedral in Europe. It has an area of 1200 square meters and a capacity of more than 5000 people. They started building it at the end of the 14th century on the site of the original parish church, while at that time there were also Dominican and Franciscan churches in the town.
(Courtesy of TASR).



Fig. 2. The towers are part of the Church of St. Cross on the Calvary in Prešov. The emergence of the Baroque complex is linked to the work of the Jesuits and re-Catholicization. The church on top of the hill is 253 years old and some chapels were built in the 19th century. The oldest chapel of Christ on the Mount of Olives was already consecrated with a cross in 1721. (Courtesy of Korzár/Michal Frank).



Fig. 3. The late-Renaissance watchtower is a dominant feature of the historical center of Rožňava, old mining town. It was built in the years 1643-1654, that is, in the period when the country was threatened by the Turks and it performed an important defense function. The Turkish cannon ball can still be seen in its southern facade. (Courtesy of TASR).



Fig. 4. The pediment of the Roman Catholic Church of the Assumption of the Virgin Mary in Spišská Nová Ves is 87 meters high, and thanks to this, it is entered in the Book of Slovak Records. In the tower there is the highest bell tower on Spiš and there are 7 clock-faces on it. According to the legend, the "blind Spiš people" could not see the clock placed higher, so they added another ones a little lower.



Fig. 5. The second highest church in Slovakia: the St Martin's Cathedral in Bratislava. Compare it with Fig.2. The tower is 85 m (279 ft) high and at one time was part of the medieval city fortifications. The cathedral is constructed in the traditional crucifix shape. It is the largest and one of the oldest churches in Bratislava, known especially for being the coronation church of the Kingdom of Hungary between 1563 and 1830. The tower is capped by a replica of the Hungarian Crown resting on a gilded pillow.
(Courtesy of Wikipedia).



Fig. 6. Church of St. Stephen the King from the end of the 18th century stands near Domaša, a multipurpose water works - a water reservoir.
(Courtesy of Matúš Vavrek, KOCR Severovýchod Slovenska).



Fig. 7. In the second half of the 19th century, the Evangelicals from Kežmarok, former king town, decided to build a new church. That is why the delegation in 1870 chose the Danish civil engineer Teofil von Hansen, at that time the chief landscape architect in Vienna, who presented his own projects and those - as tradition says - were originally intended for the Orient. Construction lasted 22 years with an 11-year break. The height of the tower is 42 meters and the height of the dome is 33 meters. (Courtesy of TASR).



Fig. 8. The Church of the Holy Spirit in Medzilaborce rises on a hill above the city and inside hides a feast for the eyes. This majestic Orthodox temple was built in 1949 in honor of fallen soldiers in both world wars. It stands on the site of an old wooden church and is also a memorial to fallen heroes - Slavs from the First and Second World Wars. (Courtesy of Pravoslávna cirkevná obec Medzilaborce).



Fig. 9. Roman Catholic wooden church of St. Francis of Assisi, made of red spruce, was built in the Gothic style at the end of the 15th century. It is located in the center of Hervartov village in the Bardejov district. Since 2008, it has been included in the list of UNESCO monuments. It is the oldest and one of the best-preserved wooden churches in Slovakia. (Courtesy of J. Štovka).



Fig. 10. The Premonstratensian Monastery with the Church of St. John the Baptist in Jasov is the largest late-Baroque monastic complex in Slovakia. It has symbolic 365 windows, 12 chimneys and four gates. The first mention of the Premonstratensians in Jasov dates from 1255. They started building the complex as we know it today in 1746. (Courtesy of Opatstvo Jasov).



Fig. 11. In the 13th century, the people of Levoča town built a small chapel in the forest above the town, out of gratitude that the Virgin Mary protected them from the Tatars. In 1470, it was rebuilt into a Gothic temple, which was rebuilt several times. The Church of the Visitation of the Virgin Mary is a cruciform building, built according to the project of Anton Müller. The longitudinal nave is 27 meters wide, 16 meters across and 42.5 meters long. The statue of the Virgin Mary on the main altar dates from the first half of the 15th century. The Marian holiday is celebrated there every year by thousands of believers, and the Pope was not missing among them. (Courtesy of Wikipedia/Karelj).



Fig. 12. Minor Basilica of St. Aegidius is the most important monument of the square in Bardejov. The temple took the form of a magnificent Gothic basilica with three naves, it has 11 late Gothic wing altars, and the most valuable is the side altar of the Nativity, which is included in the circle of the Krakow workshop of W. Stwosz. The basilica has stood in the middle of a unique square since the 15th century, and the interior, among other rarities, also contains a sculpture by Master Pavol from Levoča. (Courtesy of Wiki/Ondřej Žvácěk).

2. Slovak Cultural heritage on postage stamps

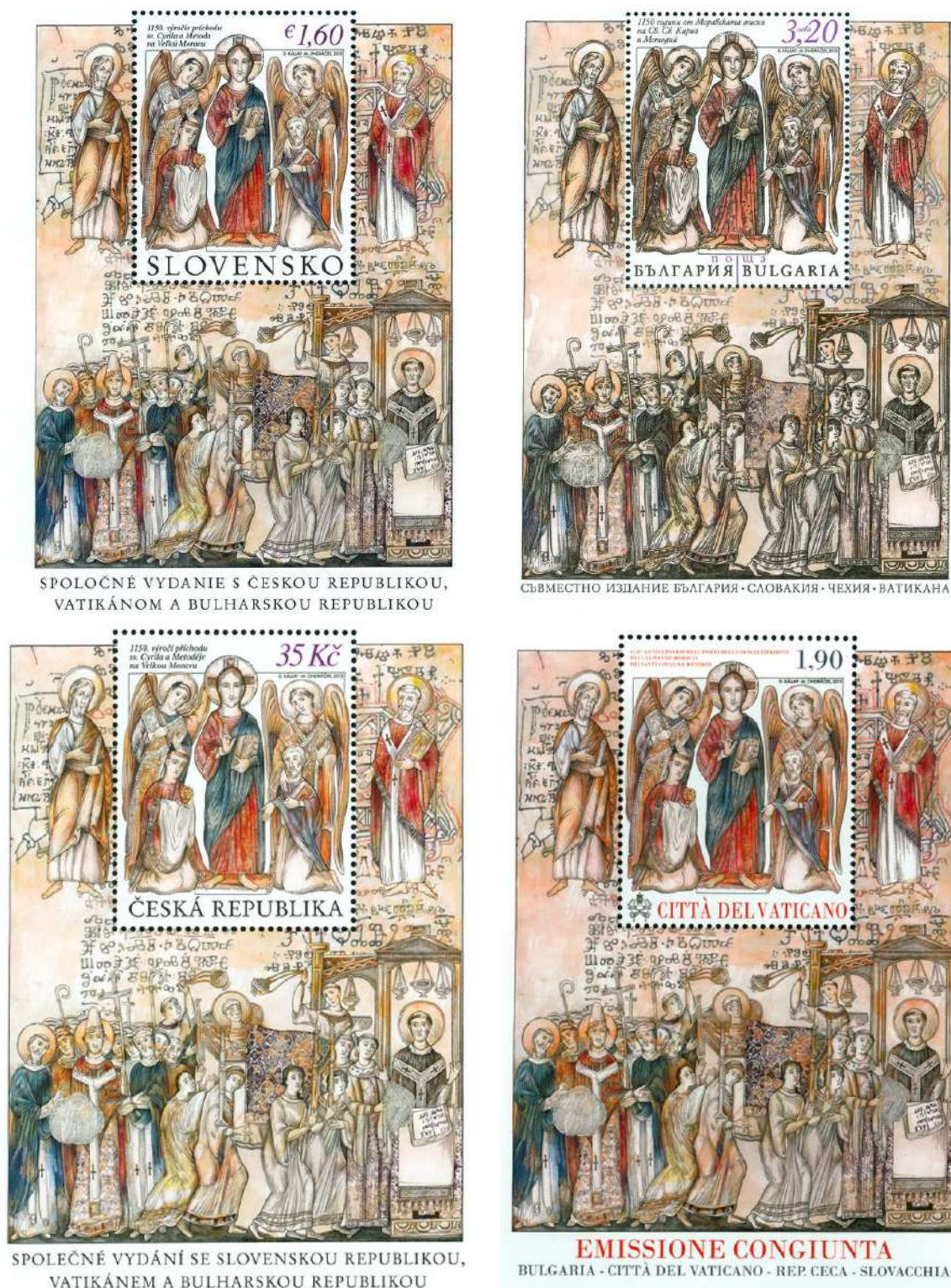


Fig. 13. 1150th anniversary of the recognition of the Slavic liturgical language. For the first time in history Slovak Post was awarded the award for the most beautiful stamp in the world with a Christian theme in the competition "Il Premio Internazionale d'arte Filatélico San Gabriele". The stamp was issued in 2013 as a joint edition with Bulgaria, the Czech Republic and the Vatican City State, on grounds of the initiative of Slovak Post, which provided the art design of the stamp to other countries.

1150th anniversary of the recognition of Old Slavonic as a liturgical language. This act was preceded by the forty-month stay of Constantine and Methodius in Great Moravia, after which they went with their pupils through Pannonia to Venice, then belonging to the Byzantine Empire, and to Constantinople. There they wanted to have their disciples ordained, whom they could not ordain themselves, because they did not yet have the episcopal rank. They were caught in Venice by the invitation of Pope Nicholas I to Rome, so they changed their original intention. Pope Hadrian II received them in Rome, because Pope Nicholas I died in the meantime. Their reception was very dignified because they were bringing the remains of the martyr Clement. The Pope sanctified the Slavic books they brought, approved them and ordained disciples for them. These historical events are depicted on the sheet based on the mosaics preserved on the triumphal arch of the Santa Maria Maggiore church in Rome and an important monument of Slavic literature – the Zograf Codex, named after the Zograf Monastery on Mount Athos. Old Slavic thus became the fourth liturgical language alongside Hebrew, Greek and Latin.



FIG. 14. Postage stamps *Cultural heritage of Slovakia* issued in 2nd January 2009.

0,01€ Chapel of the Saint Margaret of Antioch, near Kopčany, is one of still standing churches for which the Greater Moravian origin is considered. It belongs to the oldest churches in Slovakia. The chapel was built probably in the 9th or 10th century and was first mentioned in 1329.

0,02€ Church of the Assumption of the Blessed Virgin Mary in Boldog. The church was built in the first half of the 12th or maybe in the 11th century. Around 1220, it was extended westwards. After 1280, both the church and the whole local community passed into the possession of the Order of Poor Clares. In the years 1364-1370, they made a gothic reconstruction of the church. In 1605 the temple was damaged during insurgent fights. From 1634, the building belonged to the Calvinists, but it was not used and systematically destroyed. In 1706, the church was once again taken over by the Poor Clares, which in 1713 renovated it. In the mid-nineteenth century, the building received a new roof. In 1976, a professional reconstruction of the church took place, during which Romanesque and Gothic windows and other original architectural elements were discovered.

0,05€ Rotunda of St. Margaret in Šivetice. The rotunda was built in the second quarter of the 13th century, probably in the 1240s, perhaps from the foundation of the then village owners, the Zach family. It is believed that along with the smaller rotunda in nearby Prihradzaný and the non-existent castle under the hill Muteň, it created a defense system, in addition to its original sacred function. In addition to location and visual contact of all three, the narrow windows of the building also show defensive function. Around 1270, the

interior of the rotunda was decorated with Romanesque frescoes. The pressure of the roof on the vault led to the first renovation in the first half of the 14th century. During it, a stone lantern was demolished on the roof of the building. In 1750, next to the rotunda, a baroque belfry was built and the perimeter walls were raised while replacing the roofing. Artistic, architectural and archaeological research took place in the 70's and 80's of the last century. In 2007, the renovation started, thanks to the support of the Ministry of Culture of the Slovak Republic.

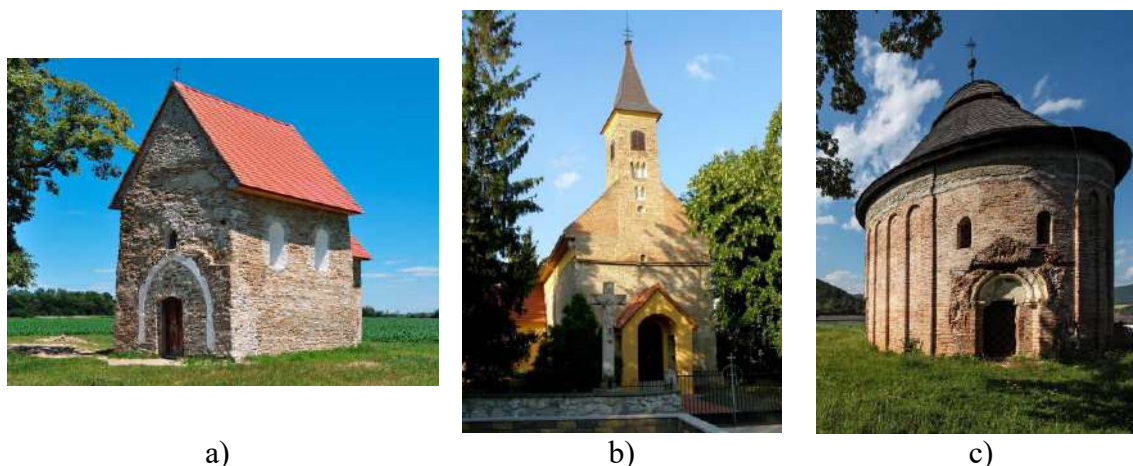


FIG. 15. a) 0,01€ Chapel of the St. Margaret near Kopčany; b) 0,02€ Church of the Assumption of the Blessed Virgin Mary in Boldog; c) 0,05€ Rotunda of St. Margaret in Šivetice, photo Marek Šeregi. (Courtesy of Wikimedia Commons).

0,10€ Church of St. John the Baptist in Sedmerovec-Pominovce. The church was built in the no longer existing village of Pominovce (“villa Pomnen”) in the 12th century. The village disappeared already in the Middle Ages. According to the local tradition, it was caused by the flooding of the Vah River, after which the population moved and founded the settlement Sedmerovec, but it is possible that the destruction was caused by the armed invasion of Moravians. In the 18th century, the capacity of the small church was increased by the addition of wooden galleries on the southern and northern walls of the nave, but in the first half of the 19th century they were removed due to poor condition. In 1884, the neglected building was renovated, with new columns installed in the tower windows and a new entrance portal. Professional repair of the church took place in 1936, and the last restoration began in 2008. During its course, wall paintings were discovered inside. A similar church is located close to Nitra, in Dražovce.

0,20€ Church of the Reformed Christian Ecclesia in Svätuše. Romanesque Church from the early 13th Century. The original church was built on a hill in a dominant position above the village sometime at the beginning of the 13th century. It was a small brick one-nave building with a western tower, an emporium and a sanctuary of unknown shape, which, according to everything, was among the most luxurious rural churches of the Romanesque period in our territory. In written sources, the village and the church are mentioned as early as 1228. In the second half of the 15th century, it no longer suited the growing number of believers, so it was radically rebuilt. The nave of the church was expanded to the south and east, which required the demolition of the southern Romanesque wall of the nave and the original sanctuary. It was replaced by a significantly larger polygonal presbytery. In addition, a sacristy with a rectangular floor plan was added to it from the north side. By extending the nave to the south, the tower went outside the axis of the west front. In the 16th century, local believers converted to the Calvinist faith, while the oldest local priest is documented as early as 1598. The church underwent a Baroque reconstruction in 1794, when the nave received a wooden coffered ceiling. The building

acquired its current form only in 1907, when a longitudinal chapel was added on the north side. This brought the Romanesque tower back to the center of the western facade. At that time, the Gothic sacristy no longer existed, and only parts of the walls remained, which were transformed into the supporting pillars of the presbytery. The church was restored in 1909 and 1967, since 1963 it has the status of a national cultural monument. The complex restoration was carried out in 2005 - 2017.

0,33€ Church of the Nativity of the Virgin Mary in Čierny Brod (Black Ford).

The church was built in the second or third decade of the 13th century. Around 1550 or at the beginning of the 16th century, the building was enlarged. The original apse was replaced by a polygonal presbytery, a tower was added from the west, and the central nave was widened with aisles. At the end of the seventeenth century, the temple was in bad condition, in the second decade of the eighteenth century there was even the threat of collapse. The change took place in 1752 when a thorough renovation of the building was carried out. An important breakthrough in the architectural development of the building was a fire in 1915. In the following years, the aisles were removed and original facades were discovered. Another reconstruction of the church took place in 1940-1943. At that time, the tower was removed and the façade was restored to the original state.



a)



b)



c)

FIG. 16. a) 0,10€ Church of St. John the Baptist in Sedmerovec-Pominovce, photo Marek Šeregi; b) 0,20€ Church of the Reformed Christian Ecclesia in Svätušé, photo Szeder László; c) 0,33€ Church of the Nativity of the Virgin Mary in Čierny Brod (Black Ford), photo L. Repta. (Courtesy of Wikimedia Commons).

0,50€ Church of St. Martin is located in the town of Spišská Kapitula and is the cathedral church of the Spiš diocese. The cathedral was built between the 13th and 15th centuries in the Romanesque and Gothic styles. It is one of the largest and most interesting Romanesque monuments in Slovakia. It contains many medieval carved altars and is the resting place of many lords of Spiš Castle; the 15th century carved marble tombstones of the Zápolya family are of exceptional quality. A recently restored wall-painting from 1317 depicts the coronation of Charles Robert of Anjou as the King of Hungary; another painting in the cathedral is the source for the provisional name of the anonymous Master of Kirchdrauf. Spišská Kapitula became the main seat of the church administration in the region in the 12th century. In 1776 it became the seat of the Diocese of Spiš, after Hungarian queen Maria Theresa split the Diocese of Eger (now Archdiocese of Eger). Spišská Kapitula was visited by Pope John Paul II in 1995.

0,66€ The Church of St. Aegidius in Ilija. Ilija municipality is a small but beautiful village situated under the highest hill of the Štiavica Mountains – Sitno. The picturesque village is dominated by already mentioned Roman-Catholic church dedicated to St. Abbot Aegidius. The church of St. Aegidius (Svätý Egídius) played a crucial role in development of the village. Entire settlement was named after the patron of the church: until 1526 Ilija

was called Sanctus Egidius in Latin-written documents and Swaty Ilik in old-Slovak language. We don't know much about the development of Ilija from 1266, when several written sources mention the Church of Saint Aegidius, to the late 14th century.

0,83€ Church of the St. Stephen the King in Žilina-Závodí. The church was built in the first thirty years of the 13th century. Shortly after completion of construction, the interior of the chancel was decorated with late-Romanesque murals. The decorations of the church were made by Dominicans from Kláštor pod Znievom, who stayed there before 1250. In the 15th century, a funeral chapel was built in the southern part of the church. The church served as a parish temple for many remote settlements. Even during the Reformation, it remained Catholic. In the turbulent 17th century, it was fortified with a perimeter wall. Probably in 1762, the Jesuits carried out an extensive rebuilding, replacing the original wooden ceiling of the nave with a new vault. Unfortunately, in this period also the original Romanesque windows were also replaced by new, larger ones. At the beginning of the 19th century, the church served as a military warehouse and was not repaired until 1849. At the end of the 1940s, murals in the corpus, apse and eastern part of the nave were discovered, prompting the church to be renovated. The paintings were restored in 1955-1956. Archaeological research carried out in 1995, revealed the foundations of the funeral chapel. In 2008 and 2009, research was carried out, and then the frescoes in the interior of the church were renewed.



a)



b)



c)

FIG. 17. a) 0,50€ Church of St. Martin in Spišská Kapitula, photo J. Kotulič; b) 0,66€ Church of St. Aegidius in Ilija not far from Banská Štiavnica; c) 0,83€ Church of St. Stephen the King in Žilina-Závodí, photo Z. Grúňová. (Courtesy of Wikimedia Commons).

1,00€ Church of the Assumption of the Blessed Virgin Mary in Bíňa. The church was built in the late 12th or early 13th century as part of a Premonstratensian monastery, founded by Comes Omodej, before he set out on a crusade alongside King Andrew II in 1217 or 1218. It was first recorded in written sources in 1273, in a document containing the last will of the deceased Magister Stefan, a member of the Hunt-Poznan family, who was to be buried in the church in Bíňa and which confirmed the donation made to the monastery by his father Omodej. This donation was most likely transferred to an already functioning convent. At the turn of the 12th and 13th centuries, at least some stonecutters worked on the construction of the monastery, taking part in the reconstruction of the cathedral and the royal and archbishop's residence in Esztergom. Works in Bíňa probably lasted until the end of the second decade of the 13th century, being in its final stage at the time of the founder's crusade. Omodej's descendants kept the patronage over the monastery and church probably throughout the Middle Ages. In the 16th and 17th centuries, the surrounding lands and the abandoned monastery were under the rule of the Turks, with the Premonstratensians leaving Bíňa already in the 16th century. The

monastery complex was captured in 1683 by Polish troops returning from Vienna, which led to the destruction of most of the buildings. The renovation of the church itself took place in 1722-1732, when the nave and the vestibule were vaulted again. During it, the Baroque reconstruction of the temple was carried out (transformed windows in the nave, porch and side chapels, building the southern sacristy), and in the years 1861-1862 another modernization took place, unfortunately carried out in a rather arbitrary way. It caused so much opposition that in the years 1895-1898, further renovation works were carried out, this time in the neo-Romanesque style. The building was severely damaged during World War II. The explosion destroyed the vestibule (porch), almost the entire north tower and the upper part of the south tower, as well as the nave's roof. Fortunately, in the years 1951-1955 a comprehensive reconstruction of the monument took place, combined with architectural research.

1,33€ Church of the Holy Cross in Hamuliakovo. The first building in the area of today's church was a polygonal, octagonal rotunda, built around 1220. After about 40 years, it was enlarged into a single-nave church with a tower. Subsequent reconstructions took place in the Baroque period, and later in the 19th century. At that time, a sacristy was built on the northern side of the chancel, and due to the problems of the structure with the static, the chancel was reinforced with buttresses. Additionally, a brick vestibule was added in front of the original Romanesque portal. In the early 1930s, the rebuilding of the tilting tower was considered, but fortunately the authorities did not allow it, due to the great value of the church. In the 70s of the last century, research was carried out, the results of which became the basis for the renovation project of the monument in 1979-1981, during which, among others, medieval wall paintings were discovered. After the building was statically secured, because it was still in danger of collapsing, the early modern buttresses at the presbytery and the southern vestibule were removed.

2,00€ Church of Archangel Michael in Dražovce. Romanesque church of St. Michael was founded at the beginning of the 12th century on the site of an iron age settlement and on the foundations of an earlier temple from the mid-eleventh century. In the twelfth or thirteenth century, it went under the administration of the Benedictine monastery. At that time, it was also added a western turret and most likely a gallery. At the end of the 18th century, a rebuilding was carried out, during which the perimeter walls of the nave were raised, the entrance was widened, the porch and windows on the tower were modified, and the interior fittings were replaced. In 1803, a new church was built directly in the village, so the church of St. Michael ceased to fulfill its original role. Fortunately, it avoided demolition, and renovation works were carried out in the 19th and 20th centuries.



a)



b)



c)

FIG. 18. a) 1,00€ Church of the Assumption of the Blessed Virgin Mary in Biňa, photo Marek Šeregi; b) 1,33€ Church of the Holy Cross in Hamuliakovo; c) 2,00€ Church of Archangel Michael in Dražovce. (Courtesy of Wikimedia Commons).

Conclusion

Information about 24 historical religious structures belonging to the cultural heritage of the Slovak republic [1]. Some of them were promoted on the postage stamps.

Information about cultural heritage together with knowledge of terminology of civil engineering and architecture [2] is very important part in education of structural engineers. Experience of members of Department of Languages at Faculty of Civil Engineering, Slovak University of Technology in Bratislava, Slovakia, confirms it.

Acknowledgement

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SURVEY AND ANALYSIS OF DETAILS FOR CONNECTION BETWEEN TIMBER FLOOR CONSTRUCTION AND MASONRY WALLS IN HISTORIC BUILDINGS

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Abstract: *This paper presents a literature review on the traditional and innovative wall-to-timber floor anchors used in unreinforced masonry buildings taking into account experimental works, analytical and numerical studies. Out of plane failure of unreinforced masonry walls is considered one of the most vulnerable seismic hazards in medium and high earthquake-prone regions. Efficient structural connections enable global equilibrated mechanisms, ensuring the "box-behavior". Timber floors are widely used as horizontal diaphragm in historical constructions, and their connection to the boundary walls is crucial to facilitate internal force redistribution and to restrain out of plane walls*

Key words: *out of plane failure, timber floors, box-behavior, numerical studies*

ПРЕГЛЕД И АНАЛИЗ НА ДЕТАЙЛИТЕ ЗА СВЪРЗВАНЕ МЕЖДУ ДЪРВЕН ПОДОВ ГРЕДОРЕД И СТЕНИ В ЗИДАНИ ИСТОРИЧЕСКИ СГРАДИ

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Абстракт: Докладът съдържа обстоен преглед на съществуващи: традиционни и иновативни детайли за анкериране на връзка между под и стена, използвани в сгради от неармирана зидария, като се вземат предвид експериментални разработки и други аналитични и числени решения на проблема. Извън равнинното разрушаване на зидани тухлени или каменни стени се счита за една от най-уязвимите сеизмични опасности в средно и силно земетръсни райони. Ефективно изпълнените конструктивни връзки между дървения подов гредоред и зидарийно изградените стени, позволяват възникването на глобални уравновесени състояния, характеризирани като механизми, осигуряващи появата на т.н. поведенческо поведение на кутияобразните черупкови конструкции: „box-behavior“. Обикновено дървените подови гредореди (конструкции) се считат за хоризонтални диафрагми в историческите подови конструкции, като връзката им с фасадните стени, играе важна роля при проявата на вътрешно преразпределение на силите на контакта между тях.

Ключови думи: *извън равнинно разрушаване, дървен гредоред, числено изследване*

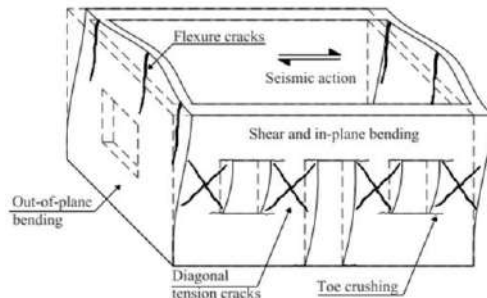
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1. Въведение

Съгласно [45], разрушаването извън равнината на неукрепени, зидарйино изградени тухлени или каменни стени, което явление е наблюдавано както в минали, така и в по скорошни случили се сеизмични събития (M6.3 L'Aquila от 2009 г.), се приемат от специалистите в тази област, като сериозна и нежелателна първо-причина [1], за развитие на последващи катастрофални повреди. Старите зидани сгради често са изградени без прилагане на сеизмични концепции и практики, като реагират слабо дори при земетресения с ниска и средна интензивност [2]. Голямото разнообразие от видове стротелни материали и геометрия принуждава анализаторите да изследва всеки проблем като уникален, като се има предвид (когато е възможно) да получат по експериментален път свойствата на вложените материали. Без правилно решени и реализирани аксиални връзки: между дървения подов гредоред и фасадните стени (разположени) перпендикулярно на направлението на сеизмичните сили), същите са силно уязвими към повреди извън тяхната равнина [3]. Достатъчно коравата подова диафрагма (спрямо фасадните стени) се изисква също така, да бъде свързана към страничните стени в равнина (успоредно на посоката на сеизмичните сили) чрез подходящи връзки на срязване, способни да разпределят инерционните сили към сеизмично-устойчивите стени. Подовата диафрагма, съвместно с добре изпълнените сеизмични детайли, осигурява т.н. кутиеобразно черупково поведение, „box-behavior”, с активиране на глобални уравновесени механизми [4] (Фиг. 1).

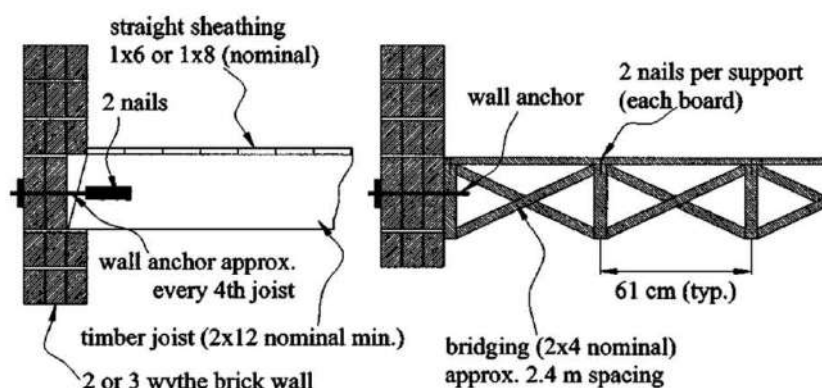


Фиг. 1. Глобални равновесни механизми, които обикновено се наблюдават в историческите сгради на URM[4]

Въпреки, че влиянието на сеизмичното анкеряване между стените и подовите конструкции на историческите обекти е добре познато сред инженерната общност, това е често пренебрегвана тема при оценката на съществуващите сгради. Освен това изследователите се фокусират главно върху глобалното сеизмично поведение на сградите и се предлагат решения за укрепване само на първичните конструктивни елементи, като стени и подове или покриви [5,6]. Тази статия представя преглед на детайлите за анкеряване между стените и подовия гредоред, използвани за намаляване на сеизмичната уязвимост на зиданата конструкция. Най-често срещаните в практиката съществуващи решения, а така също и по-съвременните, иновативните закрепвания, открити в литературата, както и експерименталните и числените подходи са обобщени в изложения по долу текст, в съответните: 2, 3 и 4 точки на доклада.

2. Описание на анкерни устройства между подов гредоред и ограждащи стени

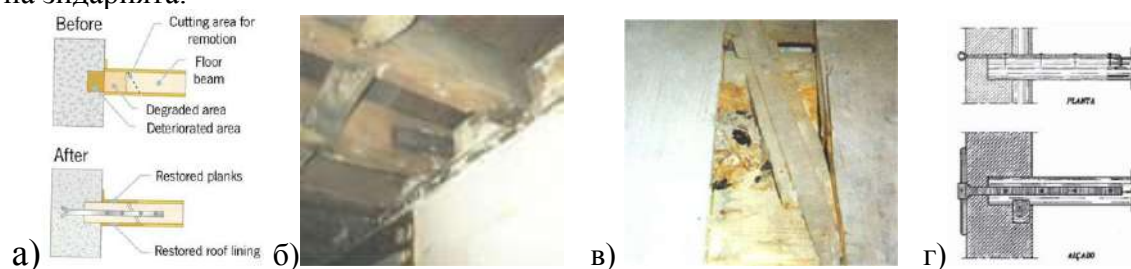
Съгласно [45] една от най-често срещаните и широко разпространени видове хоризонтални диафрагми в историческите обекти са изпълнени от дървени гредореди, изпълнени от масивна или многослойна дървесина. В тази строителна практика през онази епоха, елементите от дървеният гредоред се полагат в специални легла с размерите на гредата създадени в зидарията. По този начин се създава връзка между двата елемента, чиито капацитет е обусловен от големината на аксиалното изтръгване, породено от силите на триене между тях, генерирани от вертикалните статични натоварвания. В практиката се срещат различни решения, при които се използват метални анкерни елементи, които свързват външната тухлена или каменна стена с дървената диафрагма. Тези устройства (изпълнени главно от желязо или стомана) имат за основна функция да осигурят връзка между дървения гредоред и зидарията, чрез използването на метални плочи, които са закрепени към дървесината чрез болтови съединения или гвоздей. От друга страна към тези метални пластини се закрепват, чрез заваряване, коване или нитоване дълги шпилки, които пронизват зидарията и се свързват със специална плочеста котва, която се опира във фасадата на сградата и чрез своята повърхност закотвя към себе си подовата диафрагма на сградата, като я принуждава да се движат комплексно по направлението на сеизмичните въздействия. За това голямото разнообразие от подобни връзки, може да се каже, че те могат да пренасят безпроблемно, аксиални или срязващи сили при статични вертикални сили или динамични странични въздействия. Примери за реализирани на подобни анкерирания за стени, могат да бъдат намерени в зидани тесгради от 50-те години на миналия век в Средния Запад, в Съединените щати, където подовите конструкции са фиксирани към зидарийната стена, чрез просто приковаване към дървената гредка към стенния анкер, осигурявайки относително слаба връзка със зидарията на стената, перпендикулярно или успоредно [7,9] (Фиг. 2).



Фиг. 2. Железни котви, свързващи външни стени с перпендикулярна (а) или успоредна гредка (б) [8]

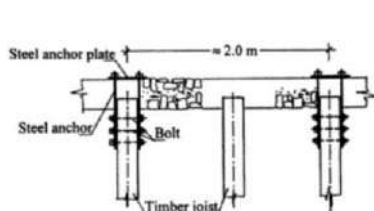
Съгласно [45] подобни връзки (наподобяващи пръти с типична форма на рибена опашка) могат да се използват за свързване на новата дървена глава (на мястото на деградиралата) с останалата част от гредата и стената [9] (Фиг. 3а). При тези детайли, трудно може обаче да се постигне сеизмична ефективност на срязване на тази връзка. Примери за по-сложни системи могат да се видят в сградите: "Помбалино" в Лисабон, построени след земетресението от 1755 г., като опит за създаване на антисеизмична зидарийна сграда. В периметъра на зиданите стени се изграждат дървени вертикални сеизмични рамки (Помбалино), които

подобряват връзките между дървения скелетен материал и зидарията. Дървените плочи се поставят над и под главата на гредоредата, вътре в зидарията и се фиксират с гвоздеи [10], което позволява по-добро разпределение на натоварването към стената и подобрява аксиалното и срязващо съпротивление. Типизираните сеизмични подови връзки представляват Г-образни, метални листови котви, приковани към дървения материал и фиксирани в зидарията (Фиг.3б). Освен това дълги масивни дървени елементи (под формата на пръти с дължина по 1м), заковани диагонално към гредите и вградени под дървените дъски, могат да бъдат закотвени към зиданата стена, създавайки по-ефективни решения (Фиг. 3в). Решенията, показани от Sóias [9], включват обикновени железни пръти, квадратни, звездовидни или кръгли плочи, закрепващи прът към външната страна на зиданата стена (Фиг. 3г), увеличавайки сеизмичната устойчивостта на връзката, при което се разчита и на якостта на натиск на зидарията.



Фиг. 3. Анкерни устройства между греда и стена[9,10]: а) Деградираше дървена греда-саниране на нейната опорна част; б) Желязна котва; в) Диагонално разположен прът между подовите греди; г) Анкери между покривни греди и стени

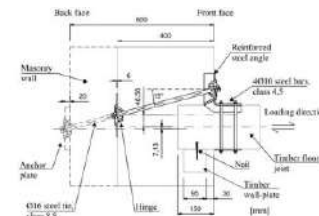
Усилващо сеизмично решение е предложено от Tomažević [4], което включва двоен стоманен анкер, симетрично закрепен към гредата, за да се избегне ексцентричното му поведение при сеизмични въздействия (Фиг. 4).



Фиг. 4. Двойна анкерна система [4]



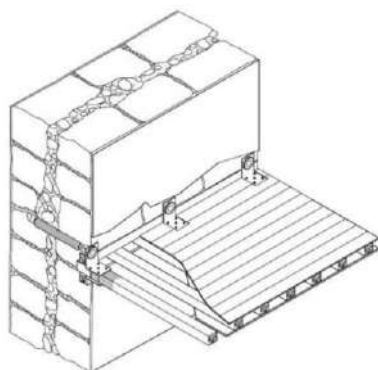
Фиг. 5. Анкерирание чрез Г-образни СОП



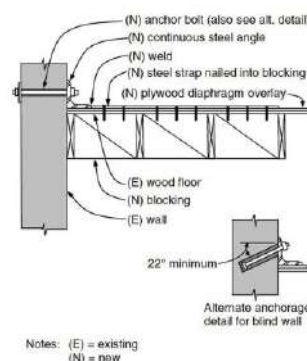
Фиг. 6. Решение на Moreira [12]

Съгласно [45] болтовите връзки увеличават съпротивляемостта на срязване и подобряват сеизмичното поведение на разтвора. Допълнителни решения за усилване на анкерните връзки, приети след земетресението в залива Хоукс през 1931 г. в Нова Зеландия, са описани от Blaikie и Sprigg [11] и включват стоманени котви, свързани към стоманена ъглова плоча с болтове към гредата. Закрепването към зидарията се осигурява от използването на стоманени носещи плочи (Фиг. 5). Иновативните решения, изследвани от Moreira [12], включват ставни връзки в двата края на стоманения прът, позволявайки по-лесно производство на самата връзка, която се фиксира диагонално върху подходящо подсилени стоманени ъглови профили директно от горната част на пода (Фиг. 6). Използването на инжекционни котви също се превръща в интересно решение, благодарение на въвеждането на смоли или разтвори на епоксидна основа и позволява минимална намеса в аспекта на оригиналната фасада. Пример за такова решение е предложен от Sóias [9], където

инжектирането на фугираща смес контролира поведението на свързване между стоманената връзка и ограждащата зидария (Фиг. 7). Връзката с дървената греди обикновено се осигурява от стоманен ъглов профил с болтове.



Фиг. 7. Решение за укрепване Помбалино [9]



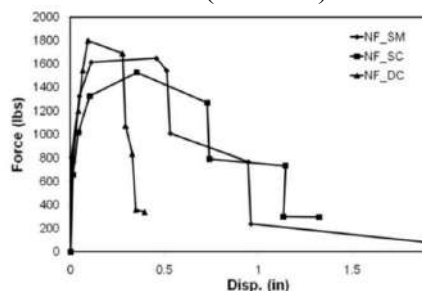
Фиг. 8. Връзка за укрепване на сгради с помощта на непрекъснати Г-образни СОП [13]

Съгласно [45], в началото на 20-ти век, появата на стоманобетон (RC) доведе до почти пълното изоставяне на зидарията, като основен конструктивен материал. Освен това, съвременните сгради от URM са снабдени с (RC) свързващи греди на нивата на пода, позволяващи връзката между плочата и стената, да създаде ефективно поведение на диафрагмата, което обикновено е с ниско качество в съществуващите сгради от (URM) с гъвкава диафрагма [4]. И все пак дори и да е била очаквана подмяната на тези леки дървени подове или покриви с тежки RC плочи, това решение трябва да бъде избягвано поради увеличаване на опасните инерционни сеизмични сили. Hsiao и Tezcan [13] предлагат решение за укрепване на сградата на базата на американския стандарт FEMA [14], като се избягва по този начин, повреждането на оригиналната фасада на историческата сграда. Решението е показано на (фиг.8) и има за цел да усилва връзките между дървените диафрагми и носещите стени на URM и да увеличи капацитета на огъване на диафрагмите, като се използва непрекъснат стоманен Г-образен ГВП по ръбовете на дървената подова диафрагма, свързана към зидарията с помощта на анкерни болтове или алтернативни инжекционни анкерни елементи, с цел да се запази оригиналната конструкция на външната фасада.

3. Експериментални изследвания

Съгласно [45], експерименталната лаборатория или тестът на място са изключително важни, за да се разбере влиянието на укрепващите връзки върху цялата сграда и за характеризиране на механичното поведение на съществуващата и засилена връзка. Глобално разбиране на ефективността на връзките може да бъде постигнато чрез тестове върху земетръсна маса с разклащане върху образци в реален или намален мащаб. Такива тестове обикновено се извършват в глобален мащаб на сградата [15,16,17] или локален мащаб на стената [18,19], но само няколко имат за цел да проучат ефективността на връзките стена-под, при което се предлагат укрепващи решения само за първичните елементи [6, 20]. Статичното и динамично охарактеризиране на връзките може да бъде постигнато чрез тестове на срязване, насочени към разработване на диаграми за: сила-преместване, необходими за валидиране на числения модел. Типичните връзки между диафрагма и тухлена стена са възпроизведени от Lin и LaFave [7] и са изследвани чрез статични монотонни,

както и динамични циклични тестове, като се има предвид приносът на триенето и наличието на анкерните устройства (Фиг. 9а). Резултатите показват, че динамичната реакция е по-малка в сравнение със статичната такива (Фиг. 9б).

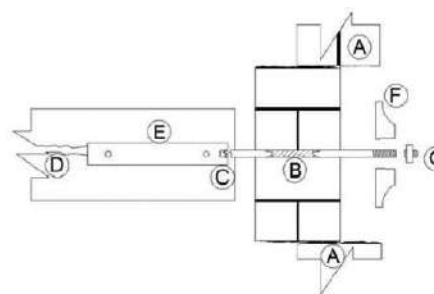
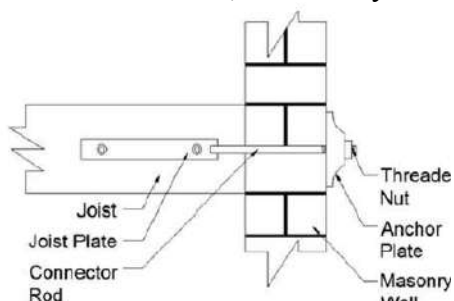


а)

б)

Фиг. 9. Експериментална програма проведена върху типизирана стенна диафрагмена връзка (7): а) образци за изпитване; (б) сравнение между статичен монотонен (SM), квазистатичен цикличен (SC) и динамичен цикличен (DC) изразени чрез усреднени диаграми - криви, отчитайки приносът на триенето и наличието на прикованите щрипси (NF)

Шест съществуващи стоманени котви, взети от повредена сграда на URM след земетресението M7.1 в Christchurch (Крайстчърч) от 2010 г., са проучени от Campbell et al. [21] чрез аксиални разрушителни изпитвания. Въз основа на такъв сравнително малък брой тестове на обрзци и като се имат предвид наблюденията на последователността на щетите от земетресения в Canterbury (Кентърбъри) от 2010/11 г., авторите предлагат седем правдоподобни режима на повреда на връзката и стойностите на крайната якост на опън, като се отбелязва, че са необходими допълнителни тестове, които да увеличат набора от данни (Фиг. 10).



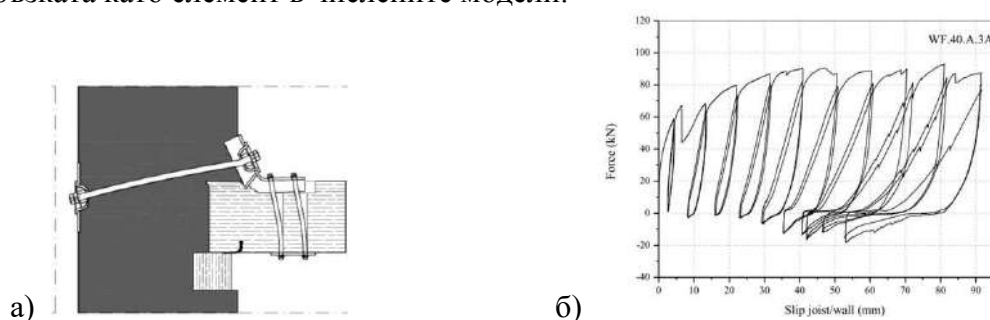
а)

б)

Фиг. 10. Типични анкерни устройства прилагани в Нова Зеландия: (а) монтажно състояние; б) местоположение на режимите на отказ

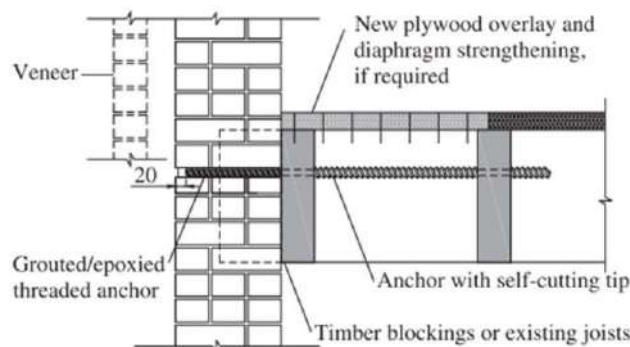
Jacks и Beattie [22] изследват статичното и динамично поведение на анкерите с проходен болт и епоксидна смола в стените на URM. Не се установени повреди при болтови анкери дори при високи нива на натоварване, докато образуването на пукнатини, разпределени по цялата ширина на тестовия панел, се получава за анкери с епоксидна смола. Набор от уравнения за проектиране е предложен от Karim et al. [23] за оценка на болтовата връзка на дървената гредя въз основа на експериментални проучвания, фокусирани върху механизма на повреда. Минималната стойност на якост управлява капацитета на цялата връзка. Типичните връзки между стена и подова конструкция, от края на 19-ти век в Лисабон са експериментално изследвани от Морейра [12], както в неукрепена, така и в усилена конфигурация чрез квазистатични монотонни и циклични тестове. Подсилената връзка (Фиг. 6) се базира на стоманени анкерни пръти, закотвени към стената с анкерни плочи. В този детйл, ъглови стоманени профили са поставени от страна на

помещението на сградата. Те са свързани чрез хоризонталното си рамо с болтове към дървената гредка. Вертикалният пояс на ъгловия профил е свързан с анкерното устройство. С други думи, от вътрешната страна на помещението се поставят ъглови стоманени профили, за да свържат стоманените анкери към дървената гредка с помощта на болтови съединения. Ставни устройства са разположени в двата края на връзките, намаляващи силата на огъване, предизвикана от дървената гредка свързана към анкера, закрепен диагонално в зидарията. Капацитетът на опън на усилените връзки е приблизително 19 пъти по-голям от този на неукрепените (гредоред, просто закован към долната дървена плоча и вграден в зидарията) и се характеризира с податливост на стоманения ъгъл, и смачкване и срязване на дървения материал (виж: Фиг. 11). Препоръчано е проектно предложение за модернизация и се предлагат бъдещи дейности за установяване на хистерезисни правила, полезни за реализиране на връзката като елемент в числените модели.



Фиг. 11. Цикличен тест на иновативна усилена връзка (12): (а) наблюдавана повреда и режим на повреда; (б) препоръчват се циклични криви и се предлагат бъдещи работи за установяване на хистерезисни правила, полезни за реализиране на връзката като елемент в числени модели.

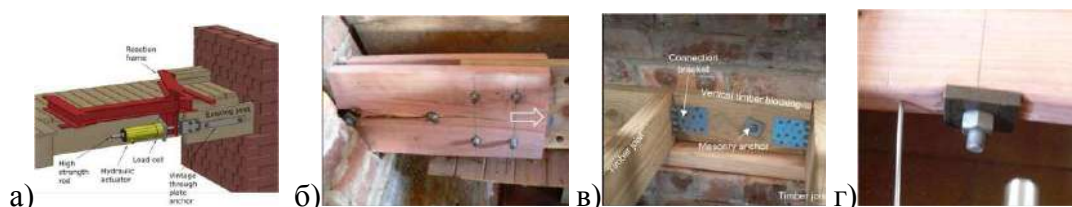
Капацитетът на изтръгване (**РОС**) на фугираните анкери е изследван от Ismail [24], като възможно решение за модернизиране на връзките: стена-диафрагма (Фиг. 12). Изследвано е влиянието на дължината на вграждане на анкера, качеството на монтаж, местоположението на анкера, състоянието на зидарията и състоянието на материалите на основата върху работата на анкера. Най-високата вероятност за повреда по време на земетресение е свързана с повреда на връзката котва-диафрагма и по този начин ограничава постижимия РОС на системата за закрепване стена-диафрагма. Въпреки това, този тип повреда се наблюдава като доста дуктилна.



Фиг. 12. Възможна конструкция за укрепване, предложена от Ismail [24] на базата на фугиран резбован анкер

По-нататъшни проучвания върху връзките на лепилни анкери между тухлени URM стени и подова диафрагма са предприети от Dizhur et al. [25], имайки пред вид

повече от почти 400 екземпляра. Авторите се фокусират върху ролята на основните параметри върху РОС на цялата връзка. Циментовата фугираща смес представлява подходящо анкерно лепило при оптимален размер на диаметъра на пръта равен на 16 mm. Ниското тегло на нанесеният материал оказва негативно влияние върху РОС, докато точният процес на нанасяне го увеличава значително. При нито едно от изпитванията не се наблюдава конусна повърхност на зидария и се предлага проектно уравнение на РОС на лепилните анкери. Неотдавна Dizhur et al. [26] предприемат експериментални тестове на два различни типа закрепвания между анкера за зидария и дървената диафрагма: (i) метален съединител, закрепен с болтове към гредата през гредоред (Фигура 13а) и (ii) връзки, използващи блокиране на дървен материал (вертикално и хоризонтални дървени елементи, свързани помежду си между гредите, закрепени с болтове към анкера за зидария) (Фигура 13б). Резултатите показват как анкерната връзка на плочата на греди основно се компроментира при разцепването на дървената гред (Фигура 13в), докато най-високата качество се постига при хоризонтално позициониране на дървената блокировка. Наблюдава се повреда на дървесината в мястото на подпирането на гредата ако се използват малки по размер шайби (Фиг. 13г).

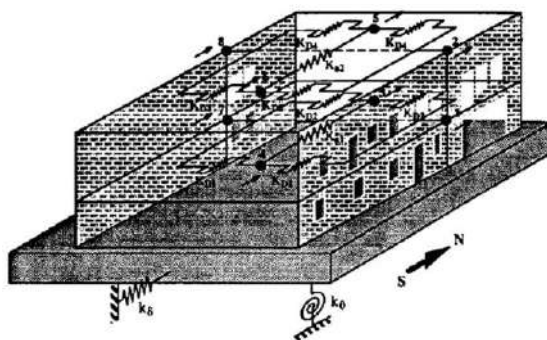


Фиг. 13. Експериментална програма върху образци от стена-диафрагма [26]: (а) схематична диаграма на свързване на гредата с плоча; б) разрушаване при редово срязване на дървените греди; в) система за блокиране на дървен материал; г) повреда на дървесината в мястото на подпирането на гредата, при малки по размер шайби

4. Числени и аналитични методи за анализ на поведението на връзките

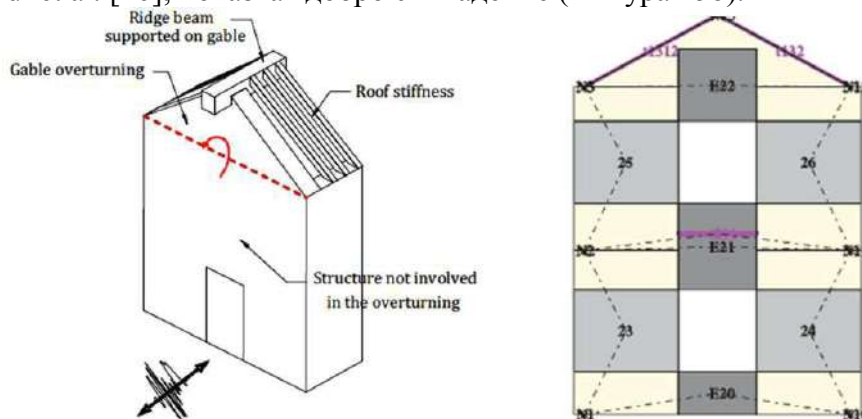
Съгласно [45], налични са значителен брой техники за моделиране при сеизмичен анализ и оценка на URM конструкции в мащаб на сградата [27,28] и локален мащаб (т.е. ООР механизъм) [29,30], но само няколко включват поведението на връзките между интерфейсите на вертикалните и хоризонталните елементи [19,31]. Нещо повече, авторите често разглеждат връзката като приемат опростена хипотеза (т.е. перфектни ставни връзки или линейни еластични пружини), пренебрегвайки цикличното нелинейно поведение, характерно за много видове връзки и намалявайки броя на степените на сложност на динамичната задача. Най-често срещаните техники за моделиране включват подходи с твърд блок, числени модели на крайни елементи (FEM) и модели на отделни (или дискретни) елементи (DEM). За подходи с твърд блок капацитетът може да бъде изчислен чрез статичен или кинематичен анализ. От друга страна, статично преместване и динамичен анализ с интегриране на времеви етапи при изкуствени или реални акселерограми са често срещани инструменти, когато се приемат FEM или DEM техники [32,33]. Трябва да се отбележи, че подходите, базирани на сила (FBA) са предпочитани от практикуващите като прости и директни методи, въпреки високите консервативни резултати, докато подходите, базирани на преместване (DBA), изглежда са подходящи за напукани стени, попадащи в нелинейния диапазон (29). Влиянието на пода върху сеизмичната реакция на URM конструкции е изследвано в работата на

Tena- Colunga и Abrams (34) чрез дискретни динамични модели, базирани на линейно поведение на зидарията и гъвкави и линейни диафрагми, моделирани като еластични пружини (Фиг. 14). Резултатите показват как ускоренията на диафрагмата могат да се увеличат с гъвкавостта на диафрагмата. Освен това ефектите на усукване намаляват значително, с увеличаване гъвкавостта на диафрагмата.



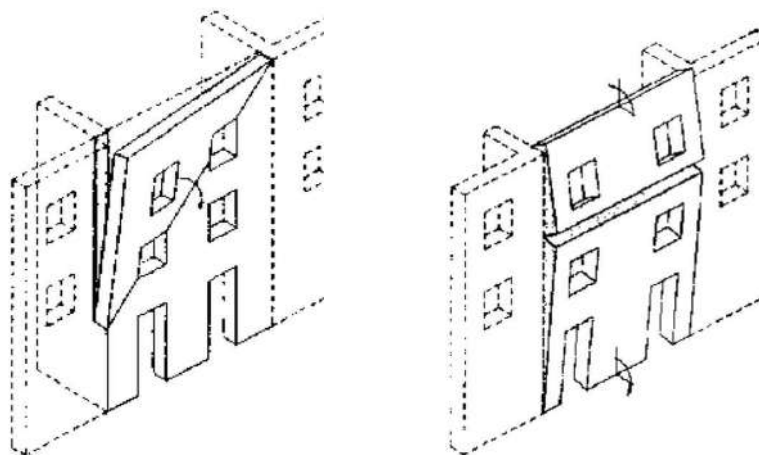
Фиг. 14. Дискретни динамични модели на сгради URM от Tena-Colunga и Abrams [34]

Съгласно [45], неконвенционална процедура за симулация на динамичното поведение на ООР на сгради URM, като се има предвид коравината на покрива, се извършва от Costa [35], където локалните механизми се моделират като кинематични вериги от части от зидария (обикновено се приемат като безкрайно твърди тела), чието нелинейно поведение е концентрирано в контактните области (Фигура 15а). Пружините симулират IP коравината на хоризонталната диафрагма и натрупаната маса, разположена на билото, а горните разпръскващи греди представляват масата на покрива. Експерименталното поведение на пълномасщабна сграда URM, тествана в EUCENTRE (15), се симулира с достатъчна точност чрез този числен подход, при който еластичните пружини са поставени в горната част на всеки наклон на покрива. Предварителното дефиниране на формираните локални механизми определя границите и недостатъка на метода, но подходът е бърз и прост метод, разчитащ само на допускането на няколко параметъра. Софтуерът TREMURI [36] се използва като еквивалентен рамков подход, където е възможно да се включат нелинейни прътови елементи, за да се отчете наличието на RC пръстеновидни греди и връзки стена-диафрагма на нивото на пода. Кривите на Pushover анализа, получени чрез този подход, се сравняват с експерименталната крива на съпротивление, разработена от Senaldi et al. [16], показват добро съвпадение (Фигура 15b).



Фиг. 15. Модели на макроеlementи на сгради URM: (а) базирани на динамика на „multi-body „ подход (35); (б) еквивалентен рамков подход, разработен в TREMURI [16]

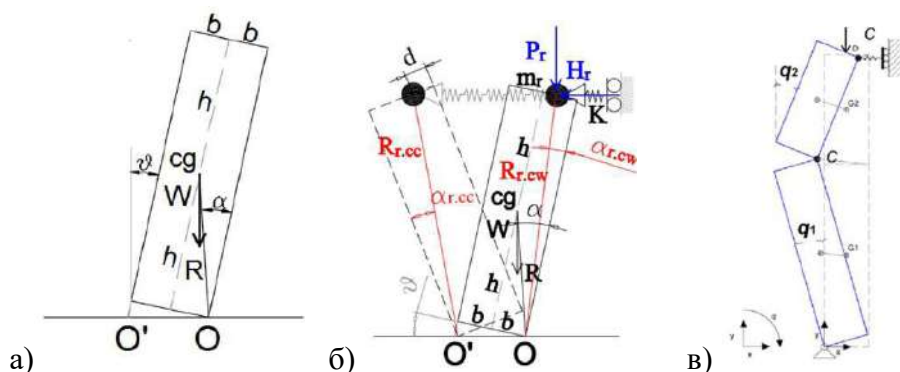
Подходите в локален мащаб са от съществено значение при оценката на местните механизми (т.е. ООР завъртане на части от стените), често активирани при удари на сгради URM от сеизмични събития, с висок или нисък интензитет. Започвайки от работата на Giuffrè [37] от 1994 г., анализът на границите на кораво тяло разкрива, че е мощен инструмент за проверка на сеизмичната безопасност на локалните механизми, където е възможно да се отчетат техники за укрепване като вмъкване на обтегач, корава диафрагма, котви и др.. Максималният хоризонтален множител на натоварване може да бъде сравнен с максималния очакван PGA, оценен от спектъра на реагиране за билинеен недисипативен модел. Допускането на локалния механизъм е необходимо, за да се използва такъв подход. D'Ayala и Speranza [38] предоставят няколко формулировки на коефициента на натоварване за всеки механизъм за преобръщане (Фиг. 16), внедрен в процедурата FaMIVE (Failure Mechanism Identification and Vulnerability Evaluation) за онлайн оценка на сеизмична уязвимост на фасада или сграда. Такава процедура може да се прави с голям брой сгради, за да се извърши статистически анализ, като се вземе предвид наличието или липсата на укрепващи устройства.



Фиг. 16. Механизми на разрушение, проучени от D'Ayala и Speranza [38]

Съгласно [45], когато може разумно да се приеме зидарията за монолитен модел участващ в механизма на движение, динамичният анализ на люлеене представлява надеждна алтернатива на кинематичния подход. Въз основа на пионерската работа на Хауснер [39], люлеещото се движение на обикновен твърд призматичен блок около два ъгъла О и О' (Фигура 17а) може да бъде анализирано при свободни вибрации, постоянни или синусоидални ускорения и реално или изкуствено движение при земетресение. Приносът за прилагане на уравнението на движението върху компютърна програма е даден от Aslam [40], докато Giresini [41,42] въвежда влиянието на възможни тласъци и хоризонтални ограничения за симулиране на прътови обтегачи, гъвкава диафрагма или напречни стени (Фиг. 17b). Методът представлява обещаващ подход, но все още не намира приложение в стандартните правилници, дори когато сравненията с експериментални резултати демонстрират неговата точност. MDOF системите могат да бъдат по-реалистично представяне на ООР люлеещите се механизми в стените на URM. За да отчетат такива по-сложни системи, различни автори (43,44) извеждат уравнението на движението за 2 или 3DOF модели, но само Landi et al. (31) отчита влиянието на гъвкавите диафрагми върху ООР огъването на URM фасадите. Диафрагмата се симулира чрез въвеждане на пружина отгоре (Фигура 17в) и моделът се калибрира при експериментални

тестове. Резултатите показват високата чувствителност на модела към коравината на пружината.



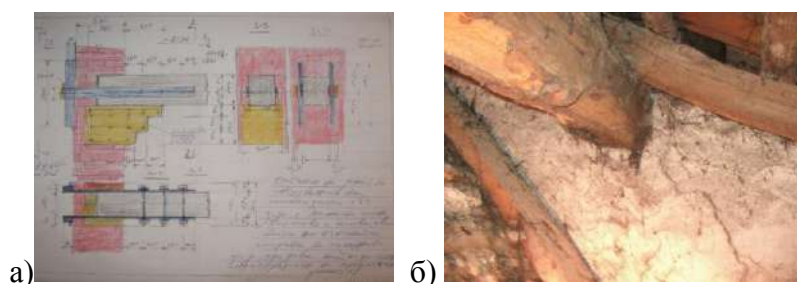
Фиг. 17. Анализ на динамична устойчивост на корави люлеещи се блокове: (а) Модел на свободен блок на Хауснер (39); (б) ограничено при движение люлеещ се блок [41]; (в) 2DOF модел [31]

5. Проект за анкериране на дървени подови греди на емпорето към каменната стена на църквата Св.Димитър-гр. Кюстендил

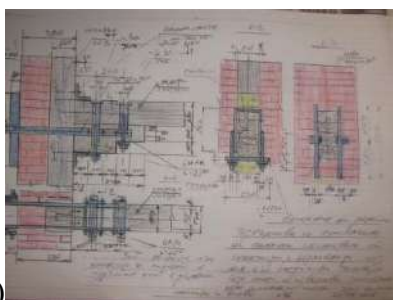
След разкриването на ламперията под подовите греди и дъсчената обшивка над тях е необходимо да се извърши анализ на „носимоспособността” на отделни образци от повтарящи се по типоразмери греди, при отчитане възрастта на отделните елементи и намалените им еластични модули и якостни характеристики (E и R). Ако е необходимо по преценка на проектанта подовите греди се усилват със стоманени ивици или ГВП, прикрепени двустранно към стените на гредите [46]. При силно изразени дефекти: провисвания, пукнатини и изгнили или унищожени от биологични вредители техни части по преценка на проектанта тези греди могат да бъдат сдвоени или в краен случай подменени.

5.1. Анкериране на подовите греди към каменната стена на емпорето

Анкерирането на подовите греди към каменната или тухлена стена се изпълнява чрез метални шини, закрепени към гредите отстрани с помощта на болтови съединения БМ16. Към шините се заваряват болтове, които се прокавват през отвори в стената и свързват със стоманени закотвящи плочи, които след това се притягат към стените, чрез навиване на гайките върху шпилките, вследствие което се реализира еластична връзка между подовата греда и стените на емпорето (фиг. 5.1а,б; фиг. 5.2а,б; и фиг. 5.3а,б,в;). Тази интервенция подобрява поведението на подовата конструкция при хоризонтални сеизмични въздействия и представлява рутинна операция при реконструкция на подобни подови конструкции в исторически обекти.



Фиг.5.1 а,б. Закотвяне на столците към външния зид по ос 8 чрез стоманени шини

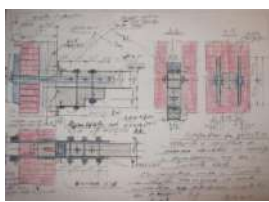


а)

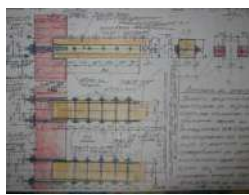


б)

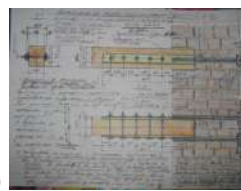
Фиг. 5.2а,б. Закотвяне на столицата към външния зид по ос 1 при колона К1Б чрез стоманени шини, пристягане на пакета: столица и възглавница чрез болтови съединения



а)



б)



в)

Фиг. 5.3а, б, в. Анкерирание на подовите греди към каменната стена

6. Заключение

Тази статия представя литературен обзор върху анкерни закрепвания между стени и дървен под в сгради на URM, като се имат предвид съществуващите традиционни връзки и основните укрепващи решения в експериментални и числени изпълнения. Въпреки обичайното използването на системи за анкерирание в съществуващите сгради на URM като ниско-инвазивни антисеизмични устройства и други, много инженери използват неподходящи решения, които могат да променят оригиналната естетическа и структурна концепция на историческите сгради. Освен това, инвазивни материали като RC често се използват за замяна на оригиналните такива, поради тяхната наличност. Укрепването на връзките с помощта на съвместими материали (като дървен материал и други) може от своя страна да бъде по-устойчива алтернатива за възстановяване на безопасността на сградата, като се използват иновативни материали, като стомана или нови смоли, на местно ниво и поддържане на оригиналните външни и вътрешни стени от исторически аспект. Отчитането на връзките стена-под при анализа на URM изглежда е от решаващо значение за оценката на съществуващи и усилен сгради. Експерименталната характеристика на единичния тип връзка е предизвикателство, но е необходимо, за да бъде изпълнен надежден модел. Голям набор от параметри може да бъде изследван чрез параметрични числени анализи на калибриран модел. Линейният и нелинейният кинематичен анализ изглежда най-използваният като прост и мощен инструмент за оценка на локалните механизми. Освен това подходът на люлеене отчита еволюцията на движението във времето и дисипацията на свойствата на пластичната става, около която се върти зиданата стена не се пренебрегват. Такива процедури се основават на хипотезата за монолитни блокове, което не винаги е вярно за зидани стени или фасади. Въпреки че FEM анализът е най-използваният сред практикуващите като бърз и прост, той се основава на несигурни механични характеристики на зиданите елементи, поставяйки границите на метода.

7. Благодарности

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SEISMIC DAMAGE OF BUILDING STRUCTURES AND THEIR REHABILITATION

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Abstract: *Building's structures were designed and built according to different standards that were valid at the time of construction. Those standards were, more or less, less demanding than the current European, American and other modern standards. Serious earthquakes events cause damage of the elements, with different level, even their collapse. It is very important to save damaged or weakened structures by emergency strengthening, and to perform rehabilitation of their seismic resistance. The method of evaluation of seismic resistance of damaged elements and structures are discussed. After evaluation of the damage degree and determination of required and available seismic resistance of structures, redesign must be performed. Also, method of rehabilitation of the elements and entire damaged building structures are analysed, including both traditional and advanced techniques. The focus of this paper is on reinforced concrete (RC) buildings, and less on masonry and mixed structures. Post-earthquake assessment and analysis of seismic performance are analysed. Review and comparison of international Code provisions related to topic are discussed as well. Criteria for the choice of structural interventions are discussed, as well.*

Key words: *Earthquake damage, assessment, condition, performance, seismic resistance, rehabilitation*

1. Introduction and terms

Depending on structures' condition after earthquake, there is a need for structural upgrading in order to meet design requirements. International Federation for Structural Concrete (*fib*), has placed considerable emphasis on assessment and rehabilitation of existing structures. Document [11] is related to assessment and retrofitting of buildings. In the US, different organizations prepared particular documents for seismic evaluation (FEMA 178), for evaluation of the buildings with the wall structural system (FEMA 306).

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Both groups, of concrete and masonry buildings, are more or less damaged in earthquakes, depending on the strength of the earthquake and the quality of the buildings. Masonry buildings are significantly more vulnerable to seismic effects, so they experience a lot of damage and even collapse. Inspection of buildings after an earthquake allows one to look at failures in design and construction, the state of the structure and the severity of damage, and evaluate the possibilities of retrofitting. It is very complex process because of the unpredictable behaviour of earthquakes. Among them are historical buildings that belong to the architectural heritage and require special treatment to improve seismic performance, because it is imperative to reduce the loss of their historical and architectural value [8]. This requires an adequate methodological approach to the analysis, aligned with the importance of the building's function. The first step is to investigate the material characteristics of the structural elements in situ or in the laboratory. The second step is the simulation of linear and non-linear behaviour, usually using the finite element method (FEM) in order to determine the behaviour mechanism, i.e. seismic performance. The rehabilitation phase is conditioned by inadequate performance, especially insufficient seismic resistance [52].

Damage is any adverse consequence for the physical state of a structure or structural component caused by earthquake. *Assessment* comprises the process of gathering and evaluating information about the form and current condition of a structure or its components. *Repair* is process of reconstruction and renewal of the existing buildings, either in whole or in part, while *renovation* is substantial repair or alternation that extends a building's service life. The term *repair* comprises restoration of the initial mechanical characteristics in order to remove the consequences of adverse-seismic events. *Strengthening* includes interventions made to increase the load resistance of a components or to improve overall structural stability and the overall robustness of the structure; or shorter *strengthening* is Interventions to increase the bearing capacity of a structure. *Retrofitting* is an action aimed at modifying the functionality or form of a structure or its components and improving of future performance (minimizing damage during specified earthquake). *Upgrading* is the action undertaken in order to improve the quality of a building, including the increase of capacity to withstand earthquakes. *Replacement* is an action of providing new substitute components for the ones which have experienced deterioration, damage, degradation or mechanical wear[6], [19] and [22].

Sometimes the term *rehabilitation* includes interventions undertaken in order to restore the performances of a structure or its component parts that are in changed to the original level of performance, including all types of repair, retrofitting and strengthening that lead to reduced earthquake vulnerability. *Reconstruction* is a radical intervention, which includes improvements - upgrading/retrofitting, strengthening, replacement of components, and the removal of all or part of a structure, if it necessary to achieve a requested performance. Seismic retrofit comprise: technology of application; experimental data on retrofitted members; engineering models and expressions for the retrofitting design; recommendations for the conceptual and the detailed design of the retrofitting. Retrofitting strategies and instructions for selection of the retrofit technique also included [16]. Besides strengthening, increasing ductility was adopted in order to upgrade seismic performance of the structures and minimise damage during earthquake (retrofit). Sometimes, the criterion on the evaluation of seismic safety of existing RC Buildings, and their aseismic retrofitting is very important.

Architectural Heritage - Buildings and complex of buildings (towns, etc.) of historical value. *Adobe* is bricks made from clay and simply dried in the sun. Some organic materials like straw or animal excrement can be used to improve durability or reduce shrinkage. *Conservation* – Operations which maintain the building as it is today, even if limited interventions are accepted to improve the safety levels. *Diagnosis* n. - The act or process of

identifying or determining the nature and cause of damage and decay through, observation, investigation (including mathematical models) and historical analysis, and the opinion derived from such activities. *Heritage Value* - Architectural, cultural, and/or historic value ascribed to a building or site. *Heritage value* may have varying definitions and importance from culture to culture. *Historical Approach* - Evaluation based upon historical research and past experience. *Holistic* adj. Emphasizing the importance of the whole and the interdependence of its parts. *Structural Typology* - The types of structures interpreted as regards their structural behaviour and their capacity to bear loads. *Safety Evaluation (assessment)* - Evaluation of the safety margins of a structure with regard to heavy damage, partial or total collapse. *Restoration* - Process of recovering the form of a building as it appeared at a particular period of time by means of removal of additional work or by replacement of missing later work [22].

This paper provides an overview of the literature, on the behaviour of historic masonry elements and building, as well as publications related to the behaviour of subassemblies and building on earthquake, and some Code related to behaviour and damage in structural element and structure of concrete and masonry buildings. Considered in in-plane shear and simultaneous compression, out-of-plane bending. In doing so, the most frequent damages are analyzed with the indicated causes of their occurrence. The method of evaluating the state of damaged structures, forecasting the remaining service life, and related to that the selection of appropriate interventions aimed at removing damage and/or strengthening the structure was discussed. The frequent construction type, especially historical buildings (built heritage), are obtain in discussion. At the same time, the difference between the intervention of a seismically damaged structure (balance strength, ductility and stiffness) and a structure damaged by some other effects was emphasized. Some of the most common intervention methods were analyzed in order to achieve the desired level of seismic performance, taking into account the state of the construction and the cost-effectiveness of the interventions. The advantages and disadvantages of certain interventions are indicated.

2. Damage and assessment condition of buildings structure

Concrete and masonry building structures behave differently under seismic effects. Their behaviour depends on project solutions and the quality of building construction and maintenance. The evaluation of seismic safety of existing RC buildings is very important before structural redesign for earthquake resistance. In high-seismicity area rehabilitation of buildings must provide ductile instead of brittle behaviour [18]. In design frame structures, damage is likely to occur first in beams then in columns, and columns in frame should be stronger than beams, and foundations should be stronger than columns. It is very important to provided rigid diaphragm action (floor slabs stiff in own plane).

Using the information collected from site observation, performed in order to determine the residual strength, stiffness and ductility of the structure. For estimation the residual strength, stiffness and ductility of a structure, the structural engineer (SE) has to trace the damage in the structural system and non-structural. Crushing of concrete or part of masonry at the top or the bottom of a column or walls accompanied by buckling of the longitudinal reinforcement, X shaped cracks in shear walls with significant axial loading and in short columns are some of damage examples [41].

Concrete structures (CS) can become deficient due damage caused by earthquakes, especially old buildings which are not design according to contemporary codes. Wide classification of concrete structures and mechanisms of failure was discussed in [18] and in

[17] for different structural systems of building..Data relevant to classification can be classified into following groups:

1. Identification parameters;
2. Structural and quality parameters;
3. Damage and usability parameters.



Fig. 1. Poor detail of transversal reinforcement

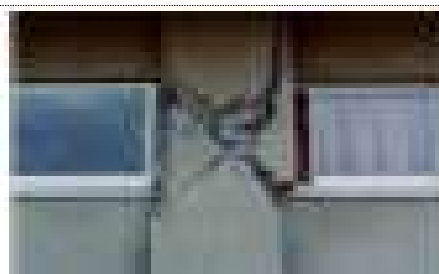


Fig. 2. Shear and axial failure of column due to partial-height infill walls



Fig. 3. Collapse with column shear and axial failure. Koceali Eq. 1999.



Fig. 4. Column failure due to excessive axial load and insufficient stirrups



Fig. 5. Beam-column joint damage, Turkey, Koceli, 1999, after [17]



Fig. 6. Bursting failure in a column, Northridge, California, 1994, after [5]

According to the influence of damage on the global stability of structures, damage can be classified into: slight, moderate, serious and collapse. Serious damage includes: the occurrence of plastic hinge, strong beam-weak column, pounding of adjacent buildings, short beams and short columns, partial infill walls created short columns and collapse near joints as brittle mechanism. Greater damage accompanied by reinforcement bar buckling (inadequate transverse reinforcement) or breaking and concrete crushing appears in the case of considerable overloading caused by seismic action, poor concrete quality, infill wall damage (diagonal and X mode of damage) etc.[18].

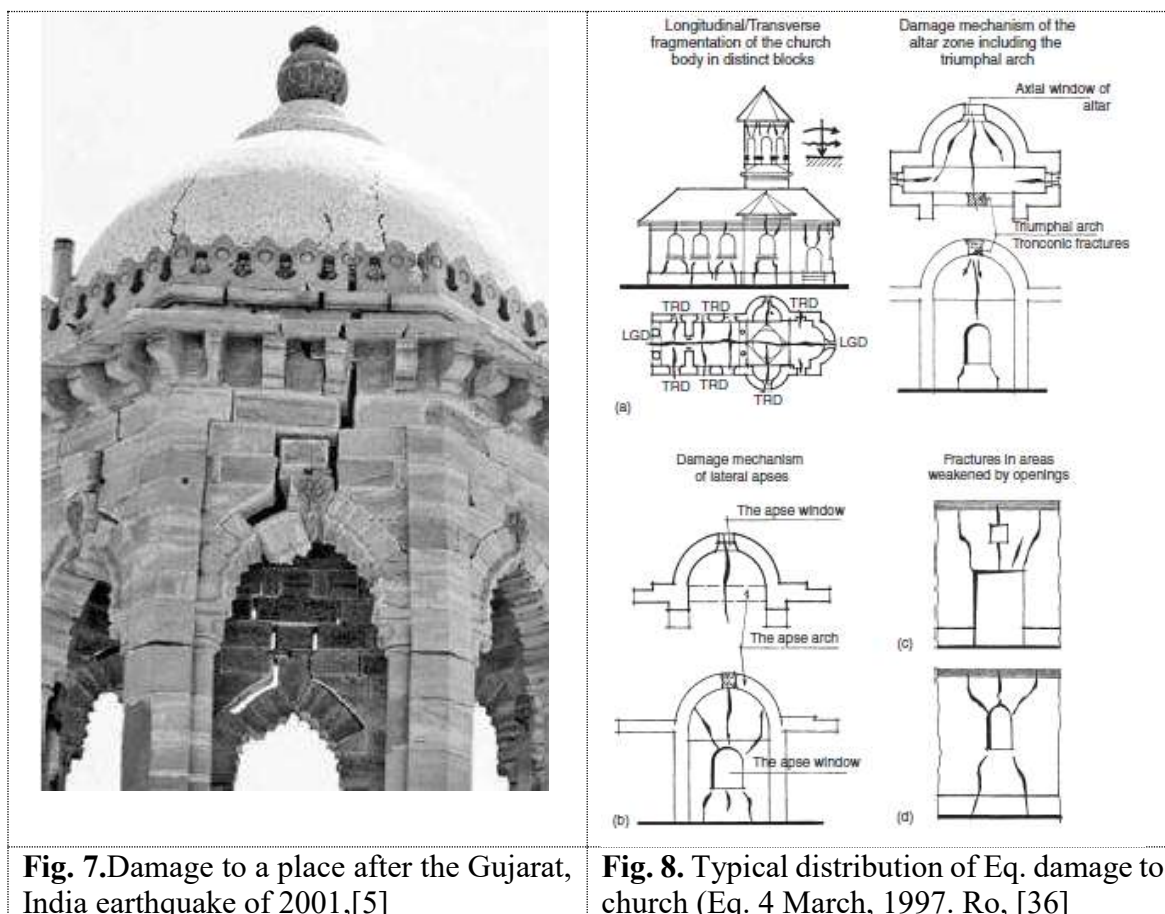


Fig. 7.Damage to a place after the Gujarat, India earthquake of 2001,[5]

Fig. 8. Typical distribution of Eq. damage to church (Eq. 4 March, 1997. Ro, [36])

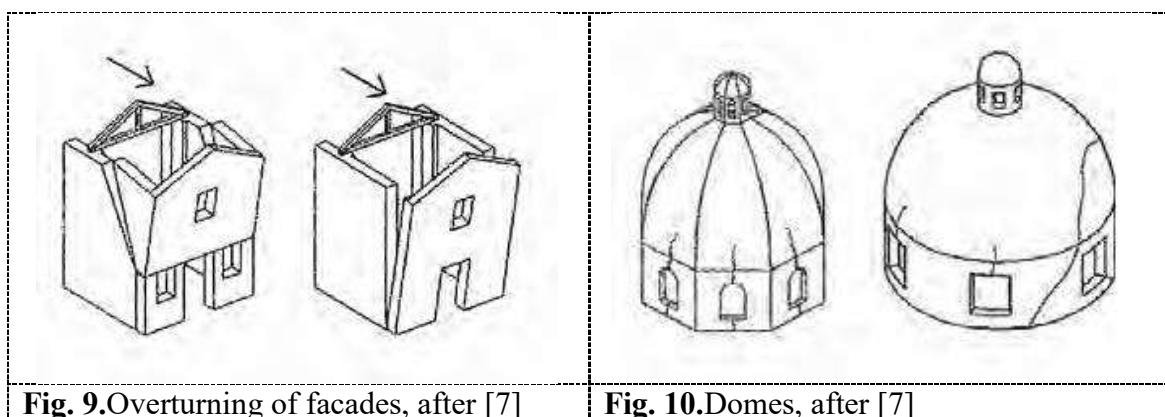


Fig. 9.Overturning of facades, after [7]

Fig. 10.Domes, after [7]

Weakness to shear and a lack of adequate transverse reinforcement were evident in many examples. This induces the crushing of concrete and the buckling main reinforcement. Weak column – strong beam in frame RC structures caused a failure of the column below the frame joint, i.e. unconvinced mechanism, soft and weak stories. Generally, insufficient

detailing of the structural elements is another reason of damages. Short column, hammering, unconfined gable wall and in plane/out-of-plane movement of the walls caused damage [53]. Failures of both reinforced and unreinforced masonry in-plane are common. That's why it's significant to evaluate the residual seismic capacity concrete and masonry damaged structures. Experience shows that non-structural items which are suspended, such as ceiling systems and light fittings, perform badly. Appendages such as parapets also suffer high levels of damage.

While complete replacement of a deficient structure is a desirable option, strengthening/repair is often the more economical one and hence the aspect of renewal has received, especially historic buildings, considerable attention over the past few years throughout the world [2].



Fig. 11. Severe damages to shear walls and spandrels 2009 Eq. L'Aquila, Italy, [22]

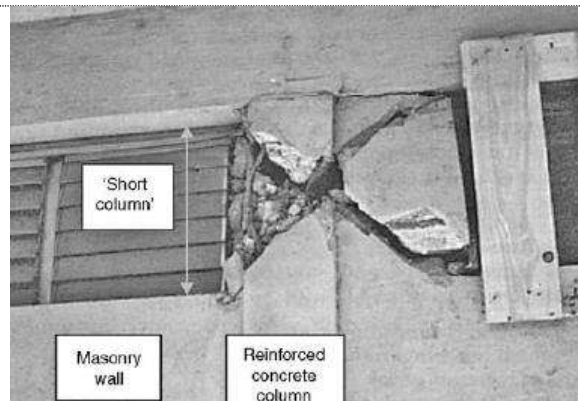


Fig. 12. Shear failure of a lightly reinforced concrete column and the adjacent masonry, opposite a window opening in St Johns, Antigua, 1974, [5]

It should be borne in mind that brick buildings are significantly more vulnerable especially unreinforced masonry (URM) buildings. Masonry is one of the oldest fundamental materials used in construction exhibit poor performance seismic actions, weak under lateral loads and tension forces. Structural and geometrical typologies, variability of constituent materials, construction methods, connections, intricate configuration, and cultural and artistic specifications are the influential factors in masonry behaviour, particularly under seismic loads [49].

Older buildings also commonly have timber floors with joists that are poorly tied to the supporting walls. Any lack of tying together in a building is quickly exposed by earthquake shaking. In plane, masonry is very stiff, so that the forces transmitted by ground shaking are high; masonry is also brittle so that failure is accompanied by a marked reduction in strength and stiffness. Damage normally comprises either collapse or diagonal cracking in both directions and damage will often be worse around openings.

Characteristic damage are: Cracks between walls and floors; Cracks at corners and wall intersections; Out-of-plane collapse of the perimeter walls; Cracks in spandrels beams and/or parapets; Diagonal cracks in structural walls; Partial disintegration or collapse of structural walls. Evaluation of URM buildings under seismic conditions for both experimental investigation and numerical simulation requires accurate information [51].

Horizontal reinforcement laid in the mortar bed joints substantially increases in plane strength and ductility, as does vertical reinforcement in mortar columns cast into hollow blocks. Introducing masonry tie-beams and columns into masonry walls to form smaller confined panels also substantially improves in-plane resistance. Out-of-plane, free-standing

masonry or masonry that has separated from any adjacent structure is liable to toppling failure.

3. Analysis and evaluation conditions of building structure

Depending on structures' condition after earthquake, there is a need for structural upgrading in order to meet design requirements. International Federation for Structural Concrete (*fib*) has placed considerable emphasis on assessment and rehabilitation of existing structures [15]. An overview of post-earthquake damage and determination the residual capacity for RC buildings in Japan is subject of the paper [33]. Yet, to consider the determination of lateral strength and ductility corresponding to the damage state of each lateral – load resist member. In the damage evaluation Guidelines (JBDPA 2015) the state of damage of each structural member is classified into one of the five classes [33]. Wide post-earthquake evaluation and strength assessment for Balkan Country discussed in book [6]. In USA are used documents ACI [2-3], and FEMA [12] and in Europe [11], and before mention *fib* [15] was used. In Japan, beside [33], [28] and [39] are used. For seismic calcification of structural damage, damage index recommended in [45], and for masonry building [49] and [53].

In U.S. Vision 2000, which gives the numerical definition of four intensity levels in terms of their return period or probability of exceed (*frequent* 50% in 30 years), occasional (50% in 50 years), rare (10% in 50 years), and very rare (5% in 50 years) is still used Fig.13, cited in [19]. Performance levels and relative damage are shown in Fig. 14, [16].

Seismic behaviour and resistance deficiencies during a seismic event, the global behaviour of a masonry building are greatly influenced by the type of connection between walls and slabs. A good connection avoids out-of-plane brittle failures and mobilizes the in-plane shear resistance of walls – “box behaviour”. Shear failures may occur in three different types: Rocking, Sliding and Diagonal Cracking. Mixed masonry-concrete buildings (and other masonry structures) generally have insufficient strength for in plane and out-of-plane seismic actions.

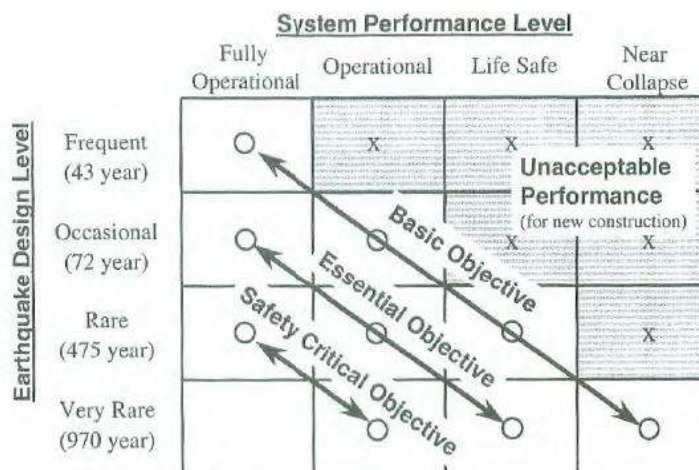


Fig. 13 Performance matrix 2000 vision

For historical buildings, the methodology proposed by ICOMOS is generally used, which shown in Fig. 15. In beginning Data acquisition (content historical investigation, field and monitoring); after structural behaviour, follow diagnosis and safety with explanatory report, and remedial measures with execution documents.

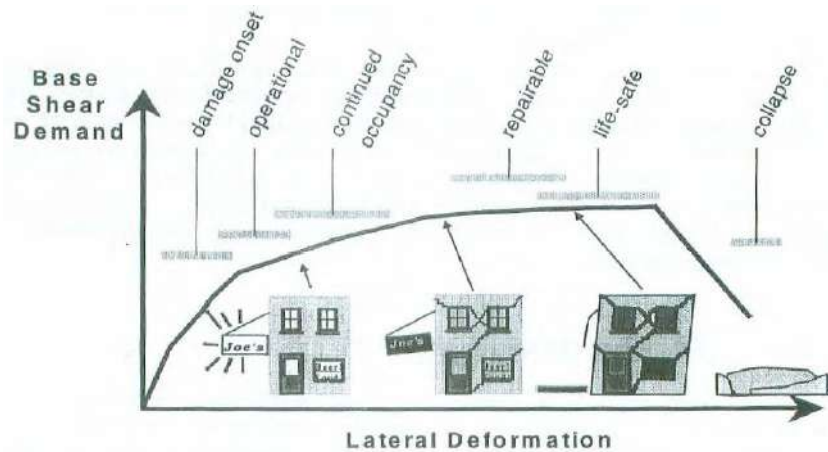


Fig. 14. Performance levels and relative damage, after [16]

Due to the great variety of structural configurations and materials used, rational approaches for the assessment of the seismic safety levels of such buildings are needed. A three-storey masonry building was considered with aspect of applicability of two contemporary seismic strengthening measures: the use of fibre-reinforced polymer (FRP) composites and the implementation of base-isolation to achieve the seismic protection levels. Non-linear static analyses (NSA) for different levels of seismic intensities were conducted on a model of the fixed-base [31]. Also microelement modelling technique for URM structures can use. The techniques, materials and design procedures in use for the structural restoration of historical buildings and monuments, damaged by earthquakes were outline in [42].

During the structural restoration, the original documents from earlier periods of construction are respected. At some time the use of traditional techniques and materials is clearly preferable. Stressed that modern techniques and materials are admissible where adequate capacity cannot be ensured by traditional techniques. Measures are necessary to protect and safeguard fresco and mosaic decoration excluded the use of some strengthening techniques that may cause damage. Analysis by means of FEM with microelements for mortar and bricks are used. The behaviour factor necessary for a quasi-static or dynamic-elastic analysis of the structure cannot be taken from the codes of practice for masonry structures. Bending combined with shear deformation has to be taken into account [21].

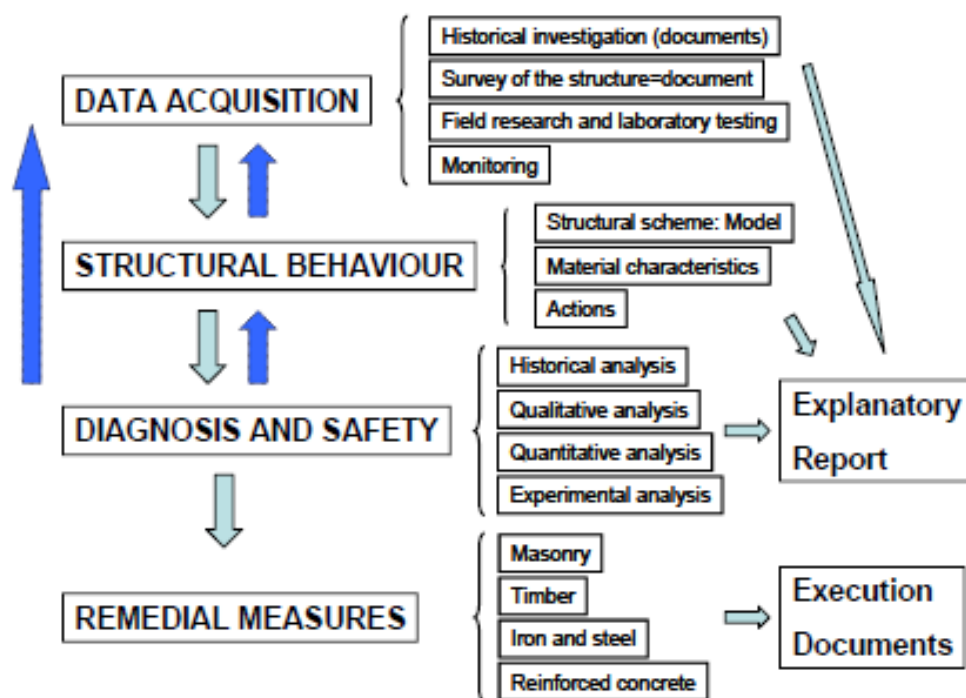


Fig. 15. ICOMOS methodology

After minor shaking, the building will be fully operational within a short time and the repair costs will be small. After moderate shaking, the building will be operational once the repair and strengthening of the damaged main members is completed. But, after a strong earthquake, the building may become dysfunctional for further use, but will stand so that people can be evacuated. In order to determine the residual strength, stiffness and ductility must using the information collected from site observation the damage in the structural system and non-structural elements. The nonlinear static evaluation procedure is known as the NSA method. Idealized roads of beam column concrete can be considered as: concentrated plasticity- plastic hinge or nonlinear spring hinge; and distributed plasticity with finite hinge zone, fibre section and finite element [16].

Modern European codes (EN 1998-1 and 1998-3, 2004) provide the rules for seismic design of new and existing damaged structures, respectively. Federal emergency management agency (FEMA) and applied technological council (ATC) work very actively in assessing conditions of structures after seismic events and for its upgrading. Current seismic design of structures is based on the presumed ductile response and yielding probability of some part of a structure, without transforming the structure into a mechanism of brittle failure. The Code provisions generally have two levels, limit state design, for earthquake resistant of buildings [52].

In the seismically active zones of southern and central Europe, unreinforced masonry (URM) structures are one of the most common types of buildings. Some of them possess a high historical value and could therefore be classified as part of the architectural heritage, requiring special attention with regard to their preservation and retrofitting measures [31].

Due to the great variety of structural configurations and materials used, rational approaches for the assessment of the seismic safety levels of such buildings are needed. The paper analyses the applicability of two contemporary seismic strengthening measures, namely the use of fibre-reinforced polymer composites and the implementation of base-isolation to achieve the desired, code-based seismic protection levels. A three-storey masonry building was considered in the study. Non-linear static analyses for different levels of seismic intensities were conducted on a mathematical model of the fixed-base, FRP-strengthened and base-isolated variants of the structure [31].

Setting a criterion on the evaluation of seismic safety of existing RC buildings is very important before structural redesign for design earthquake. Approach to the rehabilitation problem of a damaged building should be done in the following steps:

- Examination of a damaged building; development of alternative rehabilitation schemes;
- Examination of the feasibility as well as cost of each alternative;
- Final rehabilitation design.

Depending on the intensity of the earthquake and its effects, it affects the cost and duration of rehabilitation. Levels of Performance (Limit States in the EN) and Hazard Levels (Seismic action) of [11] adopts a fully performance-based approach for the evaluation and rehabilitation of existing buildings (Fig. 7): 1) *Near Collapse* (NC), the *Collapse Prevention* in US codes. The structure is heavily damaged, retains little residual strength and stiffness against lateral loads, but vertical elements can still carry the gravity loads. Most non-structural elements have collapsed; 2) The “*Significant Damage*” (SD) level, corresponding to the “*Life Safety*” (in the US) and to the local-collapse prevention performance level for which new structures are designed according to Part 1 of EN 1998. The structure is significantly damaged, but retains some residual lateral strength and stiffness and certainly its vertical load-bearing capacity. Repair may be uneconomical. 3) The “*Damage Limitation*” (DL) level, “*Immediate Occupancy*” in the USA[13]. The structure is only slightly damaged, without permanent deformations and do not need repair. Performance requirement Description Limit State of Near Collapse (NC). The structure is heavily damaged, with low residual lateral strength and stiffness, although vertical elements are still capable of sustaining vertical loads. Most non-structural components have collapsed. Large permanent drifts are present. The structure is near collapse and would probably not survive another earthquake, even of moderate intensity.

Limit State of Significant Damage (SD) - with some residual lateral strength and stiffness, and vertical elements are capable of sustaining vertical loads. Non-structural components are damaged, although partitions and infill have not failed out-of-plane. Moderate permanent drifts are present. The structure can sustain after-shocks of moderate intensity. The structure is likely to be uneconomic to repair. Limit State of Damage Limitation (DL) - only lightly damaged, with structural elements prevented from significant yielding and retaining their strength and stiffness properties. Non-structural components, such as partitions and infill, may show distributed cracking, but the damage could be economically repaired. Permanent drifts are negligible. The structure does not need any repair measures.

In USA Overall damage; Collapse Prevention(CP) level; Severe; Life Safety (LS) level; Moderate Immediate Occupancy(IO) level; Light Operational level Very light General Little residual stiffness and strength, but load bearing walls function. Some exits blocked. Building is near collapse. No out-of plane failure of walls or tipping of parapets. Some permanent drift and damage to partitions. Building may be beyond economical repair. No permanent drift.

Negligible damage occurs. Power and other utilities are available, possibly from standby sources. Comparison with performance intended for (new) buildings under the NEHRP provisions, for the design earthquake Significantly more damage and greater risk. Somewhat more damage and slightly higher risk. Less damage and lower risk.

For analysis and evaluation conditions of tructure used software packages. However, the calculation of masonry buildings is very complex. Masonry is an anisotropic, heterogeneous, and non-linear composite material that has been widely used in developing countries [51]. Although the response of the walls is likely to become highly non-linear

(have cracked), a linear static or response spectrum analysis using force reduction factors is likely to give sufficient information on these points in well-designed buildings of regular form. Non-linear static or dynamic analysis may be employed to investigate redistribution of forces between walls in existing buildings, where one or more walls reach their ultimate strength for a seismic intensity below that considered for design on model [30], Fig. 16.

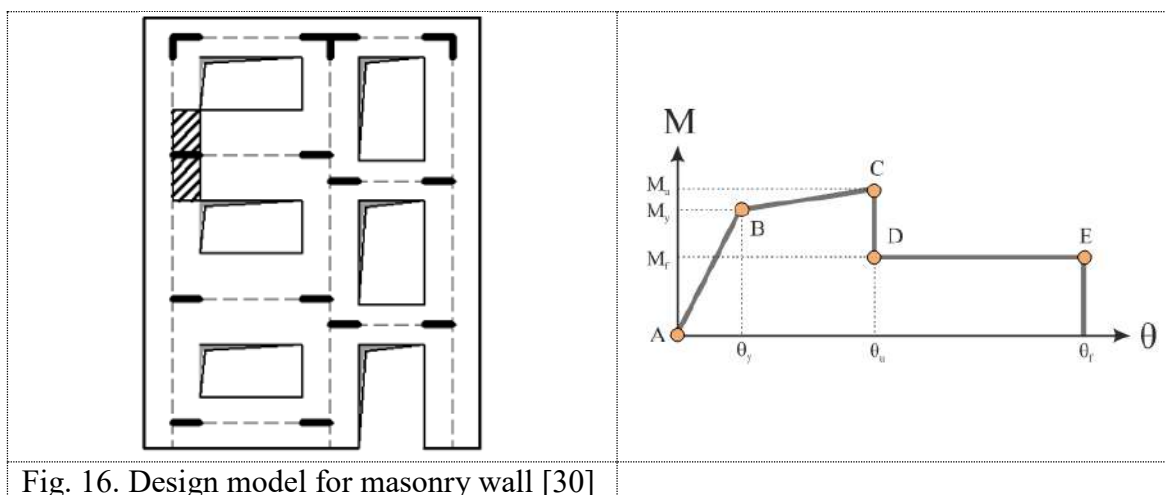


Fig. 16. Design model for masonry wall [30]

The flow diagram for the evaluation of the disease and the support of the diagnosis and the determination of the measures of therapy is attached in fig. 17.

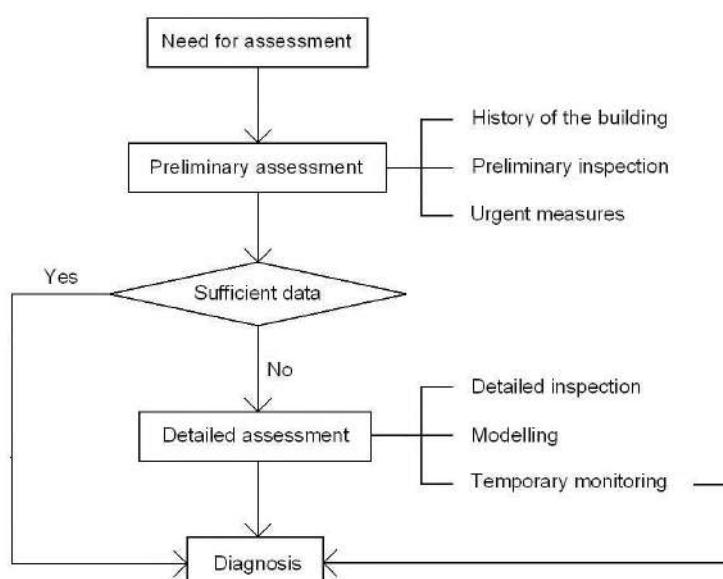


Fig. 17. Flowchart of the actions to be undertaken for the diagnosis of the building [22]

The relationship between strategies and measures applied in rehabilitation is shown in the flow chart in the figure 18. Flowchart of the actions to be undertaken in the phase of the structural assessment of the structure of building and make decision between repair and strengthening show in Fig. 19, and typical seismic strengthening methods for provide strength and ductility of structures in Fig. 20.

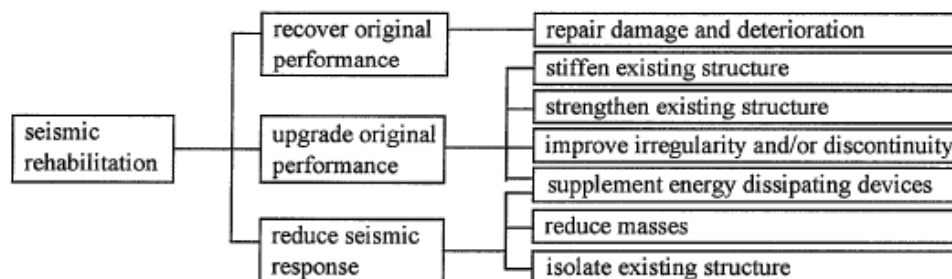


Fig. 18. Seismic rehabilitation strategy and measures, adopted from [51]

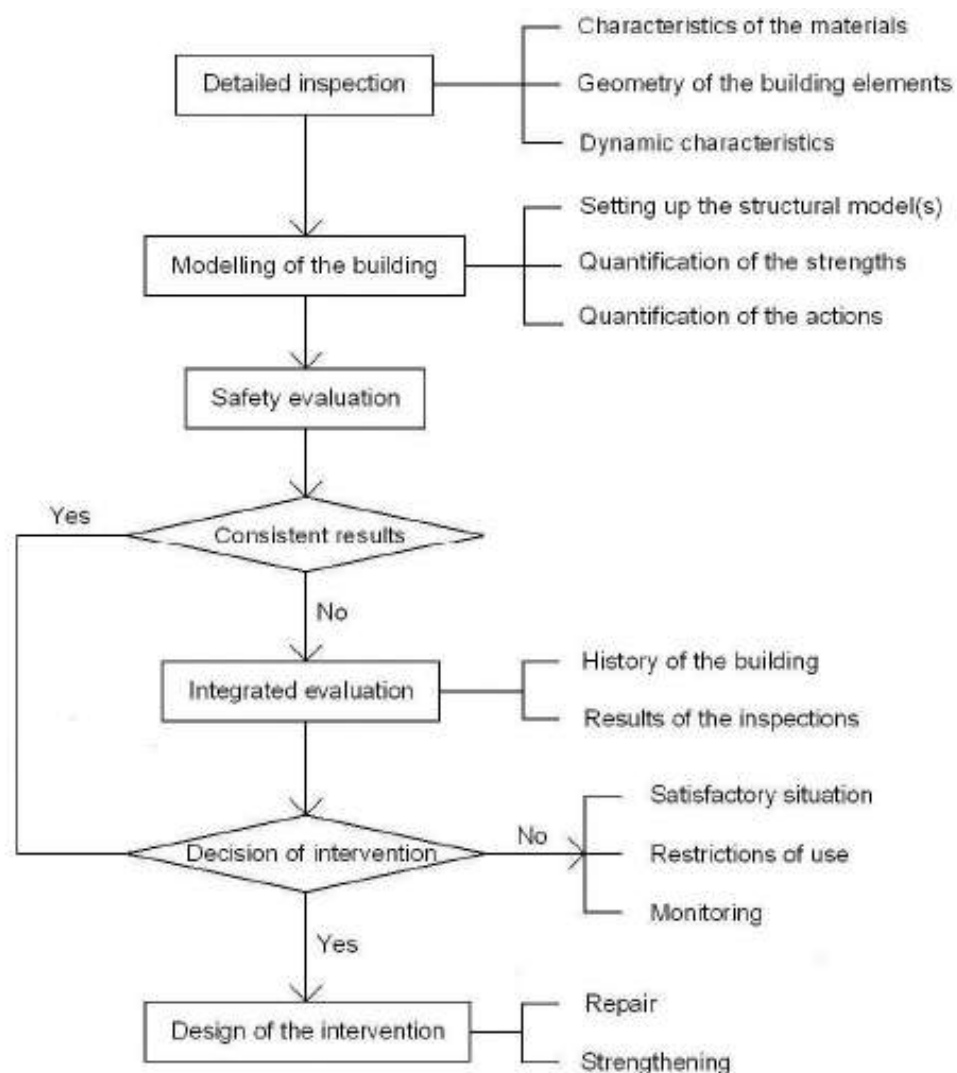


Fig. 19. Flowchart of the actions to be undertaken in the phase of the structural assessment of the building [22]

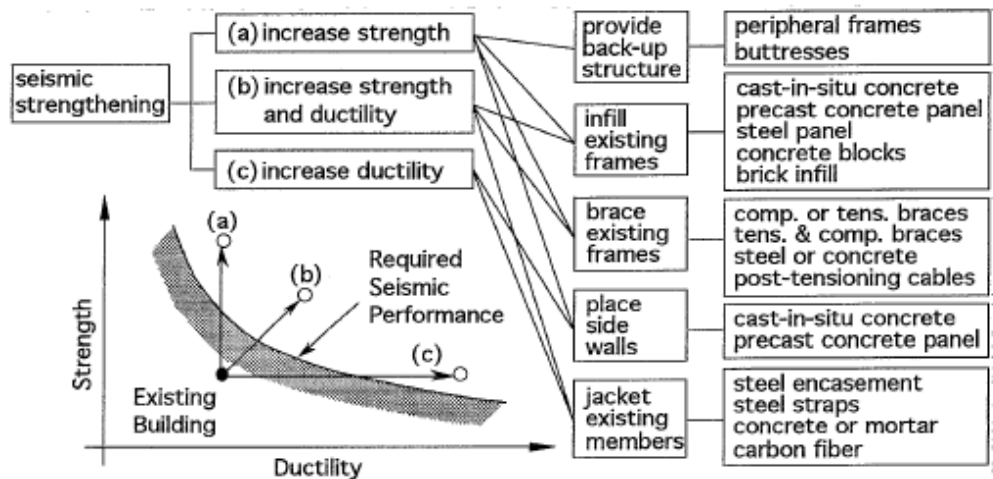


Fig. 20. Typical strengthening methods [22b]

For historical buildings, i.e. architectural heritage overview of the procedure for selection of retrofitting technique showed in Fig. 21, adopted from [48] and [50].

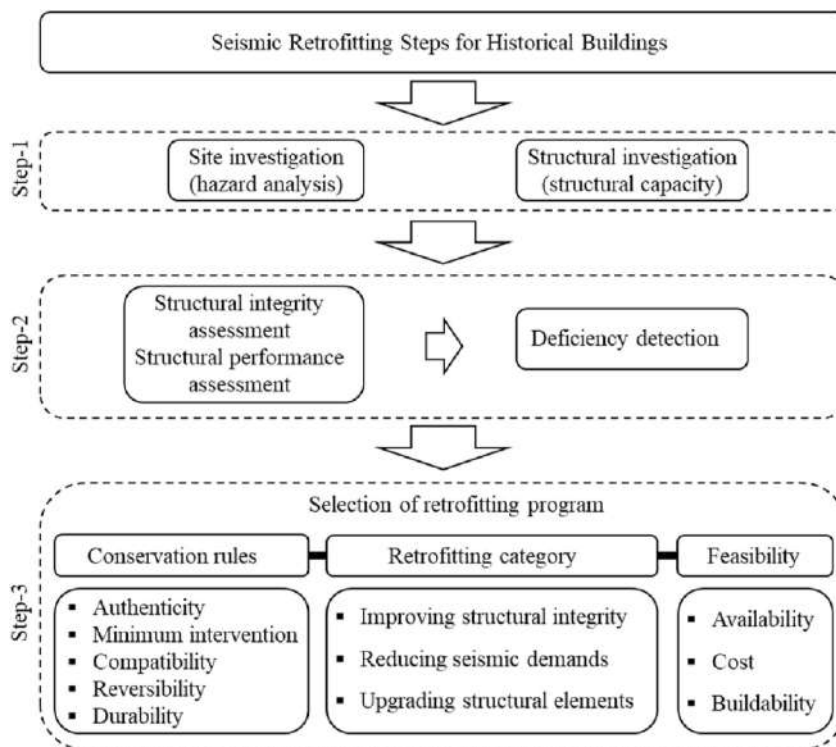


Fig. 21. Overview of the procedure for selection of retrofitting technique for heritage buildings

The diagram shown in Figures 17 to 21 can be useful for making adequate decisions about the type and scope of interventions on the structures of concrete and masonry buildings. Buildings of architectural heritage are also included.

In *seismic rehabilitation*, analyses are made to estimate the safety and performance of the building prior to and after rehabilitation. The knowledge factor is used in the evaluation of the capacity of the cross sections and of elements to account for possible uncertainties.

Frequently level of knowledge is assumed to be limited factor is 0.75; for medium 0.90 and 1.00 for comprehensive level [11].

The performance goal of the strengthening needs to be defined. Is it merely to restore the building to its condition before it was damaged by the earthquake, or is it to be upgraded and if so to what level? Damaged members need to be reinstated at least to the level that they can safely carry their gravity loads; otherwise they must be replaced. Grouting under pressure of cracks in concrete and masonry with epoxy mortar is a well-established technique which is quite reliable in reinstating the concrete to its previous capacity. Jacketing and plating, as previously described, can also be used to reinstate and strengthen damaged elements. Additional elements can then be added to take the demand away from inadequate members under conditions of earthquake loading.

4. Rehabilitation/structural intervention on structures of buildings

4.1. Some rehabilitation of RC and masonry structures of buildings

Initial considerations may include the following.

- (a) Damaged elements are likely to need repair or replacement.
- (b) Irregularities in plan leading to torsional response (with an adequately arranged shear walls). Adding shear walls or cross-bracing reduce the torsional eccentricity.
- (c) Irregularities in elevation giving rise to soft or weak storeys should be eliminated.
- (d) Lack of tying together of a building may well be a concern in masonry buildings.
- (e) Non-structural elements need protection (cladding elements, chimney, parapets, i.e.).
- (f) Geotechnical aspects should be addressed (possibility of slope instability at or near the site, and foundation movements).

General criteria for the choice of a structural intervention: cost, both initial and future; durability of original and new elements, possibility quality control, materials; aesthetics and duration of work. *Technical criteria* guidelines for the choice of the intervention scheme: possible increases local ductility; and spreading the areas of inelastic behaviour as much as possible across the entire structure should be one task of intervention. Mass reduction, addition of new structural elements; base isolation; addition of new elements (bracing or infill walls) may be selected [41]. Braced frames are generally considered to be stiff systems in the elastic range. Their nonlinear response depends on their ability to redistribute forces between bays and drifts between stories. Important considerations for upgrading the seismic resistance of existing including investigation of existing structural characteristics, identification of significant deficiencies, and selection of appropriate upgrade criteria and retrofit systems are given in [23]. A review of strengthening schemes for RC buildings was shown in [46].

To prevent disaster in the future earthquakes, the existing deficient buildings need to be retrofitted. The basic requirements of rehabilitation and investigations of various retrofit techniques should be considered before selecting retrofit schemes. One way of retrofitting of the columns in RC multi-storeyed buildings is concrete jacketing [19] and [44].

Furthermore, methods are categorized based on the nature of the structure, application, and possible availability. A comparison of each method's advantages and shortcomings is presented in terms of economy, sustainability, and buildability. Disruption to the continued occupation of a building is important. The transformation of the existing RC walls into anti-seismic shear walls is presented in [35]. In [37] state of research on seismic retrofit of concrete building structures (typical configurations and details) in the US is given.

New methods are often based on fiber reinforced polymers (FRPs) as new materials for intervention. Confinement of columns with composite materials leads to increase of

lateral deformations. Deficiencies [53] have been identified during the seismic assessment of the building may be modified for improved earthquake resistance and/or deformation capacity within the seismic upgrading (such as the addition of shear walls [20]). Concrete, steel or fiber reinforced polymers (FRP) are normally used for the jacket. Steel and FRP jacketing are very popular in many countries since they have the advantage of a minimal increase of cross sectional dimensions, can be executed quickly with little interruption to the use of the structure while work is carried out [19].

Focus during the last few decades has been on seismic retrofitting and strengthening techniques using composite materials, both RC and masonry buildings structures. This review attempts to systematically investigate URM structures and commonly used retrofitting methods based on type and previous studies. Some examples for intervention on RC structure are shown in Fig. 22, for strengthening RC columns, for special details with anchorage and strengthening top and bottom of columns Fig. 23, and for column-beam joint on Fig. 24.

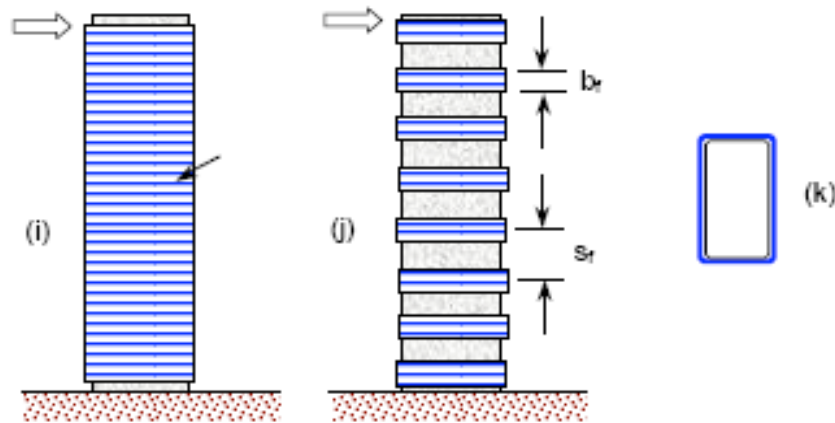


Figure 22. Schematic illustration of RC element strengthened in shear with externally bond: i) complete FRP wrapping; j) discrete strips of FRP; k) four sided wrapping of columns as i) and j), after [38]

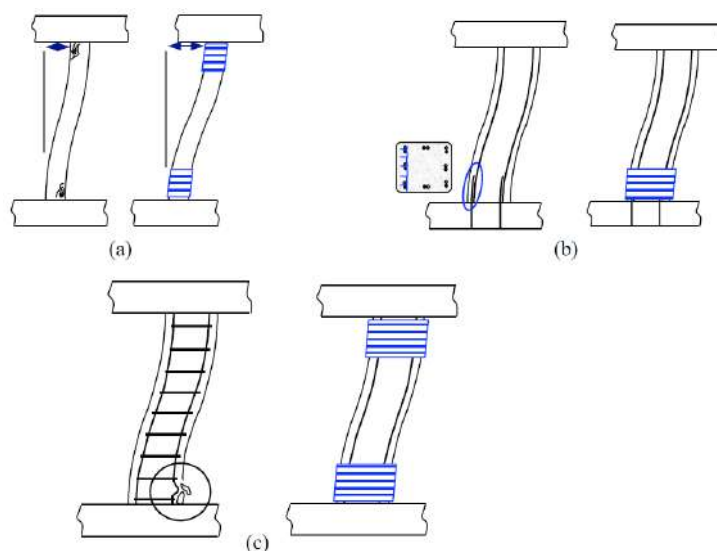


Fig. 23 Confinement of RC element with composite materials to: a) increase lateral deformation; b) prevent lap splice failure; c) delay rebar buckling

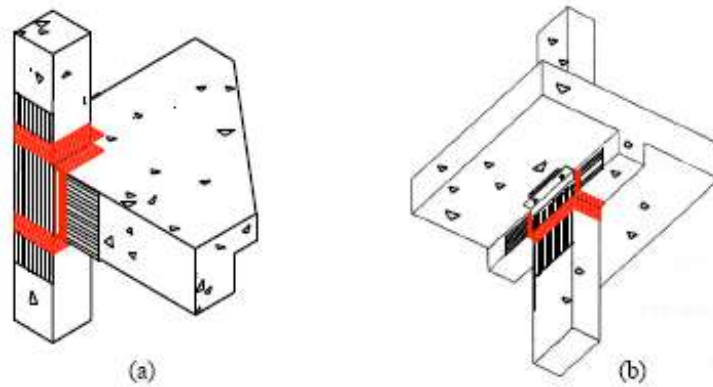


Figure 24. Typical composite material configurations for beam-column joint strengthening, a) external, b) interior joints [35]

Comparative strengthening columns with FRP a) jacketing; and b) steel cage technique using steel straps or c) steel plates showed in Fig. 25.

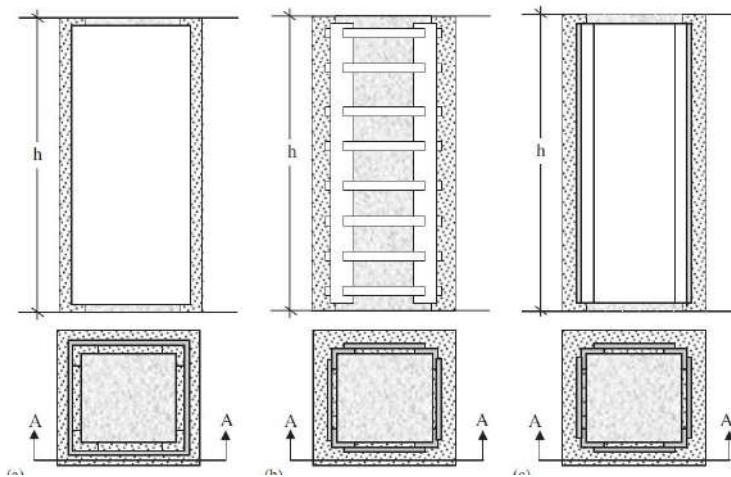


Fig. 28 Strengthening columns: a) jacketing; b) steel cage technique using steel straps or c) steel plates, after (Thermou and Elnashai, 2006) cited in [19] and [35]

Insufficient bearing capacity of slab structures can be strengthened by increasing its thickness and by placing additional reinforcement. The integration of the existing and the new layers of slab, i.e. the force transfer between old and new concrete layer, is of prime importance [17]. For this purposes, steel dowels or anchors can be used. For beams and RC columns similar procedures are used. Method for integration new and old concrete are show in Fig. 29.

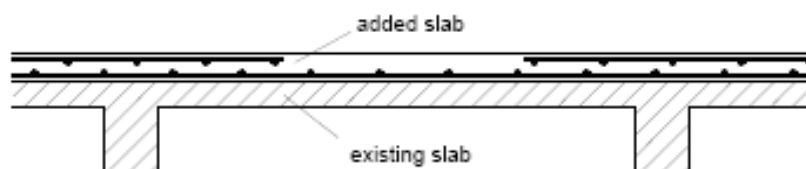


Fig. 29 Strengthening of a slab (concrete to concrete)



Fig. 30. Integration of the new and existing slab, after [35]

A deferent method for enlargement RC columns and jacketing application in Fig. 31 was shown.

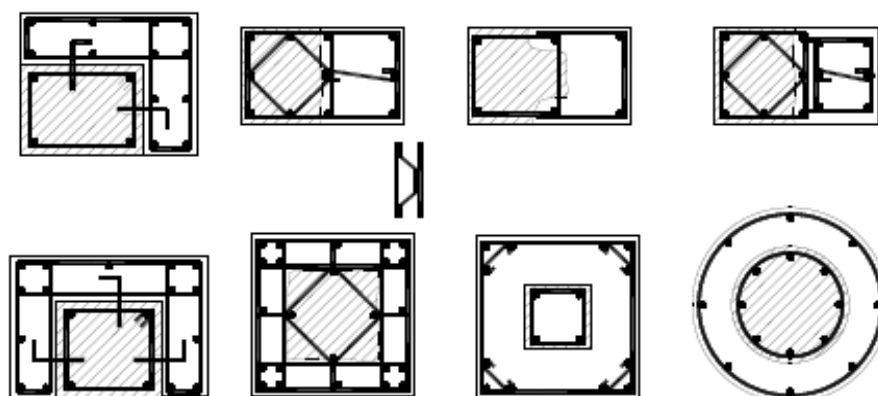


Fig. 31. Jacketing of columns, after [27]

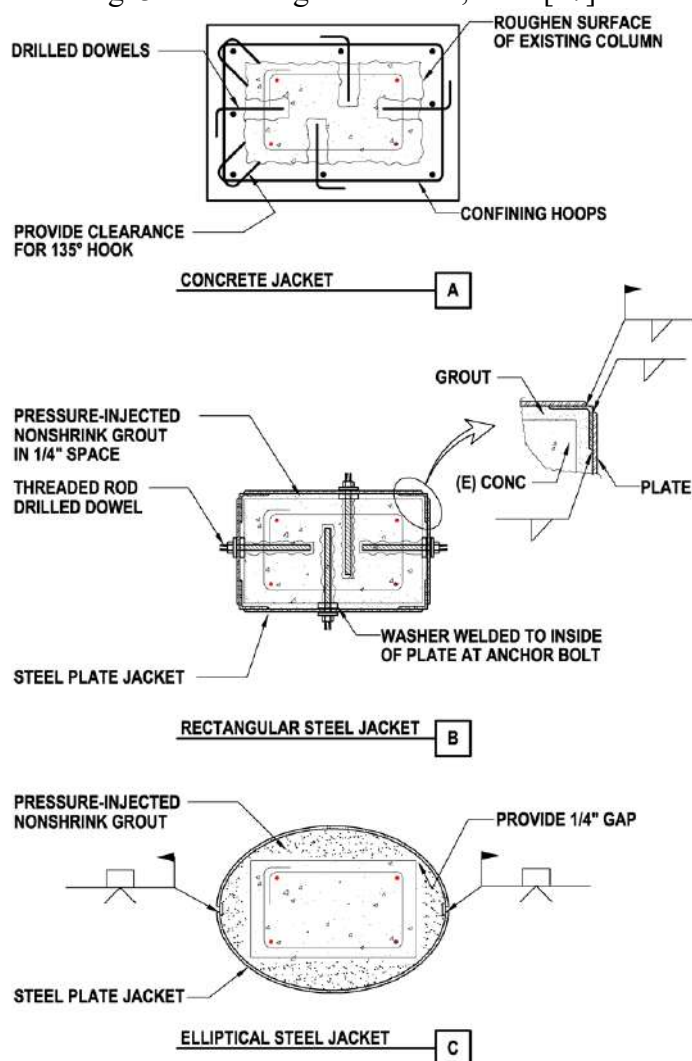


Fig. 31. Schematic illustration of RC element strengthened with different method, Concrete and steel jacketing of columns [35] and USA practice, cited in [19]

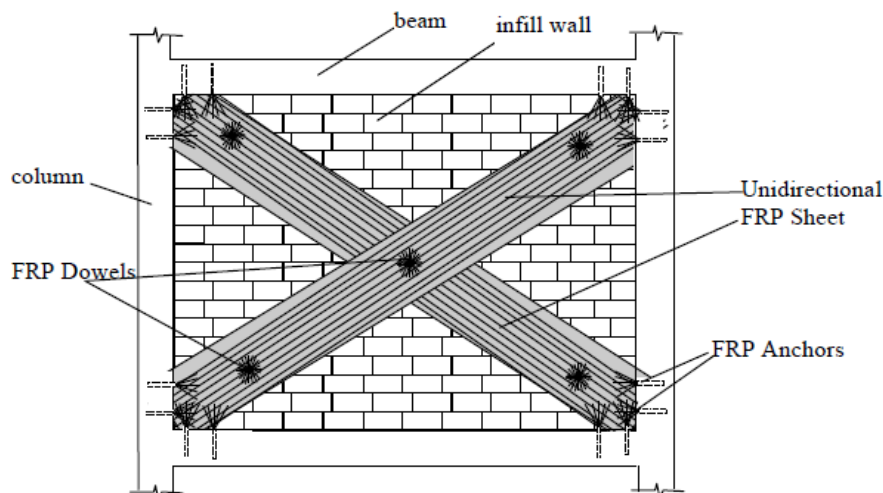


Fig. 32. FRP strengthening method for reinforced concrete frames with infill walls [38]

The performance goal of the strengthening needs to be defined. Is it merely to reinstate the building to its condition before it was damaged by the earthquake, or is it to be upgraded and if so to what level? Damaged members need to be reinstated at least to the level that they can safely carry their gravity loads; otherwise they must be replaced. Grouting under pressure of cracks in concrete and masonry with epoxy mortar is a well-established technique which is quite reliable in reinstating the concrete to its previous capacity. Jacketing and plating, as previously described, can also be used to reinstate and strengthen damaged elements. Additional elements can then be added to take the demand away from inadequate members under conditions of earthquake loading. ASCE/SEI Standard 41-06 is a valuable tool for structural engineers and the public for improving seismic performance of existing buildings.

In [5] discuss the application of *cost-benefit analysis* to the choice of performance level. Achieving higher performance levels implies higher initial costs, but it also implies a potential for future savings due to reduced damage in future earthquakes.

4.2. Upgrading of historic buildings

A number of special considerations apply to historic buildings, including:

- (a) Their historical significance implies that they need preserving for many future generations, so the return period of the design needs to be correspondingly long.
- (b) The upgrading needs to pay particular attention to the cultural and architectural values of the original construction. Balancing the need to prevent future damage while preserving the original delight and value of an old building is perhaps as much an art as a science, particularly where the 'original construction' has been built and altered over many years.
- (c) Often, it will be difficult to gain full knowledge of the properties of the materials and methods of construction.
- (d) As far as possible, strengthening measures should be reversible, so that they can be removed and modified without damage to the original, if in future more effective measures are developed. In practice, this may be difficult to achieve fully, but the strengthening measures should 'respect, as far as possible, the character and integrity of the original structure' (Feilden 1987) citrate in [5]. Prepared by ICCROM (International Centre for the

Study and Restoration of Cultural Property, Rome). These contain much practical advice on the issues involved.

Seismic isolation technique has the potential to protect both structure and non-structure from earthquake damage, while minimising the intervention required to a single isolation plane. It is therefore particularly suitable if preservation of the existing architecture.

In the most frequent types of historic masonry (double or three-leaf masonry with loose connection between leaves), there is a more or less continuous vertical joint within the thickness of masonry. The failure of those types of masonry in compression is characterized by the occurrence of vertical cracks on the faces of masonry. Actually, although the two families of cracks open at almost the same vertical load, the transverse ones grow faster. Thus, the failure of masonry is due to simultaneous compression and out-of-plane flexure of the leaves. It should be noted that cracks within the thickness of masonry are not visible or detectable (unless significant out-of-plane deformation of masonry has occurred). Such cracks may be due to decay of materials, as well as to previous normal and seismic actions on the structure including arches and domes (Fig.10).

The behaviour of masonry elements under in-plane shear is of major significance for the seismic response of buildings, as documented by typical damage, i.e. diagonal or bi-diagonal cracks in walls and spandrels. Thus, numerous research works were devoted to the behaviour of masonry under shear. It is well known that in historic buildings subjected to seismic actions, the out-of plane behaviour of (solid or with openings) walls may be critical. The vulnerability to out-of-plane actions is due to typical characteristics of historic masonry buildings, namely, the flexible floor and roof diaphragms, as well as the defective connection between floors/roof and walls (allowing for significant out-of-plane deformations of walls), the defective connection of walls at building corners, the presence of openings close to the corners [21], [31], [36] and [47].

This paper covers the following topics [47] and [51]:

- Design strategies for strengthening;
- Assessing the seismic adequacy of existing buildings;
- Methods of strengthening;
- Assessing earthquake-damaged buildings;
- Special considerations for historic buildings

Among the *possible* options are the following.

- (a) Addition of shear walls;
- (b) Cross-bracing;
- (c) Passive dampers;
- (d) Jacketing of concrete frame elements.
- (g) Reinforcing wall-to-floor connections-The connection of a timber floor to supporting masonry walls may be improved by bolted to the existing floor;
- (h) Guniting of masonry walls - Masonry walls can be strengthened adding a thin layer of mortar to one or both faces. The layer is strengthened with a light mesh reinforcement, and the mortar is;
- (i) Seismic isolation

Floors play a vital role in seismic resistance by distributing the inertial forces generated in an earthquake back to the lateral resisting elements, and by tying the entire structure together. The strength and stiffness of timber floors can be proved by screwing additional plywood sheets to the floor joists and providing. Adding steel cross-bracing to an

inadequate concrete or steel moment frame building is an alternative technique to the addition of concrete shear walls.

A number of special considerations apply to historic buildings, including:

- (a) Their historical significance implies that they need preserving for many future generations, so the return period of the design needs to be correspondingly long.
- (b) The upgrading needs to pay particular attention to the cultural and architectural values of the original construction.
- (c) Often, it will be gain full knowledge of the materials and methods of construction.
- (d) As far as possible, strengthening measures should be reversible, so that they can be removed and modified without damage to the original, and should 'respect, as far as possible, the character and integrity of the original structure'

Eurocode 8 provides Simple rules for masonry buildings without calculation (simple MB). The need for structural rehabilitation of heritage buildings is, motivated by:

- The existence of visible defects in the building;
- Damage after a particular event that affects its stability (earthquake, etc.);
- The change of the use of the building for most severe conditions; and
- Requirement of the competent authority, for instance, when there is an increase in the actions (earthquake action, traffic action, etc) imposed by new codes [22] defends the adoption of the following principles:
 - Guarantee of structural safety;
 - Respect for the cultural value of the building;
 - Minimum intervention;
 - Reversibility of the intervention;
 - Integration on the whole building;
 - Compatibility of the materials;

The purpose of the *historical survey* is to understand the conception and the *significance of the building*, the techniques and the skills used in its construction in both the structure and its environment and any events that may have caused damage. Direct observation of the structure is an essential phase usually carried out by a qualified team and to give an subsequent investigations decay and damage and applied urgent measures. Tests usually aim to identify the mechanical characteristics of the materials and of the structure (and any discontinuities). The structural scheme and damage also survey.

Before redesign intervention on structure designer must be determine whether or not the safety levels are acceptable, by analysing the present condition of both structure and materials. The safety evaluation is therefore an essential step in the project of restoration [22b]. Understanding the complexity of an ancient building or monument, uncertainties regarding material characteristics, the unknown influence of previous phenomena and imperfect knowledge of alterations and repairs carried out in the past. For chose appropriate decisions must used adequate mathematical model. For safety evaluation, can combined analysis of the information obtained from each of them, which may lead to the 'best judgement'.

The term masonry here refers to stone, brick and earth based construction adobe. Materials have a very low tensile strength and may easily show cracking within elements. Local intervention techniques are applied to a group of members that suffer from structural deficiencies and a combination of these techniques. Another effective technique consists in the application of FRP (carbon or glass fibers) confine completely the column. The column in all directions and injected with cement grout or epoxy resin.

Crack injection is a versatile and economical method of repairing RC structures. The effectiveness of the repair process depends on the ability of the adhesive material (epoxies) to penetrate into the fine cracks of the damaged concrete. This repair method can be used in minor (<0.1 mm), medium (<3 mm) size cracks, and large crack widths (up to 5 - 6 mm). In case of larger cracks, up to 20 mm wide, cement grout, as opposed to epoxy compounds.

Shotcrete is used as a repair method for RC and MS. Shotcrete can be applied to almost any surface; it can also be used in combination with RC jacket. It achieves excellent bond to most competent surfaces.

Addition of new reinforced concrete walls is one of the most common methods used for strengthening of existing structures [20]. Method is efficient in controlling global lateral drift and reducing damage in frame members and achieved a regular building configuration [13]. Seismic rehabilitation of substandard R.C. buildings with masonry infill.

FRP techniques for strengthening /upgrading may be noted as following:

- Externally bonded FRP (EBR).
- FRP plate bonding.
- FRP Confining or jacketing.
- Sprayed fiber reinforced polymers.
- Near Surface Mounted with FRP.
- Prestressed FRP.

Constructed structures provide context to aesthetic, cultural, social, archaeological, and technological issues for future generations to treasure. Seismic behaviour for masonry structures is highly dependent upon material properties, geometry, unit arrangement, connections, foundation, etc.

Steel bracing can be a very effective method for global strengthening of buildings. of external steel systems minimum disruption to the function of the building and its occupants. to provide a significant increase in horizontal capacity of the structure. Moreover the comparison between conventional and innovative devices showed that shape memory alloys (SMA)-based devices are far more effective than rubber isolators in reducing seismic vibrations.

In general, seismic rehabilitation may aim to either recover or upgrade the original performance or reduce the seismic response [42]. The crack injection (epoxy resin injection or grout injection) technique and the member replacement (substitute part of the damaged member) may apply.

The type of intervention includes (EC8):

- (a) Injection of cracks.
- (b) Local replacement of damaged concrete and steel.
- (c) Using thin steel plates for strengthening of RC slabs or beams.
- (d) Confinement of RC element (mainly of columns) by means of collars, spirals, etc.
- (e) Providing additional RC layers for flexural or shear strengthening of beams.
- (f) Providing infill in the frames.
- (g) Providing reinforced concrete jackets on columns.

The types of intervention on masonry buildings are as follows [11]:

- (a) Reduction of the mass, by removal of heavy roof covering, and parapets, etc.
- (b) Reduction of the eccentricity between the mass and the stiffness centers;
- (c) Addition of new bracing walls;
- (d) Improvement of the connections horizontal diaphragms with vertical elements;

- (e) Improvement of the diaphragmatic action of the floors;
- (f) Improvement of the quality of the masonry (by grouting);
- (g) Repair of cracked walls;
- (h) Application of vertical and horizontal confining elements to the walls;
- (i) Application of transversal confinement to the edges of masonry piers;
- (j) Strengthening of buildings by means of steel ties.

For ordinary buildings seismic risk assessment is typically carried out for the performance condition of life safety and collapse prevention, related to a seismic hazard scenario related to a 10 % probability of exceedence in 50 year or 475 year return period. For historic buildings in city centre and in case of assets of particular value, it might be more appropriate to consider the performance condition of damage limitation or significant damage associated to lower-intensity and shorter return period seismic hazard [9], [21], [32], [38] and [46-47]. ISMEP project. A most important criterion of distinguishing vulnerability approaches for Historic buildings, is whether the method is purely empirical, i.e. based on observation and record of damage in past earthquake, from which a correlation between Seismic Vulnerability and Risk Assessment of Historic Masonry Buildings in Italy

Direct techniques use only one step to estimate the damage caused to a structure by an earthquake, employing two types of methods; *typological* and *mechanical*: *Typological methods*—classify buildings into classes depending on materials, construction techniques, structural features and other factors influencing building response. *Mechanical methods*—predict the seismic effect on the structure through the use of an appropriate mechanical model, which may be more or less complex, of the whole building or of an individual structural element. *Indirect techniques* initially involve the determination of a damage index[45], by establishment of the relationships between damage and seismic intensity(peak ground acceleration, PGA),supported by statistical studies of post-earthquake damage data.

The rapid screening ATC-21 techniques extensively used in the U.S. Conventional techniques are essentially heuristic, introducing vulnerability index for the prediction of the level of damage. There are essentially two types of approach: those that qualify the different physical characteristics of structures empirically and those based on the criteria defined in seismic design standards for structures, evaluating the capacity-demand relationship of buildings ATC-13 and ASCE [3]. Uncertainty is treated explicitly through a probabilistic approach. The capacity of the structure, spectral displacement and inter-story drift limit is defined for different levels of damage. The ACI code for Assessment, Repair, and Rehabilitation of Existing Concrete Structures (ACI 562M-16) states that the design of the repair system shall consider the properties and installation of the repair materials and systems. *Hybrid techniques* combine features of the methods described previously, such as vulnerability functions based on observed vulnerability and expert judgment.

4. Conclusion and final remarks

This paper provides an overview of the literature on the behaviour of historic RC and masonry elements and buildings. Thus, selected publications are evaluated related to the behaviour of elements in compression, diagonal compression, in-plane shear and out-of-plane bending. Yet, this is a prerequisite for a reliable assessment of the current state of historic structures and, by way of consequence, for the selection of adequate intervention techniques for their preservation.

Damage affects the behaviour of individual components differently. Some exhibit ductile modes of post-elastic behaviour, maintaining strength even with large displacements (which is desirable). Others are brittle and lose strength abruptly after small inelastic displacements (which is undesirable) [14]. The task now is to identify acceptable forms of

damage and desirable building behaviour during earthquakes. Failure of RC columns happened primarily due to the lack of lateral reinforcement.

The present seismicity of the region, characteristics of the ground motions, and damage mechanisms must be investigated. In high-seismicity area, rehabilitation of buildings must provide ductile (convenient mechanism) instead of brittle behaviour. In frame structures, the damage is likely to occur at first in the beams and then in columns (columns in frame should be stronger than beams). Members must be detailed properly in order to have large ductility (despite seismic energy). It is very important to provide rigid diaphragm action (floor slabs are stiff in its own plane).

It is very important to choose an adequate structural system and foundations and to provide the capacity and limiting deformation. It is necessary to achieve the basic performance of foundations in its strength, stiffness and ductility. Those documents provide a framework and guidelines for evaluation conditions and decision of the type of intervention on damaged structures [19].

Seismic evaluation of the existing structures is a complex task. Use of sophisticated evaluation and strengthening procedure is reasonable when the level of knowledge for the structural system is relatively high. Use of advanced technology materials is only recommended, when concrete quality of the existing buildings is relatively good [17] and [19].

In decision process for the degree and the type of intervention, the following factors are considered: the layout of structural system; the strength of structures; the flexibility of structures and their ductility. The ductility requirements for RC buildings are: strong columns – weak beams; adequate shear reinforcement, so that bending mode of failure is provided; confined compression zones with close hoops or ties (EC8).

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ABOUT THE 55 YEARS HISTORY OF ZDIAKOV BRIDGE IN CZECH REPUBLIC

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Abstract: After more than 55 years of history, the Žďakov bridge continues to represent one of the largest and most beautiful bridges in the Czech Republic. 2017 marks the 50th anniversary of its commissioning and, at the same time, a successful reconstruction was completed just after that period. The Zhdiakov Bridge is made of steel as a double-jointed arched structure and still ranks among the largest and most beautiful arched bridges in the world.

ОТНОСНО 55 ГОДИШНАТА ИСТОРИЯ НА МОСТА ZDIAKOV В ЧЕШКАТА РЕПУБЛИКА

Томаш Ротер¹,

Abstract: Ждяковският мост след повече от 55 години история продължава да представлява един от най-големите и красиви мостове в Чехия. През 2017 г. се навършват 50 години от пускането му в експлоатация и същевременно с приключена точно след този период успешна реконструкция. Ждяковският мост е изпълнен от стомана като двуставна подпряна дъгова конструкция и все още се класира сред най-големите дъгови и красиви мостове по света.

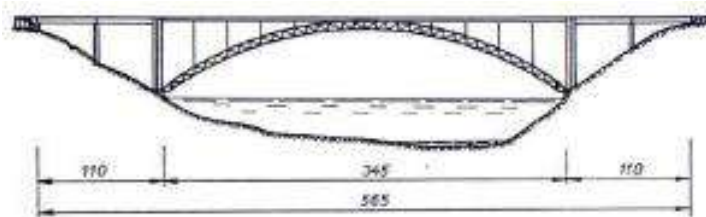
Key words: arch bridge, 330m span over Zdiakov dam

1. Въведение

За Ждяковския мост, от времето на създаването на неговата документация за изграждането му до наши дни са публикувани изключително много материали (Фиг.1 и Фиг.2). Този доклад има за цел да обобщи цялата налична информация за създаването на моста, обхващайки: кратко описание на конструкцията и монтажа му, преглед на ремонтите и подготовката и хода на реконструкцията в периода от 2015 до 2017 год. Авторът на публикацията е член и ръководител на екипа, изпълняващ реконструкцията на моста на Ждяков, което обуславя познаването на техническото решение и метода за извършване на ремонт на отделни части от моста през целия период на неговата реконструкция. Освен това, авторът чрез този доклад, прави опит да изрази своето възхищение към всички, които са участвали в подготовката на изграждането на моста на Ждяков. Причина за създаването на

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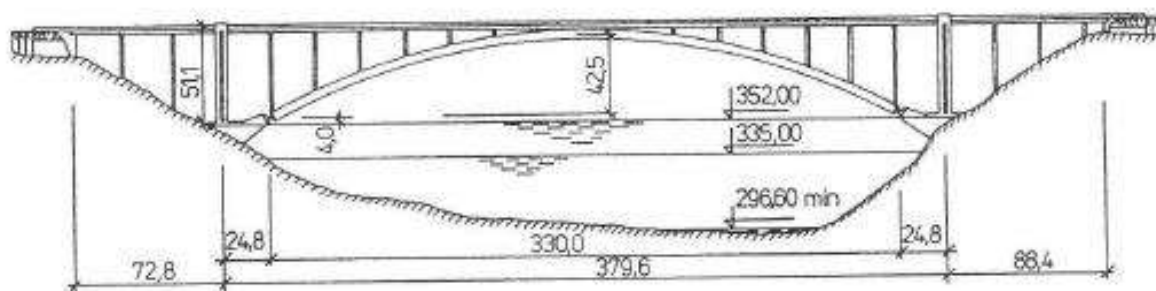
тази публикация са и сериозните изводи от този обект за бъдещите генерации, направени от авторите на проекта, голяма част от които са били негови преподаватели.



Фиг.1. Изглед към моста от 1964 г. **Фиг.2.** Проект на моста от конкурса през 1941 г

2. История на възникването на идеята за Ждяковския мост

По склоновете на дълбоката долина на река Влтава, пътят се спуска до реката близо до селището Žďákov, след което се продължава с ферибот и ново изкачване на другия бряг. През първата половина на миналия век възниква необходимостта от модерна транспортна връзка между Пилзен и Табор. Премостването на дълбоката долина близо до селището Žďákov изисква мостова връзка да бъде с дължина повече от 500 m, с ниво от приблизително 110 m над речното корито. По това време вече се знае, че ще бъде построен язовир Орличка, който ще повиши нивото на реката с 38 m. Първият ограничен обществен търг за мост е обявен през 1941 г., в който са представени варианти на гредови и дъгови мостове [1, 2, 15, 29, 39]. Спечелилият конкурса проект е изпълнен от проектантското бюро на заводите Шкода в Пилзен, съвместно с бюро на минно-металургичната проектантска организация в Прага с участието на проф. Фр. Фалтус. В централното поле на моста е проектирана сърповидна стоманена решетъчна дъга с отвор 345 m. Продължението на моста пред и зад големия му отвор е реализирано със стоманобетонни греди, съгласно Фиг.2., чието изпълнение е спряно поради събитията през втората световна война. След войната нивото на язовир Orlická се повишава с 19 m, поради което предвоенният проект е променен. Нов публичен конкурс през 1954 г., донесе нови идеи и в допълнение към стоманените варианти са представени няколко проекта и за ст.бет. мостове. Проектът на пълностенна стоманена двуотворна дъга с централна стоманобетонна бетонна опора, разположена в средата на реката, която можеше да бъде изградена преди язовирът Orlická да бъде напълнен, изглеждаше най-изгоден. Забавянето на работата по проекта обаче доведе до изоставянето на този вариант и окончателно за изпълнение бе избран проект с една стоманена пълностенна двуставна дъга с отвор m, (Фиг.3).



Фиг. 3. Конструктивна схема на моста Ждяков

Проектът за моста е разработен в държавния проектантски институт: Hutní projekt Praha под ръководството на Ing. Й. Земан [3](Фиг. 4). Главен статик на моста е инж. А. Schindler, CSc., (Фиг. 5), по-късно професор и ръководител на катедрата по стоманени конструкции във Факултета по строително инженерство, в Чешкия технически университет в Прага. Експертът и човекът, който значително повлиява на решаващото решение, до този момент е проф. д-р инж. F. Faltus, DrSc., (Фиг. 6), който е предшественик на инж. А. Шиндлер, като ръководител на катедрата по стоманени конструкции и най-важният следвоенен специалист по стоманени конструкции в Чехия[4, 5, 6, 17, 18, 19].



Фиг.4. Инж. Й. Земан
(1922 – 1997)



Фиг.5. Проф. А. Шиндлер
(1920–2006)



Фиг.6. Проф.Фр. Фалтус
(1901 – 1989)

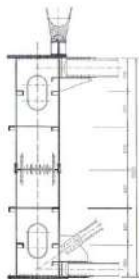


Фиг.7. Инж. Фр. Стейскал
(1937-2017)

Мостът е изпълнен от VŽKG (понастоящем: Vítkovice) и други два завода за мостове във Vítkovice, Ostrava и Lískovecká (която вече не съществува), които участват в изпълнението му. Работните чертежи са направени в проектантското бюро на Vítkovice под ръководството на Ing. A. Sehnala а O. Kaděry. Материалът за моста е доставен предимно от чешки металургични заводи, само част от него е внесен от Унгария. Продължаването на строителството на моста след прекъсването на войната започва през 1956 г. и се строи с прекъсвания до 1967 г. [20]. Стоманената конструкция на моста е сглобена от Hutní montáže Ostrava под ръководството на Ing. L. Freislera в периода 1963-65г. От 1965 г., постоянен строителен надзор на инвеститора е Ing. F. Stejskal, (Фиг. 7). Една от основните му задачи е проверката на пристегнатостта и качеството на нитове, които са общо 1 200 000 броя. Въпреки това, за разлика от първоначалното предположение за монтаж на моста над ненаводнената зона, той е монтиран над езерото, тъй като строителството на язовира Orlický е ускорено и мостовите фабрики, претоварени с изграждането на тежката промишленост, не са в състояние да доставят мостовата конструкция в съкратения срок. Всички геодезически работи по време на строителството на моста са извършени от Ing. J. Herda. Мостът е въведен в експлоатация на 27 април 1967 г., след завършването на комбинираната стомано-стомабетонна пътна плоча на моста и асфалтовата настилка върху нея.

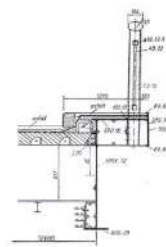
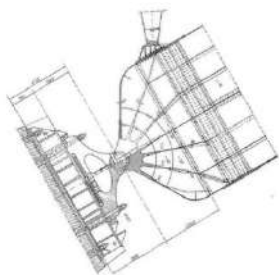
2. Описание на конструктивното решение

Основната носеща конструкция на средния участък на моста е изградена от две двуставни пълностенни дъги с отвори по 330m и стрелка на дъгата от 42,5 m, поддържани от диагонално разположени стоманобетонни опорни конзоли с дължини 24,8 m [8, 9](виж:Фиг. 3). Мостовите дъги са разположени във вертикални равнини на осово разстояние една от друга 13 m и са свързани помежду си чрез надлъжни стабилитетни връзки в горния и долен пояс на дъгите и междинни стабилитетни връзки в радиалните равнини(Фиг. 8). Геометричната форма на центъра на дъгите е съставена от три кръга, в централната част с радиус $R = 332,7$ m и в крайните части $R = 396,6$ m.



Фиг.8. Изглед на конструкцията на моста **Фиг.9.** Дъга под стойката **Фиг.10.** Укрепване между дъгите

Дъгообразните греди са с кутиеобразно сечение с височина 5000 mm, с осово разстояние на стените 1000 mm (фиг. 9). Вертикалните стебла на гредата са разделени по средата на височината им по производствени причини, като непрекъснатата надлъжна връзка е реализирана с нитовани съединения. Промяната на напречното сечение на дъгата по дължината и, се извършва с помощта на променящ се брой занитени сравнително тънки фланцевидни плочи. Формата на напречното сечение на кутиеобразното сечение се осигурява от напречни диафрагми. Под стойките, поддържащи гредата на мостовата пътна конструкция, има пълностенни диафрагми с люкове в напречното сечение, докато в междинните напречни сечения са разположени през 4000 mm прътови диафрагми. Стените на кутиите на дъгите са укрепени с едностранни надлъжни ъглови профили. Две от тях са заварени от външната страна на кутията, като по този начин допринасят за по-добър естетически изглед на моста. Във вътрешността на кутиеобразното напречно сечение на дъгата е разположена надлъжна прътова връзка в половината от височината ѝ, която осигурява формата на половините на дъгата по време на транспорта и монтажа и. Сводовите греди са изработени от стомана S355. Надлъжните прътови стабилитетни връзки в горния и долен пояс на дъгите имат формата на К-система. Отделните усилващи пръти са изработени от стомана S355 и имат кутиеобразно напречно сечение за коравина и лесна поддръжка. Те са изпълнени от заварени напречни сечения. Връзките на тези пръти към дъгите са изпълнени с нитовани съединения. Подобно изпълнение притежават и диагоналите на междинните радиални връзки, чиито детайли на свързване към дъгите са също нитовани (фиг. 10). Напречните връзки в петите на дъгите са изпълнени от пълностенно, с височина 5000 mm, кутиеобразно напречно сечение.



Фиг.11. Схема на ставното устройство **Фиг.12.** Изглед на ставата **Фиг.13.** Главна греда под ст.бет.плоча

Ставните устройства в петите на дъгата са от лята стомана. Самата ставна връзка се състои от кобилица, прътова ос с диаметър 430 mm и подложно столче. Под столчето е вбетонирана стоманена разпределителна гредоскара с възможност за монтиране на плочести преси за регулиране на позицията на ставата при необходимост. (фиг. 11 и фиг. 12). Дъгите са сглобени като триставни конструкции. Горните монтажни съединения са отстранени след въвеждането на постоянното натоварване, чрез

монтиране на средни конструктивни елементи, които директно контактуват със стените и фланците на двете половини на дъгите(виж раздел 4). Стойките, поставени върху гредите на свода, поддържат двете надлъжни главни греди на мостовата конструкция през разстояние 23,4 м. В напречното сечение на моста стойките не са разположени в оста на гредите на свода, а над вътрешните стени на кутиеобразното сечение на свода в напречно направление през 12 м. Дължината на стойките варира от 0,5 до 41,4 м. Стойките се изпълнени от заварени кръгли тръби с диаметър от 450 до 1000 mm. Те са стеснени в двата си края и завършват с лята цилиндрична стоманена става. Съединенията са структурно модифицирани по такъв начин, че да могат да поемат и възможни реакции на опън. Дебелината на стените на тръбите е степенувана от 10 до 15 mm. Интересно е да се отбележи, че поради промяната на собствената честота, повисоките стойки са допълнително напълнени с чакъл, за да се елиминират трептенията вследствие ветровото натоварване, предизвикващо възникването на Кармановите вихри. Това явление не е предвидено в проекта, но когато стълбовете на построения мост започват да вибрират при определени условия на вятъра, насипването с чакъл решава ситуацията [7, 10].(виж р-л 5). Конструкцията на пътната част на моста се състои от две надлъжни главни греди, напречни греди и стоманобетонна пътна плоча. Както вече беше казано, основните греди на мостовата пътна плоча са на осово разстояние една от друга на 12 m и се подпират от колони през 23,4 m в сводовите греди на моста. При пилони осовото разстояние между стойките е увеличено до 26,0 m. Главните греди под плочата на моста са разделени на три непрекъснати греди, чрез композиране на ставни устройства между тях, с цел намаляване на температурните външни влияния. Външните греди са с обхват: $2 \times 23,4 + 2 \times 26,0$ m, респ. $2 \times 26,0 + 2 \times 23,4 + 15,6$ m, средната греда има 14 полета с разстояние $14 \times 23,4$ m. Неподвижното ставно подпиране на гредите е разположено в средата на отвора на свода на моста. Главните греди на мостовата конструкция са монтирани върху подвижни опори върху двата устоя и върху пилоните. Изпълнени са като заварено, едностеблено несиметрично напречно сечение с височина 1800 mm, (фиг. 13). Изработени са от стомана S355 и като комбинирано стомано-стоманобетонно сечение, при което се отчита съвместното поведение на бетон и стомана при вертикални въздействия. Върху пилоните и устоите на моста, основната греда е монтирана върху цилиндрични лагери. При пилоните е добавен специален прът в опорната точка за предаване на хоризонталната реакция, тъй като в хоризонтална посока мостовата конструкция действа като непрекъсната греда по цялата дължина на моста. Напречните греди са вградени между гредите на мостовата плоча и са разположени на разстояние 2,600 m една от друга. Отворът на напречните греди е 12,0 m. Това са едностеблени, пълностенни-съчетани, заварени симетрични, с I напречно сечение греди с променлива височина на стената. Горният фланш на гредата има двустранен покривен наклон от 2%, а долният фланш е хоризонтален. Горният фланш е свързан към стоманобетонната плоча на пътната конструкция посредством заварени корави дюбели(фиг. 13). Височината на напречните греди е в диапазона: от 880 до 1000 mm. Напречните греди са заварени към стеблата на надлъжните греди в двата си края по време на монтажа. Изпълнени са от стомана S235. Стоманобетонната пътна плоча е с основна височина 180 mm. Плочата е подпряна върху напречни греди през разстояние 2600 mm и, както беше споменато, взаимодейства статически с тях. По време на създаването си, тя е излята върху трапецовидно вълнообразна, демонтируема след това, стоманена ламарина. Долната повърхност на плочата е приблизително на нивото на горния фланш на напречните греди. По проект бетонът на плочата е с марка около 250, но извършената диагностика през 2015 г. показва качество, отговарящо на днешния бетон – клас: C25/30. Мостът е с пътно платно с ширина 10,5 m и два тротоара с ширина 1,25 m. Общата широчина на платното на моста е 13,488 m. Мостът е с парапета от кутиеобразни профили с

височина 1050 mm. Общата дебелина на пътния и изолационния слой е 120 mm, а на асфалтовата настилка е 50 m. Височината на бордюрите е 180 mm. Отводняването на моста се състои от изпълнени водосливни сифони, в пътното платно на моста, вертикални и надлъжни, монтирани водосливни инсталации поставени под конструкцията на моста и от сводовата част на моста при пилоните. Стоманобетонните двуделни пилони са с осово разстояние 379,6 m и височина 46,9 m. Напречното сечение на всеки дял на пилоната е с размери 4,8 x 3,3 m. Пилоните са бетонирани в сглобяеми стоманобетонни елементи с височина 1,0 m и дебелина 150 mm, които представляват загубен кофраж. В горната си част, двата клона на пилоната са свързани рамково с напречна греда. В основата на пилоните са изпълнени две стоманобетонни конзоли с дължина по 24,8 m. Върху тези конзоли са разположени ставните устройства на сводовите греди.

3.1. Основни параметри на моста

Общата дължина на стоманената конструкция е 541,94m. Разстоянието между осите на краищата на моста е 544,17 m. Общата ширина на моста е 13,488 m. Общото тегло на стоманените конструкции е 4465 t от които стомана: клас S355 (дъги, стабилитетни връзки в сводовата конструкция, главни греди в мостовата пътна плоча) 3304 t и клас: S235 (и други) 1161 t

4. Монтаж на моста

Първоначалният проект за монтаж на моста Ждяков предполага сглобяване над ненаводнена долина [9]. От двете страни е планирано да се изградят пилоните със стоманобетонните конзоли, върху които се монтират ставните устройства. В средата на долината е планирано да се извърши монтаж на временен стълб с обща височина 92 m, изработен от материала, предвиден за мостовата пътна плоча. Дъгите е планирано да бъдат сглобени на две половини върху ниско скеле на дъното на долината. След това двете половини се повдигат постепенно с помощта на два крана и поставени върху краищата на конзолите и стъпващи върху временно монтирания стълб. Дължината на монтираните части на конструкцията възлиза на приблизително 160 m. Осигурена е възможност на монтираните конструктивни елементи да се завъртат около ставните устройства, с цел да се даде възможност за монтаж на крайната горна част на моста. След завършване на сглобяването на свода, мостовата конструкция на пътната плоча постепенно започва да се сглобява от центъра на участъка на свода към пилоните и опорите. Първоначалният монтажен проект обаче се променя, тъй като срокът за изграждане на язовир Orlická е съкратен и освен това, води липсата на капацитет на мостовите фабрики довежда до забавяне на производството на стоманената конструкция на моста. Поради това решението за проект на монтаж над ненаводнената долина е изоставен. Както вече бе споменато, по време на войната са построени само опорите на моста и по-нататъшното строителство е прекъснато. Строежът на моста е възобновен през 1956 г. и е разделен на два етапа. На първия етап са изпълнени ст.бет. основи на пилоните, включително конзолите и самият пилон. Освен това са изпълнени основите на двете монтажни кули в долината, анкерването им и частта от кулите, която се разполагат под бъдещата повърхност на язовира. При изграждането на монтажните кули по системата PÍŽMO се има предвид възможността за по-късно изтегляне на части от кулите под вода, без да е необходим достъп до водата. През 1963 г. започва вторият етап от монтажа на стоманената конструкция над напълнения язовир Orlické. Монтажът продължава едновременно от двете опори към средата на моста, с помощта на два Дерик крана с капацитет от 50 t при конзолно разстояние до

местопложението им до 25 м. Дерик-крановете се придвижват по релсови пътища, разположени върху напречните елементи на постепенно сглобяваната мостова пътна плоча, поддържана от тръбните стойки. Когато монтажната инсталация преминава зад пилоните, дъгите на моста, включително стойките и мостовата конструкция, са монтирани по конзолен начин. Свободните краища на дъгите се окачват, чрез вмъкване на монтажни диагонали от гредата на мостовата плоча към дъгите, докато се достигне върха на монтажните кули. Монтажните кули са сглобени предварително до необходимата височина. Сглобяването на дъгите продължава по същия начин до достигане на върха им, където е монтирана централната ставна връзка, която е замонолитена след като по-голямата част от постоянното натоварване е приложено. Това намалява появата на огъващи моменти върху статически неопределимата конструкция. Дори горната ставна връзка е монтирана на 280 mm под оста на свода, което допълнително увеличава хоризонталната реакция от теглото на стоманената конструкция, като по този начин се получава подходящ резерв в носимоспособността дъгата от този допълнително внесен натиск в свода. Към конзолните части на напречните греди са монтирани корнизните греди заедно с парапета. В двата края на моста са монтирани преходните фуги на моста [9, 20]. След сглобяването на цялата стоманена гредова конструкция на пътната плоча на моста, по цялата ширина между главните греди на моста е изпълнена стоманобетонната плоча. Плочата е свързана с напречните греди с помощта на корави дюбели. След това следва полагането на пътното настилка и тротоарите. Стоманените конструкции са доставени на строителната площадка с основно сярно покритие. След сглобяването стоманените конструкции са почистени чрез пескоструене. Цинковата метализация е изпълнена с дебелина от 60 до 80 μm . За тази цел е използван със зеленикав цинков хромат. Двата горни слоя са нанесени през годините от 1967 до 1977 г. по алпийски способ. Цялата стоманена конструкция на моста е със сребрист цвят, единствено ръкохватката на парапетът е със син цвят а пълнежът на парапета е с жълт цвят. Вътрешните повърхности на гредите на дъгата са оставени само на основен грунд.

5. Резултати от тестовото натоварване

Преди пускане на моста в експлоатация е извършено съответното **тестово** статично и динамично **натоварване**. По време на теста за статично натоварване са измерени деформациите на двете дъги [6, 9]. *Получени са интересни резултати в измерените отклонения на деформациите от ексцентрично поставения товар към надлъжната ос на моста. Изчислените стойности на деформацията на дъгата при натоварената и ненаатоварената дъга са доста различни. Напротив, измерените стойности на деформация на двете дъги са почти идентични. Сумата от изчислените и измерените деформации на двете дъги е еднаква.* От това следва, че действителната коравина на усукване на основната носеща конструкция е по-голяма от приетата в статическото изчисление. Действителната коравина на огъване се съгласува много добре със статическото изчисление. По време на изпитването за динамично натоварване е обърнато голямо внимание на трептенията на най-високите колони [10]. Стойките 4 и 18 са напълнени с чакъл, за да се предотвратят трептенията от вятъра. По време на теста стойките са приведени във вибриращо състояние, вследствие удар от товар с величина от 50 kg. Измерените деформации и изчисленото затихване са сравнени с измерените стойности по време на изпитването на колони, които не са пълни с чакъл. Освен това е извършено дългосрочно наблюдение на движенията на стойките в зависимост от скоростта и посоката на вятъра. Стойките, запълнени с чакъл, осцилират при натоварване от вятър в първата форма с честота $f(1) = 0,73 \text{ Hz}$, а колоните, които не

са запълнени с чакъл, осцилират в първата форма с честота $f(1) = 2,1 \text{ Hz}$. От това следва, че запълването на стойките с чакъл води до намаляване на честотата на първата форма на трептене и по този начин до намаляване на резонансната скорост на вятъра. Запълването на стойките с чакъл е довело до драстично намаляване на амплитудите на първата форма на трептене, което е благоприятно от гледна точка на умората.

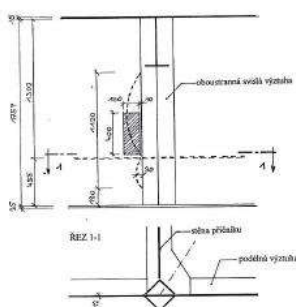
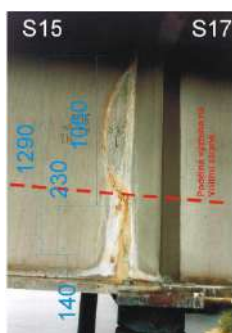
6. Ремонти в периода на експлоатация на моста

6.1 Ремонт на пътя през 1992 година

През 1992 г. е извършен основен ремонт на пътя на моста, включително ремонт на изолацията.

6.2 Ремонт на пукнатини в стеблото на главната греда на мостовата конструкция на пътната плоча през 1998 година.

Мостът Ждяков е в непрекъсната експлоатация до 1998 г., и дотогава не е извършван основен ремонт на носещата конструкция. Извършвана е само редовна поддръжка. Визуално мостът изглежда все още доста добре в края на 90-те години. Ето защо е поднесена голяма изненада, когато като част от редовната поддръжка в края на лятото на 1998 г. открива, че на три места в стеблото на мостовата греда се виждат дълги вертикални пукнатини, минаващи почти по цялата височина на стеблото на гредата, фиг.14 и фиг.15, [15, 16, 19].



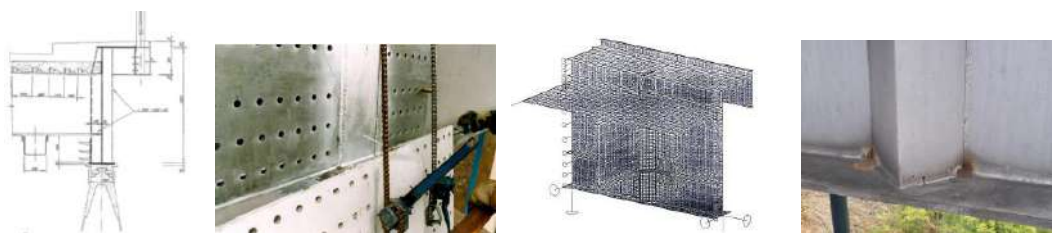
Фиг.14. Външен изглед на стеблото с пукнатина в главната греда в мостовата конструкция

Фиг. 15. Схема на стеблото в главната греда на мостовата конструкция с очерчания на пукнатината

Фиг.16. Стебло на главната греда на мостовата конструкция с усилваща плоча закрепена към външната страна на стеблото

След като са открити пукнатините, мостът е незабавно затворен за целия трафик, като предпазна мярка. Администрацията и поддръжката на Písek roads, която е администратор на моста, изисква от компанията Pontex, s.r.o. оферта за усилване на моста и определяне на причините за пукнатини с евентуално предложение за допълнителни мерки, които трябва да предотвратят по-нататъшни повреди на моста. Тъй като капацитетът на натоварване на моста Ждяков не е известен до момента на повредата, задачата е разширена, за да включи изчисляването на капацитета на натоварване по смисъла на тогава действащия ČSN 73 6220. Фирмата Pontex, s.r.o. се обръща с молба към Катедрата по стоманени конструкции на Факултета по строително инженерство на Чешкия технически университет в Прага за съдействие при решаването на тази задача. По време на проверката става ясно, че неизправностите не са причинени от локален дефект в материала. След предварителни изчисления е преценено, че няма прекомерни натоварвания от движение по моста в местата на разрушаване. Затова

вниманието е насочено към детайлите на конструкцията. Прави впечатление, че пукнатините се появяват винаги над стойките, в местата на вертикалните вкорвяващи ребра на стелото в основния трегер на мостовата конструкция. След оценка на ситуацията е решено да се извърши бърз ремонт чрез закръпване на местата с пукнатини по цялата височина на стелото с едностранни плочи от ивична стомана от външната страна на стелото. Усилващите плочи съвместно с подложките, които елиминират изкривяването на стелото с дебелина до 15 mm, са прихванати към стелото с предварително напрегнати болтове M24, клас 10.9 (фиг. 16). Тези плочи са проектирани да могат да поемат всички възможни напрежения в стелото на основната греда на мостовата плоча, така че ремонтният участък да не бъде слабо място за конструкцията на моста. Стелото на главната греда на мостовата плоча в местата с пукнатини е подсилена с двустранни вертикални укрепващи ребра, изпълнени от равнобедрен ГВП, поставен по такъв начин, че заедно със стелото е създадено профил със затворено кутияобразно напречно сечение (фиг. 17а). За да може да се укрепят повредената зона на стелото с плоча от външната му страна, укрепващо ребро на това място е отстранено, и заваръчните шевове, свързващи ГВП със стелото, са отстранени. При пробиване на отворите за закрепващите болтове в долната част на стелото се установява повишена влажност на заваръчните шевове от вътрешната страна на стелото. Затова е направен пробен отвор и водата, натрупана в пространството на кухината, образувана от вътрешното ребро и стелото на главната греда на моста, започва да изтича от отвора (фиг. 17б). Тогава не беше ясно как водата е попаднала в кухината. Предполага се, че причината е дефект във водоустойчивата изолация на мостовата настилка [15]. Истинската причина за това явление е разкрита едва по време на реконструкцията на моста Ждяков през 2015 г.



Фиг.17а. Разрез през главната греда на мостовата плоча над пилона

Фиг. 17б. Изтичане на вода след пробиване на стелото на мястото на вътрешното вкорвяващо ребро

Фиг.18. 3D модел на стелото с вкорвяващи ребра

Фиг.19. Пробиване на ГВП в долния му край

Беше направено заключение, че замръзването на водата в кухината, образувана от вътрешното укрепващо ребро върху стелото на основната носеща греда на мостовата конструкция, може да бъде непосредствената причина за пукнатината. Тук е необходимо да се добави, че стелото е укрепена от вътрешната страна с хоризонтално надлъжно ребро. В пресечните точки на заваръчните шевове, свързващи вертикалното и хоризонталното укрепващо ребро, несъмнено се натрупват отрицателните ефекти, вследствие заваръчната връзка. Така се оказва, че условията за образуване на пукнатини се крият в претоварването на термично засегнатия материал с остатъчни напрежения, които може да бъдат причина за образуването на пукнатини. Опростените статически изчисления, при които за основа е взета измерената постоянна деформация в точката на укрепващите ребра, потвърждават тази хипотеза. Следователно, като част от преизчисляването на моста, бе създаден пространствен модел на горепосочения детайл от конструкцията [15] (Фиг.18). Затвореното пространство е фиктивно запълнено с вода при изчислението и оставено да замръзне. Изчислените напрежения в стелото на главната греда на мостовата конструкция значително надвишават границата на якост

на материала в зависимост от приложените гранични условия в линейния теоретичен модел. Пиковите на напрежението в стеблото възникват точно в точките на пресичане на вертикалното и хоризонтално укрепващо ребро. Това потвърди хипотезата за причините за пукнатините. По време на ремонта на повредените участъци от стеблото на основната греди на моста е изрязан участък с размери около 200 x 400 mm и материалът е подложен на необходимите тестове за установяване на неговите материални и химически параметри. Тестовите на материалите са извършени от оторизираната лаборатория за изпитване на Факултета по строителство на Чешкия технически университет в Прага, химическите тестове са извършени в оторизираната лаборатория Poldi Kladno. Тестовите показаха, че по химичен състав и механични свойства материалът отговаря на използваната тогава стомана 11523. Не са открити белези за стареене на стоманата (намаляване на якостта на прорезите) или други дефекти. Контролният химичен анализ и металографският анализ на повърхността на тестовия образец е извършен в Изследователския и тестов институт на NH Ostrava. Местните експерти откриват, че изследваният изрез притежава свойства с пропукващ се характер на разрушение. По изследваната проба обаче не се забелязва видимо огнище на дефектиране. По време на макроскопския анализ на повърхността на разрушение, ребрата са ясно видими, което показва посоката на разпространение на пукнатината отдолу нагоре. От това се заключава, че точката на инициране на разрушението е в точката на пресичане на заваръчния шев, свързващ двустранното вертикално ребро на стеблото към едностранното надлъжно ребро на стеблото. Това предположение е потвърдено и от факта, че нивото на присъщите напрежения от заваряване, е повишено на това място. Тъй като дори в долната част на образца повърхността на разрушение няма вид, който да демонстрира жилавост при пластично разрушаване, произтича изводът, че разрушаването вероятно е настъпва поради крехко разрушение, а не поради умора на материала.

Тъй като по това време не е известно как водата е попаднала в кухнята, всички вертикални затворени ребра на стеблото на главната греди на моста са пробити в долната част (фиг. 19), така че натрупването на вода никога да не се среща в бъдеще, т.е. водата в ребрата няма да замръзне и пукнатините няма да се появят във времето на експлоатация на моста. Освен това пробитите кухни се снабдяват с антикорозионна защита на базата на парафини. Ремонтът е извършен от Hutní montáže Ostrava през есента на 1998 г. и след около два месеца мостът е отворен отново за движение.

6.3. Статическо преизчисляване на моста през 1999 година

6.3.1. Глобален статически анализ

В Pontex, s.r.o., е създаден пространствен компютърен модел, който най-подробно отразява поведението цялата конструкция на моста при нейното натоварване [12, 13, 14, 16]. Входните геометрични данни са взети от проекта, тъй като експертите се съгласиха, че конструкцията не е достатъчно чувствителна, за да изисква сложни и скъпи измервания за определяне на действителната геометрия на моста. Резултатът от глобалния анализ беше определянето на вътрешните сили върху всички носещи елементи на мостовата конструкция от отделни състояния на натоварване. Мостова плоча: Изчисляването на товароподемността е извършено съгласно действащия тогава ČSN P ENV 1994-2/1998 „Проектиране на стоманобетонни конструкции – част 2: Стоманобетонни мостове“. Товароносимостта е определена в 12 критични напречни сечения на плочата в гранично състояние на носеща способност на огъване и срязване за пластичното поведение на напречното сечение. Ефектите от съсъхването, пълзенето и температурата не са отчетени при изследване на конструкцията в пластичен стадии.

Носещата способност на напречните сечения е определена от интеракционната диаграма на огъващия момент и нормалната сила. Напречни греди: Оценката и изчисляването на товароносимостта бяха извършени подобно на тези за пътната плоча. Товароносимостта е определена в 6 критични напречни сечения от 1-во и 2-ро гранично състояние за пластично и еластично поведение на напречното сечение. Главни греди на пътната плоча: Оценката и изчисляването на товароносимостта са извършени съгласно ČSN 73 6205/1998 „Проектиране на стоманени мостове“, като се вземе предвид ČSN P ENV 1993-1-5/1998 „Проектиране на стоманени конструкции - част 1.5 общи правила - Допълнителни правила за плочо-гредови пълностенни конструкции без странично натоварване“. Товароносимостта е определена в 8 критични напречни сечения в гранично състояние на носимоспособност за еластичното поведение на напречното сечение.

Дъги и колони: Оценката и изчисляването на товароносимостта е извършено съгласно ČSN 73 1401/1998 „Проектиране на стоманени конструкции“, ČSN 73 6205/1998 „Проектиране на стоманени мостове“ и ČSN P ENV 1993-1-5/ 1998 г. "Проектиране на стоманени конструкции - част 1.5". Товароносимостта е определена в три критични напречни сечения (две напречни сечения на една четвърт от обхвата на свода и едно, в мястото на най-дългата вертикала) при граничното състояние на носещата способност за еластичното поведение на напречното сечение. За дъгите изчислението е извършено съгласно теорията от II ред, след на 10 итерационни приближения. Стойките са изследвани съгласно от първи ред.

Укрепване на дъги: Извършена е оценка и изчисляване на товароносимостта за 10 критични напречни сечения от пространствения модел съгласно теорията от 1-ви ред от граничното състояние при еластичното поведение на напречното сечение.

6.3.2. Локален статически анализ

От резултатите от глобалния статически анализ на цялата конструкция на моста не е открито такова високо напрежение в главната греда на мостовата пътна плоча, при каквито и да е условия на натоварване, което би могло да доведе до крехко или вследствие на умората разрушаване на стеблото. Както вече беше упоменато в предишния параграф относно причините за повредата, беше извършен локален статически анализ (при натоварване от лед, намиращ се във вътрешността на затвореното пространство между стеблото и вътрешното вертикално укрепващо ребро) върху триизмерен модел на плоча от стената на участък, (при главната греда на мостовата плоча) в мястото на вертикалните ребра (фиг. 18). Натоварването е моделирано чрез непрекъснато равномерно натоварване, действащо върху стеблото с големина, съответстваща на измерената деформация на стената от 10,2 mm. Това е максималната възможна деформация, когато цялото пространство се пълни с вода и след това замръзва. Кривата на напрежението върху стеблото показва концентрацията на муо при заваръчния шев на реброто със стеблото, с екстремум при пресичането на надлъжното с вертикалното ребро. Последното достига стойност до 2844 МПа при линейно изчисление, което е около осем пъти границата на провлачване $f_y = 355$ МПа. От траекториите на главните напрежения и на двата модела се вижда, че пукнатината трябва да се е разпространила перпендикулярно на посоката на главните напрежения, т.е. вертикално.

6.3.2. Изводи от статическото изследване

Въз основа на глобалния статически анализ е извършена оценка и изчисляване на товароносимостта на отделните носещи елементи на мостовата конструкция на моста

на Žďákov и не е открита причина за появата на пукнатини в стената на главния гред на мостовата плоча. Въз основа на резултатите от локалния статически анализ е доказано, че причината за пукнатините в стеблото на главната гред на моста е замръзване на вода в затвореното пространство между стеблото и вътрешното вертикално ребро. Резултатите от глобалния статически анализ, извършен върху пространствения изчислителен модел, бяха сравнени с оригиналните ръчни изчисления от 1960 г., при което беше показано, че те се съгласуват много добре с новите изчисления. Главният строителен инженер на моста Žďákov, проф. А. Шиндлер съгласува тези резултати, като изрази своето одобрение, а младите инженери на Pontex бяха изненадани какво, от днешна гледна точка, можеше да се изчисли с относително примитивните средства на статиката от миналото. Статическото изчисление доведе до следните товароносимости на моста Žďákov: при основна комбинация: 32t–лимитирана от дъгите; При разширена комбинация: 102t–лимитирана от напречните греди; Извънредна комбинация: 273t –лимитирана от стойките. В същото време бе изчислено, че за натоварването съгласно ČSN 73 6203 отделните елементи на моста се използват, както следва: пътна плоча-79% ; напречни греди -84% ; главни греди на пътната плоча на моста-100%; стойки-91%; дъги- 100%; стабилитетни връзки- 65%.

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**150 YEARS SINCE THE BIRTH OF THE GREAT GERMAN ENGINEER
PROF. DR. ENG. EMIL MÖRSCH AND HIS BRIDGES**

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Abstract: *In this paper the authors present a short autobiography of the great German engineer and lecture, prof. Dr. Eng. Emil Mörsch. It is given short information about his education and scientific works in Stuttgart. Attention is drawn to some realized Steel-Concrete bridges projected himself.*

Key words: *Emil Mörsch, Steel-Concrete bridges, 150 years anniversary of born of EM*

1.Общи данни

Emil Mörsch (роден на 30 април 1872 г. в Reutlingen, и починал на 29 декември 1950 г. в Stuttgart-Weilimdorf), представлява значим и световно известен немски строителен инженер, изследовател и университетски преподавател, който си създава име в областта на стоманобетонните конструкции(Фиг.1). Неговите теории за оразмеряване на стоманобетона имат трайно влияние върху стандартизацията в DIN и неговите предшественици. На базата на обширни експериментални серии той разработва номограми, с които се оразмеряват стоманобетонни греди и определя напречното сечение на армировката в тях. На негово име е издадена и възпоменателната монета с неговото име: Emil Mörsch. Е. Mörsch учи строително инженерство в Техническия университет в Stuttgart от 1890 до 1894 г., след което работи до 1898 г. като държавен строителен мениджър в пътната и хидротехническата администрация на Württemberg.

През 1899 г. той полага успешно държавен изпит, след което става държавен служител и се премества на работа в мостовата служба при Държавните железници на Württemberg. От 1901 г. той е главен инженер и ръководител на техническия офис на Wayss&Freytag AG in Neustadt in der Haardt. Е. Mörsch пише теоретичната част от публикацията на Wayss&Freytag, „Der Eisenbetonbau, seine Anwendung und Theorie”, която се счита за едно от пионерските постижения в областта на теоретичните дискусии в областта на стоманобетона(Фиг.2). Две години по-късно той приема професорска длъжност по статика, мостови конструкции и стоманени конструкции в Швейцарската федерална политехника в Цюрих. През 1905 г. той е назначен за консултативен член на Германската асоциация за бетон. През същата година второто издание на „Der Eisenbetonbau, seine Anwendung und Theorie”, е публикувано от Е. Mörsch съвместно с Wayss & Freytag.

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Фиг.1. Emil Mörsch



Фиг.2. Der Eisenbetonbau, seine Anwendung und Theorie”,

В третото издание, което се появява през 1907 г., Е. Mörsch се явява като самостоятелен автор. Освен това през 1907 г. той става член на Германски комитет за стоманобетон(преименуван през 1941 г. на Германски комитет по стоманобетон). През 1908 г. Е. Mörsch се завръща в Wayss&Freytag. В допълнение към практическата си работа като директор и член на борда на една от най-важните германски компании за масивно строителство, Е. Mörsch продължава да се занимава с теоретични въпроси на стоманобетонните конструкции. През 1912 г. той успява да публикува четвъртото, отново преработено издание на „Der Eisenbetonbau, seine Anwendung und Theorie”. През 1916 г. Е. Mörsch се връща към науката и приема назначението като редовен професор по статика на масивни носещи конструкции, сводести мостове и стоманобетонна конструкция в ТН Stuttgart. До пенсионирането си през 1939 г. той публикува множество публикации, включително 5-то (1922) и 6-то издание (1933) на книгата си „Der Eisenbetonbau, seine Anwendung und Theorie”, която сега се е превърнала в стандартен документ, за работа в областта на стоманобетонните конструкции. В същото време Е. Mörsch остава тясно свързан с Wayss&Freytag AG, като влиятелен технически консултант. Важен плод на знанията, натрупани в тази дейност, е неговата публикация от 1943 г. „Der Spannbetontträger“, която е една от първите монографии на немски език по темата за „Предваритено напрегнат бетон”.

2. Реализирани мостови обекти от професор Е. Mörsch

Е. Mörsch участва, също така в следните проекти за мостове, като инженер, проектант и изпълнител, и то в съвсем млада за него възраст (Фиг.3, Фиг.4, Фиг.5 и Фиг.6).

2.1. Мост Isar (Изар) в Grünwald(Грюнвалд), 1903/1904 г.(Фиг.3)

В момента на откриването си, този мост представлява най-големият триставен дъгов мост в света. Проектът е представен от Е. Mörsch, едва когато е навършил 30 години. Общата дължина на моста е 221m. Ширината на пътното плато е 8,0m. Мостът

е с два отвора по 70m. Височината на ключа на моста над правата съединяваща местата на опорите е равна на 12,80m. Опорите представляват ставни устройства изпълнени от лята стомана. Дъговите конструкции са изследвани като нермиран бетон, натоварени предимно на натиск. Известно количество армировка е поставено от съображения за сигурност срещу появата на напрежения на опън. Дъгите представляват правоъгълни напречни сечения с размер във върха на моста равен на 80см, в мястото на опората са с височина 90 cm и 120 cm в мястото на най- големите разрезни усилия. Пътното платно е подпряно върху дъгите посредством стройни колони с напречно сечение 40/40cm.

2.2. Железопътен мост над Rhone(Рона) в Chippis(Чипис), Schweiz(Швейцария), 1906 г. (Froté&Westermann, Zürich)(Фиг.4).

2.3.Мост Гмюндертobel, 1907 г./1908 г. (Фиг.5).

Днес този мост се счита за пионер на модерните днес интегрални мостове, в които носещите части на конструкцията са свързани в едно интегрално цяло, съпротивляващо се със глобално поведение при поемане на външните товари, икономична експлоатация и висока естетика. Конструкцията представлява масивна дъгова запъната в основите си конструкция с отвор от 79,0m, ширина на пътното платно 6,90m, впоследствие разширено на 10,50m. От двете страни на голямата дъга мостът е достроен с две кръгови дъги от дясно и с 4 кръгови дъги от ляво. Пътното платно е повдигнато над дъгите с помощта на колони, стъпващи върху дъгите. Дъгите са със стрелка, равна 26,50m. Във върха дъгата е с ширина 6,50m и 7,50m при контрафорсите. Дебелината на дъгите при основата си е 2,13m и във върха е 1,20m при ключа на дъгата. Дъгата е подпряна в масивни фундаменти, сериозно вкопани в склоновете около средата на дефилето, които служат едновременно и за основи на големите колони, монтирани от двете страни на дъгите при основата си. Върху двете половини на дъгите са разположени по 6 реда, с включени в тях по 4 опори, които поддържат пътното платно. Позициите на тези опори съответстват на надлъжните греди под платното. Вътрешните колони са изпълнени с квадратно напречно сечение 50.50 cm, докато външните имат, поради архитектурни съображения, Т-образно напречно сечение, поставен с фланша на Т сечението(80 cm) от вътрешната страна на дъгата. Повърхността на външните опори следват повърхността на фасадата на голямата дъга. Всички подпори са кораво свързани както с дъгата, така и с пътната плоча на моста.

2.4. Мост Grafton, Auckland(Оукланд), Нова Зеландия, 1908/1910 (Фиг.6).

Мостът свързва двете новозеландски държавни магистрали. Това е стоманобетонен мост, пуснат в експлоатация през 1910 год. с дължина на свода 97,60m, претавляващ в онази епоха най големият мост от този вид в света.



Fig.3. Isarbrücke in Grünwald, 1903/04; **Fig.4.** Eisenbahnbrücke über die Rhone in Chippis, 1906, (Froté & Westermann, Zürich);

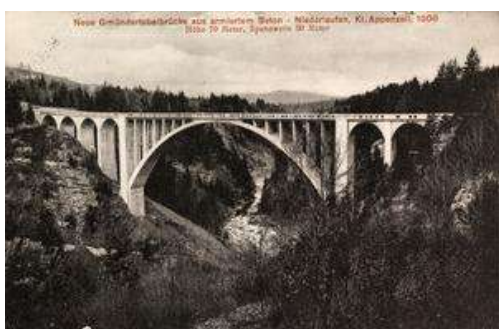


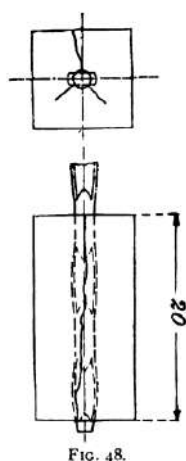
Fig.5. Gmündertorbelbrücke, 1907/1908; **Fig.6.** Grafton Bridge, Auckland, New Zealand, 1908/1910;

Железопътен мост Neckar(Некар) в Tübingen(Тюбинген), 1908/09, Prüfung und Detailbearbeitung des Entwurfs von Karl Schaechterle und Anton Jori(изпитване и работна документация на проекта от Карл Шехтерле и Антон Йори от офиса за изграждане на мостове на Генералната дирекция на Вюртембергските държавни железници). Други негови мостове са изброените по-долу в текста както следват: Мост над Дунав на Hohenzollerische Landesbahn, 1909; Долен мост Некар в Ротвайл, 1915 г.; Стоманобетонна облицовка на подлеса Spree на Weisentunels, 1931 г.; Мостът Розенберг в Хайлброн, 1950 г.

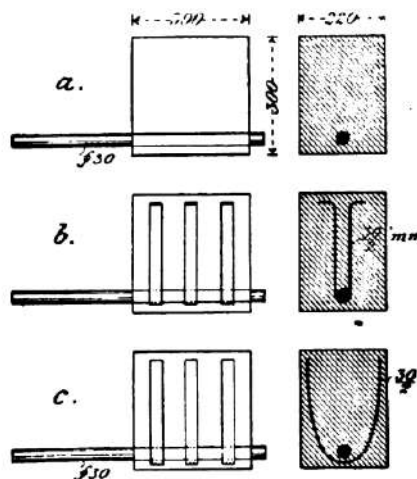
На основание многобройните направени експерименти, със стоманобетонни образци, E. Mörsch установява, че бетонът и стоманата действат съвместно, като статически добре обособен „композит”, който се съпротивлява на всички външни сили, като монолитна структура, при което се установява, че бетонът достатъчно добре поема предимно появилите се натискови напрежения, докато стоманата поема до голяма степен, възникналите опънни напрежения. С други думи, двата вида материали заедно се допълват в едно единствено цялостно творение-тяло: наречено стоманобетон. Друго отлично свойство на стоманобетонната конструкция е нейната якост на огъване. Освен това, на практика поддръжката на конструкциите от стоманобетон са били минимални(за онази епоха), което се е считало за основно предимство на стоманобетона. В комбинацията: стоманата, покрита с бетон, се е считало практически добре защитена срещу феномена на корозията. Някои изследвания на двадесет годишни канали в ст.бет.греди показват, че стоманата е абсолютно ненаранена и със същия цвят, както когато е напуснала завода при нейното валцуване.

3. Адхезия

Сцеплението между молекулите на повърхността на стоманата и бетона се нарича адхезия. За проектиране на стоманобетонни конструкции, стоманените пръти трябва да бъде така подредена по дължина, форма и дебелина, че да се осъществят безопасно пренасяне на напреженията върху бетона (Фиг.7 и Фиг.8).



Фиг.7. Тест за сцепление



Фиг.8. Адхезионен тест от Френска лаборатория

При еднопосочно армирани плочи краищата на армировъчните пръти се закотвят в големи участъци бетон, така че да няма опасност за намаляване на адхезията, поради преждевременно напукване на обгръщащият армировката бетон. В плочата, закотвянето е по-малко, но близо до краищата на гредите се въвеждат стремена, които обграждат бетона до известна степен, като по този начин съшият запазва адхезионната си сила.

4. Надлъжни деформации и съкращаване на бетона.

Съществуват напрежения, породени от температурни амплитуди или външни товари, които водят до разширяване и скъсяване на бетонните елементи.

Нормални напрежения σ - напречни на повърхнината на елемента и тангенциални τ - успоредни на повърхнината на елемента.

4.1. Надлъжна деформация на стоманобетона

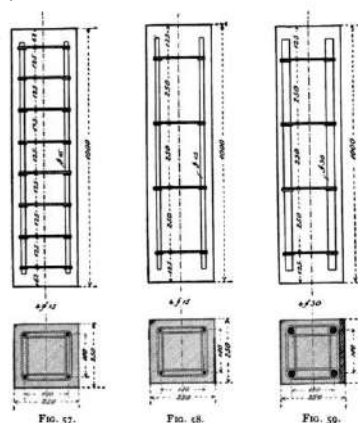
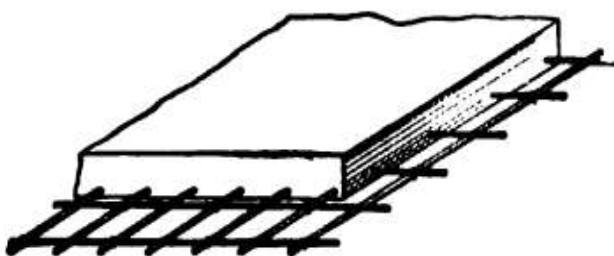
Експериментите, които се правят със стоманобетонни призми, в които разширението се получава чрез аксиално опънно напрежение имат няколко недостатъка. Те се състоят преди всичко в проблема за осигуряването на точно централно приложение на напрежението на опън и факта, че силата може да се прехвърли върху армировката само чрез големи адхезионни напрежения, така че краищата на образците се напукват преждевременно. С много по-голяма стойност са тестовите, при които разтегливостта на стоманобетона се установява чрез експерименти върху образци, подложени на огъване. От тях най-известен е Considere's. Първите му тестове са направени върху призми от бетон с квадратно сечение 6/6cm широчина и 60cm височина, подсилена от опънната страна с кръгли стоманени пръти.

Чрез правилно подреждане на армировката, тоест чрез поставянето ѝ в посоката на максималните напрежения на опън, якостта на срязване на стоманобетона може да се увеличи над това на обикновения бетон. В образци със слаба армировка, напрежението при разкъсване се повишава до граничната носимоспособност в стоманата, докато при по-тежка армировка такова напрежение не може да се достигне, тъй като използването на адхезия върху по-дебелите пръти не е достатъчно силно в краищата.

5. Конструктивни елементи от стоманобетон

5.1. Стоманобетонни плочи

Те представляват са най-простите стоманобетонни конструкции, изпълнени да поемат напрежения на огъване. Добре известно е, че горните слоеве на плочите са подложени на натиск, докато долните слоеве реагират на опън. В плочи, подпрени в краищата си, арматурните пръти трябва вървят успоредно на линиите на действие на напреженията на опън, и трябва да лежат възможно най-близо до дъното на плочата, както е прието в съответствие с правилната им защита. Напречните разпределителни пръти се използват за поддържане на носещите пръти правилно разположени по време на изграждането на плочата, и следователно двете серии от прътова стомана, са свързани заедно в точките на пресичането им (Фиг.8).



Фиг. 8. Стоманобетонни плочи

Фиг. 9. Стоманобетонни колони

5. 2. Стоманобетонни колони

Те представляват конструктивни елементи предназначени да поемат натоварване осово по своята надлъжна ос. Изпълняват се с надлъжна носеща армировка и стремена, които се връзват или заваряват за надлъжните пръти, при което се създават армировъчни скелети (фиг.9). Надлъжната армировка се изпълнява обикновено в диапазона от 12 до 40 mm. При размери на колоните до 40/40cm могат да се армират с 4 надлъжни пръта. Стремената на колоните служат за предпазване на надлъжната армировка от изкълчване и за осигуряване на по-монолитно стоманобетонно напречно сечение.

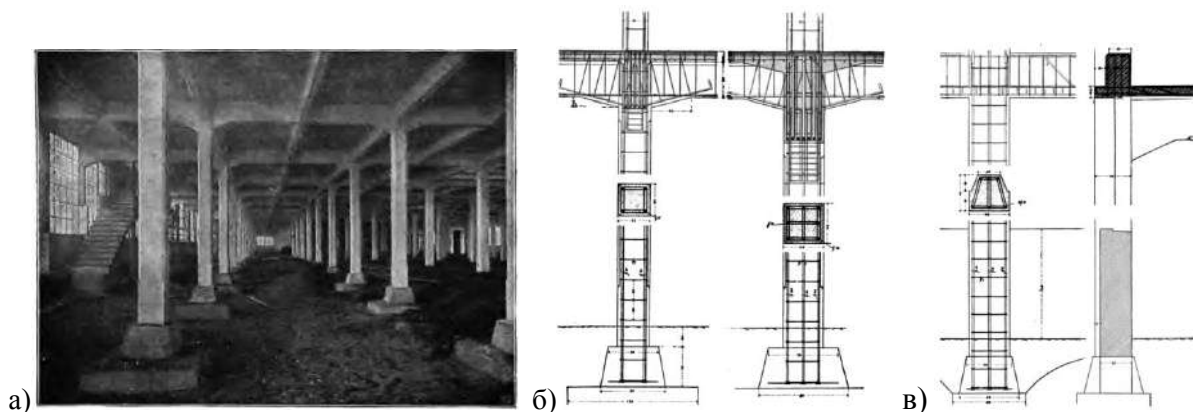
6. Приложения на стоманобетона

6.1. Исторически данни

Джоузеф Моние, който прилага за първи път стоманобетона през 1868 г., обикновено е приет за негов изобретател. Следи от този метод на строителство обаче се откриват в по-ранен период. Например, на Парижкото изложение от 1855 г. Ламбот излага лодка, изработена от стоманобетон. На Международното изложение от 1867 г., освен по-известния Моние, е представен и Франсоа Койне. От 1860 г. той проектира подове, арки и тръби, в изграждането на които основните принципи на стоманобетонната конструкция са разпознаваеми. На Моние обаче, принадлежи заслугата, че се е посветил на новия метод на строителство с постоянство и успех. Първоначално той е бил собственик на важна детска ясла в Париж. Първите му опити са да направи големи растителни вази, които да бъдат по-трайни от тези от дърво и по-лесно преносими от тези на цимента. Той се стреми да постигне целта си, като въвежда железни пръти с малък диаметър в циментовите страни на вазите и разширява този метод на изграждане до производството на големи резервоари за вода. През 1867 г. изважда първия си френски патент, който скоро следва с голям брой други на резервоари, подове, прави и сводести греди в комбинация с подове и др. В патентните му чертежи вече са разкрити всички елементи, които днес се използват в различните конструктивни детайли на съвременните строителни системи. Лесно е да се разбере как изобретението на Моние, базирано изцяло на емпирична основа, е предопределено да се развие в съвсем различни направления в ръцете на инженерите. През 1884 г. така наречените патенти на Моние са закупени от фирмите на Фрайтаг и Хайдшух в Нойшат на Хаард и на Мартенщайн и Жосо в Офенбах на Майн. Първата спомената фирма придобива правата за цяла Южна Германия, с изключение на Франкфурт на Майн и околността, чиято територия е запазена за тях от последната спомената фирма. В същото време и двете страни придобиват от Моние опции за цяла Германия, която обаче те отстъпват година по-късно на инженера Wayss. Последният, във връзка с гореспоменатите фирми, провежда тестове за натоварване в Берлин, резултатите от които са оповестени публично през 1887 г. в брошурата "Das System Monier, Eisengerippe mit Zementumhüllung", въз основа на която Ways успява да въведе системата Monier в публични и частни сгради. Появяват се първите стоманобетонни греди в комбинация с плочи още през 1886 г., във връзка с изграждането на библиотеката в Амстердам. Системата Моние (фиг. 8) използва множество разпределителни пръти, разположени под ъгъл към носещите пръти, като двете са свързани заедно в точки на пресичане. В последствие стоманобетоновия скелет на сградите доказва своята здравина и устойчивост на пожари.

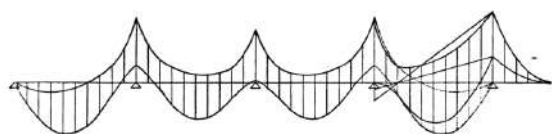
7. Исторически примери за проектиране и изпълнение на сгради и съоръжения подтикнали прогреса в стоманобетонните конструкции

На фиг. 10 е даден пример за изпълнение на основи от стоманобетон за фабрика за мотори Daimler близо до Щутгарт. В тази конструкция, натоварването за втория етаж е прието за 600 kg/cm^2 , което се прилага за изчисления на подовите конструкции, греди и трегери, като се отчете най-неблагоприятното разпределение от това натоварване върху стоманобетонните елементи. В тази връзка се предполага, че подовите плочи лежат свободно върху гредите, които от своя страна се подпират свободно върху колоните. По този начин, всички тези конструктивни елементи представляват непрекъснати греди с по-голям или по-малък брой полета. На фиг. 11 са показани положителни и отрицателни моментни диграми на непрекъснатата греда на четири полета с конзола в десния край. Направени са изчисления според таблиците E. Winkler, като е отчитено влиянието на натоварването върху конзолата.

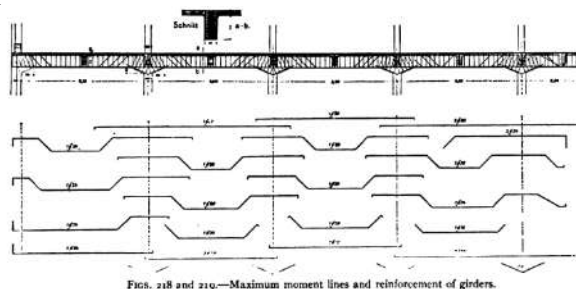


Фиг. 10а,б,в. Основи във фабрика за мотори Daimler близо до Щутгарт

Необходимата армировка отгоре и отдолу се определя въз основа на тези максимални моментови диаграми и е конструирана, както е показано на фиг. 12. Необходимото сечение на армировката в горната част над опорите, е осигурено от дълги препокривания на огънатите долни пръти.



Фиг.11. Моментова диаграма



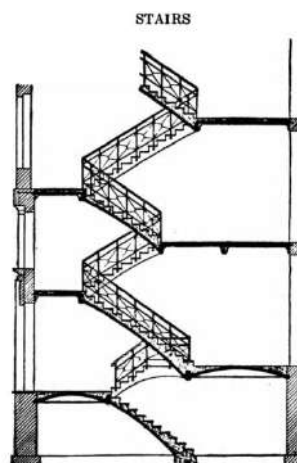
Фиг.12. Армировъчен план на гредата

На фиг. 13 е показано таванско помещение на ковашкия цех на фабриката за автомобили на Daimler. На Фиг.14 е дадено стълбище ст стоманобетон от XIX век.



FIG. 233.—Loft of the forge shop of the Daimler motor factory.

Фиг.13.Таванско помещение на ковашкия цех



Фиг.14. Стълбище

8. Методи на фундиране

8.1. Фундаментна плоча (разпределена основа)

За да се предотвратят лоши слягания, площта на основите на сградата трябва да бъде пропорционална на носещата способност на почвата. Ако товароносимостта на почвата е малка, изисква се много голям фундамент, който може да бъде осигурен чрез употреба от армирани плочи, притежаващи необходимата якост на огъване. По този начин е възможно да се намали натискът върху почвата до 0.5 kg/cm^2 , така че слабите строителни почви или насипаната земя да могат да се използват за основите. Фиг. 15 показва дъното на канализация във Висбаден с основа на 8 m дълбочина.

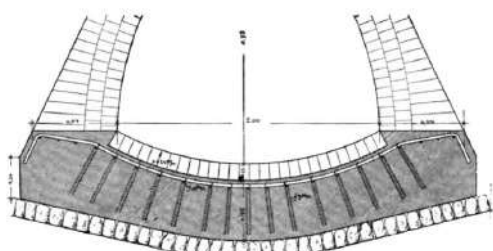


Fig. 254.—Reinforced concrete sewer base in Wiesbaden.

Фиг.15. Дъно на канализация

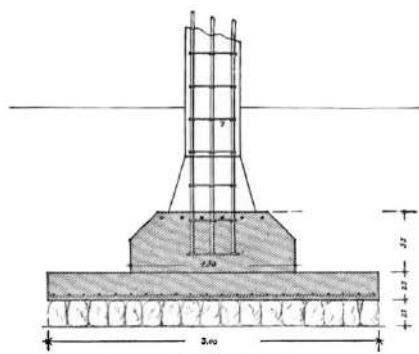


Fig. 255.—Silo column foundation.

фиг.16. Ст.бет. колона върху плоча

Тъй като почвата не може да бъде натоварена повече от 1 kg/cm^2 , цялото пространство между стените трябва да устои на натиска. Това се осъществява чрез изграждане на фундаментна плоча с дебелина 45см., простираща се от едната до другата стена. Разгледана от статическа гледна точка, плочата представлява греда, подпряна в двата си края и огъната от натиска на земята нагоре. Необходимата армировка за метър дължина на канализацията е 10 пръта 24mm в диаметър, които поради използването на кривината на повърхността, трябва да бъдат закотвени в бетона посредством стремена с 7 mm дебелина. Под стоманобетонния фундамент е изпълнена основа от 15 cm, слой от камък на вароциментов разтвор. Разпределените основи (фундаментните плочи) трябва да се използват, когато по някаква причина натоварването от конструкциите е неприложимо за земната основа. На Фиг. 16 е показана под ред от силосни колони нормалната форма на стоманобетонната основа. В този случай е необходимо да се разпределя концентрираното натоварване от колоните равномерно: надлъжно и напречно, т.е., с други думи е необходимо укрепване и в двете посоки. Надлъжната армировка служи за разпределяне на концентрираните товари върху пространството между колоните. На фигура 17 са показани детайли на армировката, особено разположението на армировката в гредата, определена да поеме срязващите напрежения в бетона.

9. Пилоти

Поради високата си якост на натиск, бетонът е особено подходящ за пилоти(фиг.18). Спиралното разположение на стремената е най-устойчиво. Сечението обикновено е кръгло или осмоъгълно. Предимството на стоманобетонните носещи пилоти и шпунтови пилоти в сравнение с дървените е, че бетонните могат да се използват над нивото на подпочвените води, като по този начин се спестяват значителна част от височината на фундамента.

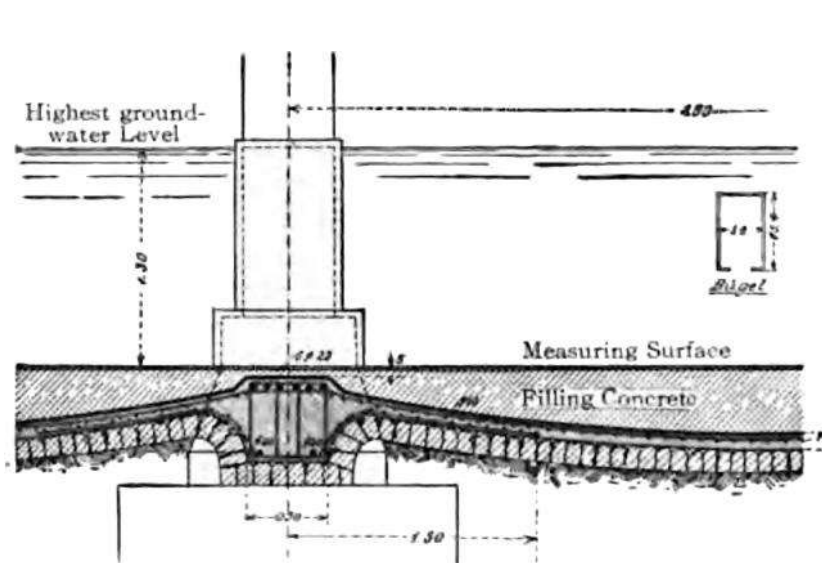


FIG. 262.—Section through beams and arches of a water-tight cell

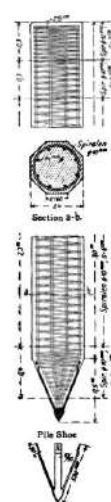


FIG. 264.— Reinforced concrete pile for the top of the canal in Mulhausen in Alsace.

Фиг.17. Разрез през гредите и дъгите на

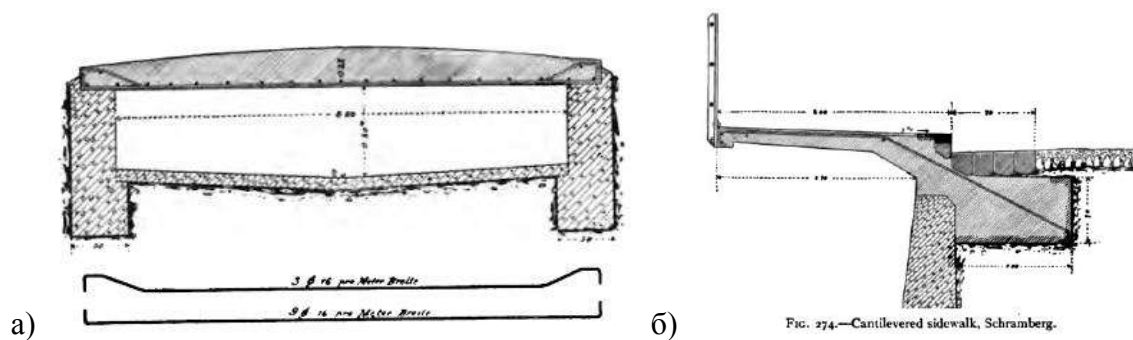
водонепропускливо мазе

Фиг.18. Пилоти

При армирането надлъжните пръти се съединяват в острия си край в желязна обувка. Добрата връзка между надлъжните пръти на разстояние един от друг е много важна. Допълнително, така че да се разпреди ударът по време на движението, е необходимо да се постави някаква среда между чука на пилотника и пилота, както и да се използва плътна шапка, която да обхваща главата на пилота, за да се предотврати разрушаването му. Тази разновидност на пилотите е въведена при изкопни работи от Hennebique. Подробно е описана от Diemling през 1904г.

10. Водостоци и мостове

С въвеждането на стоманобетона, каменните устои и дъгите от камък на мостовете се заменят с новопроектирани стоманобетонни, при което плочогредовите водостоци стават популярни тъй като с помощта на армировката е възможно бетонната плоча да поеме напреженията на огъване с разстояние между подпорите до 6,5 m, като се увеличава и ширината на пътното платно, а с помощта на Т-образните греди обхватът може да бъде увеличен още повече. На фигура 19 е показана конструкцията на плочогредов водосток със светъл отвор 4 m под пазара в Laurheim. Плочата е с дебелина 35 cm е изчислена за парен валяк. Тя е изпълнена с 12 пръта, 16 mm в диаметър на метър ширина (9 са прави и 3 огънати нагоре в краищата). Повърхността е оформена като корона за отвеждане на повърхностните води. При тестовото натоварване не е установено никакво осезаемо отклонение. Много подобни панели се поставят под улици, с ширина до 6,5m.



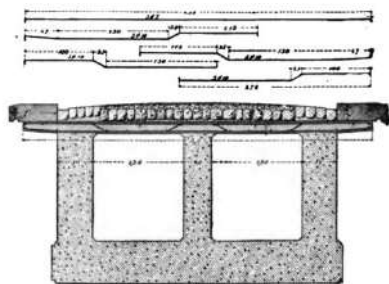
Фиг.19а,б. Плочогредов водосток-а)

Тротоарна част на водостока-б)

Стоманобетонните плочи могат да се използват не само като елементи между две опори, но и когато са закрепени в единия си край, а другият стърчи свободно - конзолно. Тази форма на конструкция може да се използва например за разширяване на улица покрай река. В този случай тротоарът може да се остави да стърчи над реката, както е показано на Фиг. 19б. В този случай армировката трябва да се постави близо до горната повърхност и да се закотви в бетонен блок зад стената на реката. Блокът трябва да е с достатъчни размери, за да предотврати преобръщането. Такова съоръжение е изградено от Wayss Freytag в Wildbad. Стоманобетонните плочи също могат да се използват с предимство като пътна настилка върху стоманени пешеходни мостове и канали, а също и за изграждане на пътна настилка на основните пътни артерии на мостовете, вместо стоманени пътни конструкции. Плочите за тротоарите в повечето случаи се изработват предварително и след това се полагат. Те могат да бъдат направени с циментова настилка или да се положи асфалтова настилка след поставянето им.

11. Дъгови конструкции

На фиг.20 и 21 са посочени решения на стоманобетонни дъгови мостове в Хамбург и Гунзесрид. Анализирани са напречните сечения на конструкциите им.



Фиг.20. Мост на магистрала близо до Хамбург, Разрез през носещата конструкция и пътната настилка.



Фиг. 21. Мост на магистралата в Гунзесрид (Алгау) - Германия. Междуосие 32 м

Осите на ребрата на тези мостове са приети като параболи, за да могат да се приложат с предимство известните формули за непосредствено изчисление на линията на влияние за моментите в определено напречно сечение на параболичната дъга. Тъй като обикновено осите на дъгите се приемат така, че да съвпадат с линията на натиска за собственото натоварване, в горните случаи се приема, че формата на параболата е допустимо избрана, тъй като собственият товар е малък и почти равномерно разпределен по целия диапазон. Трябва да се отбележи особено, че тук връзката с брега е осъществена посредством бетонни стени, които затварят кухо пространство, покрито

със стоманобетонна плоча. По този начин се намалява земният натиск върху стените на коритото.

На фигура 22 е показан магистрален мост над река Изар край Грюнвалд – Германия.



Фиг. 22. Магистрален мост над река Изар край Грюнвалд – Германия

Мостът на магистралата е с дължина около 220 м., с две арки с размах от 70 м., над коритото на река Изар и захранващия канал за електрическата централа. Двете главни полета са проектирани като арки с три ставни устройства. Пътното платно е издигнато над проектната кота с помощта на колони, подпрени върху дъгите на моста.

Технологията за изпълнението на дъгови мостове е показана на Фиг.23.

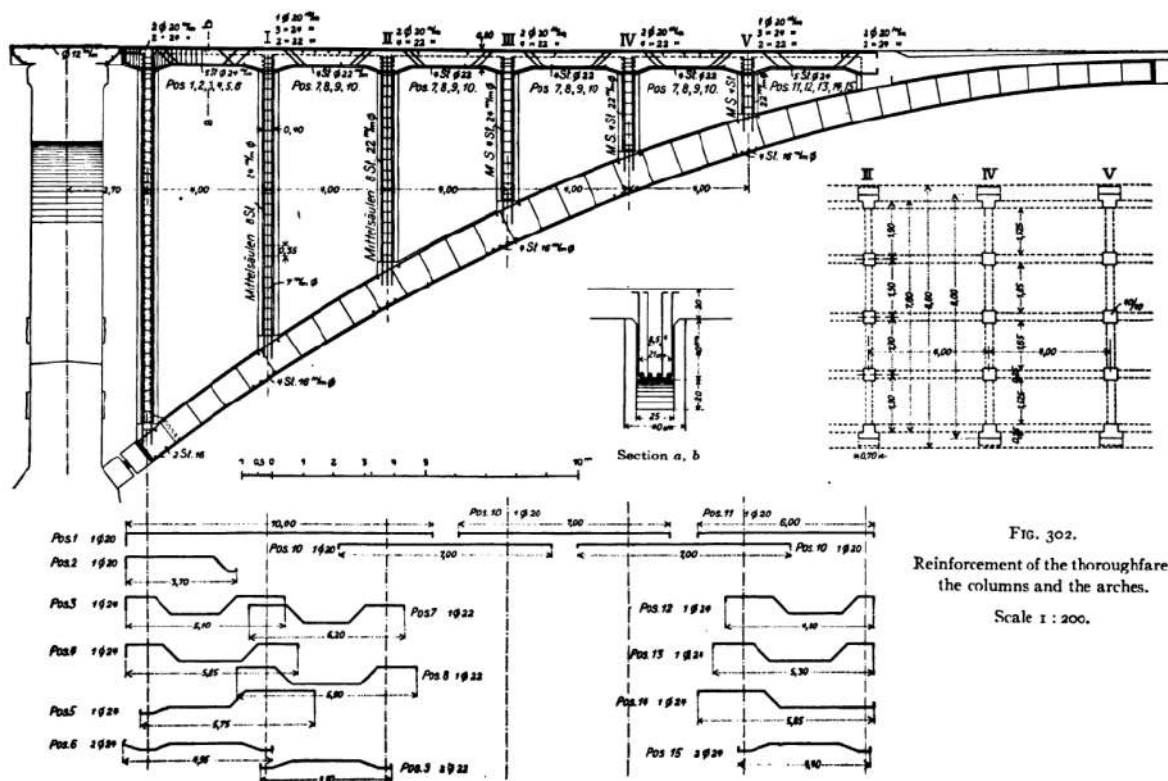


FIG. 302.
Reinforcement of the thoroughfare,
the columns and the arches.
Scale 1 : 200.

Фиг. 23. Кофражен и армировъчен план на дъгов триставен дъгов мост

Триставния свод влиза в мостовото строителство около 1880 г. и днес се радва на доста широко разпространение. През 1900г. Щутгартския професор d-r h.c. Emil Mörsch представя метод за изчисление на триставни сводове в същността на който е определянето на такава сводова форма и на такива дебелини в отделните напречни сечения, че максималните стойности на натисковите напрежения на долния и горния ръб на всяко сечение да са едни на други равни и почти равни при всички разреза, т.е. да се получи свод с еднаква сигурност. Професор Мьорш е един от пионерите в областта на стоманобетона, а приносът му към статическото изчисление на триставни сводови мостове ще остане и в бъдеще незаменимо научно познание в ръцете на съвременния строителен инженер.

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**За живота и творческия път на проф. д-н. инж. Милчо Брайнов,
по случай 100 години от неговото рождение**

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¹ *University of Architecture, Civil Engineering and Geodesy*

¹ *Университет по Архитектура, Строителство и Геодезия*

Абстракт: *Относно космополитната научна дейност на проф. д-н. инж. Милчо Брайнов, като изтъкнат и всепризнат учен в областта на строителните конструкции у нас и в чужбина, по случай 100 години от неговото рождение. Опит за разясняване на философията на конструктивното инженерство, залегнали в основата на цялата му научна дейност, практика и публикации.*

Key words: *Милчо Брайнов, философия на конструктивното инженерство*

1. Въведение

Проф. д-н. инж. Милчо Брайнов е роден на 28.09.1922г в гр. Русе. Гимназиално образование завършва през 1941г. в Русенската мъжка гимназия с отличен успех. В периода 1941-1944г. отбива военната си служба в ПВ артилерия. Като войник през 1943г. е преводач при обучението на български взвод войници по нова прожекторна и радарна техника в Германия. След обучение в школа за запасни офицери е назначен за командир на взвод до м. юни 1944г. Същата година се уволнява и с конкурс постъпва за студент по Промислено и гражданско строителство. Завършва през 1949 год., с отличен успех като първенец на випуска и получава диплома за Строителен инженер-конструктор. Като студент е водил аудиторни студентски подготвителни курсове за изпити по Строителна механика и Стоманени конструкции; ръководител-участник в студентски стаж по възстановяване на мостове в Будапеща (1948г.) работил е в служба „Внос на строителни машини“ в „Металимпорт“ През 1950г. с конкурс е приет за асистент в катедрата. Последователно е ст. асистент и преподавател. През 1958г. участва в конкурса за трима доценти, към катедрата. С протокол №11 от 25.06 1958г. на Академическия съвет на ИСИ е избрани за доценти с 15 гласа при присъствали 15 члена на съвета с право на глас. Няма данни с какви научни трудове е кандидатствал в конкурса. Като доцент е чел лекции на студентите от специалността ПГС по дисциплините „Дървени и стоманодървени конструкции“, написан и издаден учебник „Техника“ 1962 год., и 1970г. „Стоманени мостове“, а от 1967г и по „Стоманени конструкции“. Избран е за професор през 1966г. След пенсионирането на проф. Попов през 1965г. е избран за ръководител на катедрата (1966-1987год.)

Под неговото ръководство катедрата добива съвсем друг облик- попълва се с нови преподаватели и асистенти, създава се и се развива лабораторията, издаване на нова учебна литература, осъществяване на сериозна научни-изследователска дейност, активно участие при изграждане на отговорни обекти в държавата, увеличаване на международните контакти на катедрата със сродни катедри от други страни, Съавтор е на учебника „Стоманени конструкции“, от издателство „Техника“ год. 1991 год. в

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който е написал главите: Стоманени строителни конструкции, Проектно-конструкторската дейност, Науката и теорията за конструкциите и Прътови конструкции.

Бил е избран за зам. декан на Строителния факултет, зам. и и.д. Ректор на Университета, Председател на Централната техническа библиотека, Редактор е на годишника на УАСГ.

Ръководител на общо 12 аспиранти, защитили успешно дисертации.

През 1982 год., след подготвен и защитен хабилитационен труд на тема „Конструкции, биконструкции, конструктивни системи” получава научната степен „Доктор на техническите науки”. Официални рецензенти на труда са з.д.н.т. проф.д.н.т. Евгений Иванович Беленя; з.д.т. проф.к.т.н Иван Димитров Бончев, проф. к.т.н Димитър Генчев Димитров и проф.к.т.н. Тодор Димитров Карамански

Като такъв започва неговата космополитна и научна дейност на изтъкнат и признат учен в областта на строителните конструкции. В основата на цялата му дейност и в публикациите му той поставя философията на конструктивното инженерство. Мащабността на тази дейност излиза извън рамките на тази история, ето защо като Приложение към нея включвам „Равносметка”-80 години-Живот и време-Дейности, подготвена от него за честване на 80 годишнината му. По надолу ще маркираме само основни дейности в държавните органи и институции:

* Ръководител на Управление „Наука и технически прогрес”, член на Колегиума, На национални делегации в Международни конгреси и др. (1963-1964).

* От февруари 1969г до ноември 1971г. – зам. министър на Висшето образование и науката;

* В периода 1972-1983 – зам. председател на Комитета за Култура по инвестициите и икономиката. В тази област ще отбележа само Националният дворец на културата. За да се гарантира качествени и навременно проектирана на сложните конструкции по негово предложение беше създаден колектив от преподаватели към катедрата с ръководител автора на тази дейност. (Постиженията на катедрата ще се изяснят по-късно);

* Съветник в Министерския съвет по Образованието и науката;

* Секретар на Секция Технически науки на съюза на Научните работници в България;

* Вицепрезидент на Международната асоциация по конструктивно инженерство и мостове (IABSE-1985–1991)(Фиг.1 и Фиг. 2);



Vice-President of IABSE: Prof. **Milcho Brainov, Bulgaria(1985–1991)**

The Bylaws Committee 1989: **Milcho Brainov(1922-2011)**., Yukio Maeda, Tippur Subba Rao, Jorg Schneider (Chair), Giorgio Macchi; Alain Golay (Executive Director)

* Член на бюрото на Президиума на Съюза на учените в България;

* Основател и пръв председател (1990-1996) на Съюза на Строителните Конструктори.

За дейността му в Комитета за култура е удостоен със званието **”Заслужил деятел на културата”**.

2.Преглед на основните научните приноси и литература

Темите които разработва могат да бъдат групирани както следва:

1.Теоретично изследване и практическо разработване на научно-методологическите положения за формирането и развитието на Комплексната наука и систематичната теория за строителните конструкции:

* „Конструкции-комплексна наука, систематична теория, конструктивно творчество”; Издателство на БАН 1980 год.;

* Природа-цивилизация- екология: материална култура-техническо творчество-конструктивно инженерство-инженерно конструктивни дейности- ССКБ-София, 1990 год.;

* Намалвяване на рисковете от природни бедствия-преди, а не след тях-чрез конструктивното инженерство- ССКБ-София,1990 год.;

* Някои Предварителни Мисли по Предложенията на д-р Франк Прес(председател на Националната Академия на Науките на САЩ-до М.Брайнов) за Международната Декада за Природни Бедствия, БАН кн. 4 1987 год.;

*Стандартизация, открита, универсална, динамична, композиционна конструктивна система доклад, Москва, 1976г.

* Структурни Проблеми на Научната Организация на Висшето образование и на Образователната система. Техника-София-1971г.

2. Теоретични изследване на биконструкцията и създаване на обща методология и конструктивно-изчислителен анализ за тяхното напрегнато и деформирано състояние;

* Конструкции, биконструкции, конструктивни системи- дисертационен труд за присъждане на научната степен „Доктор на техническите науки”;

3. Теоретично изясняване на проблема „човешки поселения-сгради-конструкции-технологии” с изследване на условията, предпоставките, изискванията и принципите за решаване на съвременните градоустройствени проблеми;

* „Човешки поселения Високи сгради: архитектура, конструкции, индустриализация Конструктивни системи”, „Техника”, 1977 год.

* Конструктивно Инженерство: Наука-Проектиране-Практика-Общество-Издателски център София-1989г.

* Строително-конструктивната сигурност, Природните бедствия и Технологическите аварии-Ролята на Държавата)изводи от Световната конференция нас ООН, Япония 1994г). Списание на БАХ, кн.2, 1996 год.

4. Развитие на теоретичните модели и практически подходи за определяне конструктивно-технологичната оптималност на металните конструкции.

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* Брайнов М. Биконструкции – противовеетрови, противоземетръсни, стабилизиращи дискови връзки, Техника, София, 1981 год.

* Брайнов М. Човешки поселения, високи сгради, архитектура, конструкции, индустриализация – сградни конструктивни системи, Техника, София, 1977 год.

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4.Избрани публикации и доклади

* Брайнов М. Методически предимства, технико- икономически особености и възможности за усъвършенстване на оразмерителния метод по гранични състояния. Юбилейна научна сесия, 1968г.

* Брайнов М., А. Малюк .Усъвършенстване на метода за оразмеряване на строителните конструкции по гранични състояния, Архив МСА и ВИСИ-ЦТБ, 1965 год.

* Брайнов М. Методика за определяне ефективната област на стоманените конструкции в условията на съответната ефективна конюнктура, Юбилейна научна сесия, 1968 год.

* Брайнов М. Структурни проблеми на научната организация на висшето образование и на образователната система, , София, 1971 год.

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* Брайнов М.. Строителна механика на системи с податливи съединения – част първа от “Изчисляване на статически неопределими конструкции с податливи съединения”, Годишник на Инженерно-строителен институт, София, 1955 год.

* Брайнов М. Свободно висящи мрежи с единична кривина. Юбилейна научна сесия, 1968 год.

5. По отношение на Строително-конструктивната дейност

Следва да се отбележи:

* Участие при строителството на Дунав Мост –Русе (1953-1954 год.);

* Ръководител на колектива на ВИАС, изготвил два варианта в конкурса за Аспаруховия мост. Спечелил конкурса е единият от проектите. Ръководител на колектива проектирал моста.

* По негово предложение, като зам председател на комитета по култура, бе създаден основен авторски колектив за конструкцията на НДК в състав проф. М. Брайнов, проф. Тодор. Карамански, проф. Веселин Венков, доц. Любчо Венков и инж. Богдан Атанасов, чиято задача беше да одобрява всички възникнали при строителството проблеми, свързани с конструкцията. Участваше активно при обсъждане на всички проблеми, отнасящи се до проектирането, в колектива от катедрата. С негово участие и свикан от него експертен съвет беше изменено основното

решение, прието в ТИД, по отношение на пода на зала и тавана на зала 3; ръководител на работните групи по проектиране на конструкциите; като зам. председател по инвестициите на Комитета за култура, по специално решение от 18 04 1978 год., инвеститор на НДК, с всички пълномощия да го представлява пред всички инстанции, да решава всички въпроси и проблеми, да създава необходимата организация за провеждане на инвестиционния процес.

6. Послеслов:

1. Интересуващите се, които искат да се запознаят с всички обстоятелства от завещанието на г-н Евлоги Георгиев до откриване на Висшето техническо училище, могат да ползват записките на проф. инж. Георги Стефанов, почетен доктор на Техническият университет-Виена- „Половин век висше техническо образование по архитектура, строително инженерство и геодезия”- издание на Полиграфическата база при УАСГ.

2. За Историята на Катедра „МДПК” са ползвани архивни материали на катедрата от проф. Л. Венков.

Приложение:

(Гореизложеният текст представлява извадка от обширния доклад изнесен пред Осмия национален симпозиум по Стоманени, дървени и комбинирани конструкции 10-11.11.2022г., от проф. д-р инж.М. Цанков).

А. Хронология по създаването на Висше Техническо Училище

Годината е 1882. Българинът Евлоги Георгиев- търговец, индустриалец и банкер, живеещ в Букурещ, осъзнава необходимостта в България да се създаде училище, в което научните предмети, които се преподават, да бъдат предимно от приложните науки, с насоченост към индустрията. За реализиране на тази необходимост той прави завещание от 6 милиона златни лева с изискване бъдещото училище да носи името „Братя Евлоги и Христо Георгиеви от Карлово”. В завещанието си той пише „Само надеждата ми, че ще мога и аз да участвам в преуспяването и величието на отечеството, ме прави да умира спокойно, защото съм работил през всичкия си живот и съм спестявал, както аз и брат ми, ние сме били въодушевявано само от това желание, което сега остава да се осъществи от наследниците ми и от изпълнителите на завещанието”. Освен това Евлоги Георгиев подарява мястото, върху което по- късно е построен ректоратът и дава 8 хил. лв. за изграждане на сградата.

Девет години по късно, през 1891 год. Българските инженери и архитекти се обединяват в български съюз на инженерите и архитектите (БИА). Съюзът започва сериозно да се застъпва за създаване **на висше техническо училище**. Появяват се много публикации на видни инженери като Никола Сарафов и Юрдан Данчов.

Двадесет години по късно, през 1911 год. фондът от 6 мил. златни лева е нараснал на 15 мил. златни лева. Настоятелството, което обслужва завещания от частни лица, дълго умува да се създава ли самостоятелно висше техническо училище независимо от университета и как да се разпределят средствата от двете завещания. Но окончателно нищо не се решава.

Тридесет години по късно, а именно през 1941 год. Съюзът БИА (с решение No: 4764/ 15.04.1932 год., на Министерството на Вътрешните работи и Народното здраве е утвърден Устав на Българското инженерно архитектурно дружество – БИАД/, подготвя нов проектозакон, който се внася в Народното събрание от министрите на Обществените сгради, пътища и благоустройството и на Народното просвещение

Законът е приет и се публикува в **Държавен вестник от 12. 06. 1941 год.** Основава се Висше техническо училище (ВТУ) в София с два факултета: Строително- архитектурен и Машинно-технологически. В Строително-архитектурния факултет се предвиждат три отдела: строително инженерство, архитектура и земемерно инженерство. За пълно откриване на ВТУ се определят 5 години , но още на следващата година на **04.10.1942** год в Аулата на Софийския университет в присъствието на :

- Негово величество цар Борис III;
- Негово царско височество княз Кирил;
- Негово високо преосвещенство софийския митрополит Стефан;
- Министър председателят проф. д-р Богдан Филев;
- Председателят на Народното събрание г-н Христо Калфов;
- Министъра на народното просвещение г-н проф. Д-р Борис Йоцов;
- Министъра на Обществениите сгради, пътища и благоустройството г-н инж. Димитър Василев;
- Министъра на железниците и пристанищата г-н инж. Васил Радославов;
- Ректорът на Университета г-н проф. Димитър Кацаров,;
- Директорът на Художествената академия г-н проф. В Захариев,
- бивши министри, професори, общественици, официални лица, видни граждани и много инженери и архитекти

Председателят на Нарочната комисия г-н д-р инж. Юрдан Данчов в качеството на изпълняващ длъжността Ректор дава отчет за готовността за започване на учебните занятия на Висшето техническо училище.

Г-н проф. д-р инж. Тодор Радославов изнася първата академична реч на тема „*Водите като национално богатство и възможностите за тяхното използване у нас*” .

Новото **Висше техническо училище** в София е официално открито.

Висшето техническо училище през годините е, претърпяло следните наименования:

- **Висше техническо училище (ВТУ)** (1942-1945);
- **Държавна политехника** (1945 – 1953);
- **Инженерно строителен институт (ИСИ)** (1953-1963);
- **Висш инженерно строителен институт (ВИСИ)** (1963-1977);
- **Висш институт по архитектура и строителство (ВИАС)** (1977-1992)
- **Университет по архитектура, строителство и геодезия (УАСГ)** (1992 и понастоящем).

Б. Хронология по създаването на: Катедра „Метални, Дървени и Пластмасови Конструкции”

Историята на катедра ”Метални, дървени и пластмасови конструкции” през годините очевидно е била тясно свързана с развитието на Висшето техническо училище. Името на катедрата също е претърпяло промяна.. В материалите с които се разполага няма информация кога Строителния факултет е създал тази катедра. Логично е създаването на катедрата да е свързано с назначаването на нейния първи ръководител.

Ректората на Висшето техническо училище по надлежния ред прави предложение за избор на извънреден професор по **"Статика на стоманените и дървени конструкции"**.

Конкурсът е обявен в **Държавен вестник бр.117 от 23 05.1945**. Очевидно това е било и бъдещето наименование на катедрата в този период.

Документи за участие в конкурса подават:

- инж. Христо Василев Попов ;
- д-р инж. Борислав Василев Банков;
- д-р инж. Исак Мошев Браджали.

За рецензенти по обявения конкурс са избрани: проф. д-р инж. Пейчо поп Петров и д-р инж. Васил Т. Стоилов.

И двамата рецензенти на основание на представените материали препоръчват да бъде избран за **извънреден професор инж. Христо Василев Попов** и не препоръчват другите двама кандидати. И тримата членове на комисията гласуват с да предложението на рецензентите.

Първата промяна на името на катедрата се извършва през **1950 г.** След формиране на обща катедра „Строителна механика” тя се преименува в **катедра "Стоманени и дървени конструкции"**.

През **1971г.** катедрата приема настоящето наименование **"Метални, дървени и пластмасови конструкции"**.

PS.



Fig.2. Vice-President of IABSE: Prof. Milcho Brainov, Bulgaria(1985–1991)is in center, left is: Prof. Fritz Leonhardt (1909-1999)

President Hans von Gunten created the “President’s Club” that held its first **gathering** at the Helsinki Congress in 1988.