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FIRSTS POST-TENSIONED SLAB BRIDGES IN POLAND THE BRIDGE IN PAWIA STREET IN LUBLIN

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The analysis was focused on three post-tensioned slab bridges, constructed in 1950s. Two of them function normally and will probably achieve the life span of 100 years required by the relevant regulations. The third one will likely be demolished soon and replaced with a new reinforced concrete frame bridge. To its degradation contributed the faulty diagnosis of its technical condition during its periodic technical inspections. The introduction briefly characterises the development of the prestressed structure theories reviewing papers on concrete rheology and monographs looking into prestressing. The paper is based on the existing fragments of the technical design documents concerning the bridges in question. The bridges were designed by Polish civil engineers.

Keywords: bridge maintenance, first Polish post-tensioned concrete bridges, bridges in Pawia Street in Lublin

1. INTRODUCTION

Although the idea of prestressing concrete (PC) beams and slabs was born at the turn of 19th century, its effective implementation was hindered by material difficulties. Adequate high-carbon steel necessary for the production of high strength tendons, i.e. of the minimum tensile strength f_u ≥1500 MPa, was unavailable. At the same time, prestressed steel had low stress relaxation - nowadays assumed to equal 2.5% at the maximum.

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PC by metal rods was first patented by P. H. Jackson in US (1872) with regard to joining individual concrete blocks by means of tendons, [1]. In Germany, in 1888, C. F. Doehring patented prestressing a slab with metal cables, [2]. In 1903 (or 1909), Ch. Rabut [3] constructed a street strand laid on a PC cantilever of the outreach of 7 m, with traffic on the top. E. Freyssinet (1879–1962) is considered to be the “father” of PC. After 1930, he constructed numerous prestressed structures, including bridges. It sounds simplistic nowadays, but the assumption that high strength steel must be used for prestressing is to his credit.

The history of prestressing technology can be traced by analysing technical papers and monographs in the field. At the beginning of 20th century, rheological processes in concrete were not sufficiently identified. It is enough to say that the fundamental publications for the field appeared much later, e.g. W. H. Glanville (1937) [4], D. McHenry (1947) [5], F. Dischinger (1949) [6], A. M. Freudenthal (1950) [7], E. Freyssinet (1951) [8]. H. Trost’s creep theory [9] applied nowadays, also in Eurocodes, was published in 1967.

The first handbook on prestressing was published by E. Freyssinet (1936) [10]. The monographs by E. Mörsch (1943) [11], W. Ritter and P. Lardy [12] (1948), G. Magnel in (1948) [13], P. W. Abeles (1949) [14] J. Barets and E. Freyssinet (1950) [15], Y. Guyon (1953) [16], B. V. Jakubovskij (1954) [17], F. Leonhardt (1955) [18] were successively issued.

In Poland, the first monograph on prestressing concrete was S. Kaufman’s book [19]. The monograph by W. Olszak et al. (1961) [20] is the most general one and thanks to that fact it is still relevant today.

In 1952, two post-tensioned slab bridges were put into service. The first one, 12.6 m long, located in the town of Końskie, was designed by T. Kluz. The other one, designed by Cz. Eimer, was erected in Batowice near Kraków. The span of this bridge amounts to 18.9 m. Both bridges are considered to be the first Polish prestressed structures of this kind. Apart from their length, the two bridges differed in terms of the technology of anchoring prestressing cables. In the case of Kluz's bridge, only straight cables anchored according to Freyssinet's concept were used. In the longitudinal section, the height of the slab varies vertically in a manner very similar to Jackson’s arched beam. Magnel type anchors were used in the Eimer's bridge, both with regard to straight and bent cables. Due to its poor technical condition, the bridge in Batowice was demolished in 1990.

The Warsaw University of Technology started offering lectures on PC in 1946. The 1950s witnessed a dynamic development of the prestressing technology, including research, design and realisations. The first conclusions with regard to the progress in the area of PC at that time can be found in papers [21-23].

In [23] this initial PC period was summarized by indicating the following facts:

– commissioning of pre-tensioned slabs and beams factories at the beginning of 1950s,
– bridge construction using pre-tensioned concrete inserts/panels (1953),
– prototype solution of the oblique slab bridge in Chwaliszów (1954) with a span of 15.80 m,
– the simple box-beam bridge in Bydgoszcz with cantilever ends, spans: 12.0 – 40.0 – 12.0 m, designed by M. Wolff and commenced in 1963, using the overhanging technology,
– implementing the production of prefabricated bridge element systems.

In the mid-1950s, prefabricated prestressed beams began to be produced. Initially, they were post-tensioned concrete beams, and then the technology of pre-tensioned concrete girders was also developed. At present, from the perspective of the past, it can be stated that only the prefabrication of longitudinal prestressed elements, i.e. bridge I- and T-girders, had and still has a rational justification.

The use of prefabricated load-bearing elements always leads to the unification of the appearance of bridge structures. Typical solutions lead to significant technological simplifications and cost reduction. On the other hand, the repetition of the typical bridge image can provoke criticism in terms of aesthetics. This was also the case with the prefabrication systems listed below.

The *Kujan* system (1954) [24-25], based on post- and pre-tensioned beams with sections in the shape of inverted T. In this system, a beam is also a girder (analogous to the Melan System) and a self-supporting framework for the forming of the slab structure. The *Kujan* beams are still in use.

Another typical solution, widely applied, are I-WBS beams. Initially, they were produced as post-stressed concrete, later also as pre-stressed concrete. Beam spans range from 12.0 to 39.0 m with a 3.0 m module. At the same time, pre-stressed T-beams called *Płońsk* with spans of 15.0 and 18,0 m are used.

Using *WBS* beams (h = 2.1 m) with a span of 39.0 m, S. Filipiuk designed the access flyovers to the steel bridge over the Vistula River in Chełmno. The bridge was put into operation in 1963.

In 1963, the same designer designed a bridge over the Vistula River in Annopol. On the access flyovers to the current spans he used freely supported prefabricated spans with the beam length of 40.0 m. The current spans of 53.0 – 67.0 – 53.0 m were obtained by creating the RC or PC continuity joining basic prefabricates. This is one of the most interesting bridge structures from the 1960s in Poland.

Fig. 1. shows a longitudinal bridge section sourced from paper [22], including the dimensions and arrangement of longitudinal and transverse cables. The basic bridge parameters are as follows: length L=18.1 m, theoretical span L_t=16.9 m, width B=8.0 m, fixed panel height h=0.6 m. The

right-hand bevel of the bridge is equal to $\alpha = 72^{o}$. The 55 longitudinal tendons of the aggregate prestressing force of 2600 T (25 MN). 5 transverse cables parallel to the bridge bevel, are located at the mid-height of the plate. As it will be apparent, such a solution is similar to the one applied in the bridges discussed further.

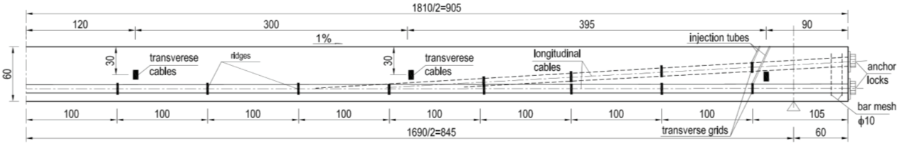


Fig. 1. Prestressed concrete bridge slab, the course of prestressing cables [22].

The list of publications presented here is significantly abridged, as there are hundreds of papers written at the beginnings of the use of prestressed structures, e.g. see W. Trochymiak [25] and J. Biliszczuk [26].

2. FIRST PRESTRESSED BRIDGES IN LUBLIN VOIVODESHIP

A pretext for the paper was furnished by the bridge in Pawia Street in Lublin, soon to be demolished and replaced with a new one. Nevertheless, there are two other bridges, in Biłgoraj and in Sosnowica. The structures of the bridges in Sosnowica and Lublin are inspired by the structure of the Biłgoraj bridge. Actually, its side view and tendon system resemble Jackson's arched beam. Chronologically, first the bridge in Biłgoraj was erected – its design was approved in May 1955. The design of the bridge in Sosnowica bears the date 24 April 1957. The design of the bridge in Pawia Street was prepared in 1961. In the design phase, two bridge variants were considered in each case, a reinforced beam bridge and a prestressed slab bridge. There are surviving fragments of the design documentation, legible, but in a bad condition. In most cases, replicas of drawings were made, however, originals were used in several places as a valuable original source.

The bridge design process spanned over a decade. During that time the bridge load standards changed. Anybody interested in the changes of bridge load models and their effects can find more information on the subject in paper [27].

3. THE BRIDGE IN BIŁGORAJ

The design was prepared according to the standard of 1952 [28]. The bearing capacity adequate to 1st class load allowed the passage of individual 20 T vehicles. Additionally, the bridge was designed to withstand a passage of heavy tractor of the weight equal to 80 T. The bridge parameters: length L=17.8 m, support span Lt=16,9, width B=10,06 m including the roadway 3x3.5 m, and two sidewalks of 1.25 m each. The bridge bevel of 90°.

The first variant of a passage over the Czarna Łada River was the reinforced concrete bridge designed of 15 October 1953. A four-beam bridge carrying deck with crossbeams, the superstructure height h=1.4 m, the span L_t=16,9 m, the clearance L_0=16,0 m.

The other variant, Fig. 2., selected for implementation, was a post-tensioned slab bridge, designed by J. Zabłocki and D. Żołnierczyk, reviewed by B. Kędzierski and T. Wojciechowski, and additionally verified by Cz. Eimer.

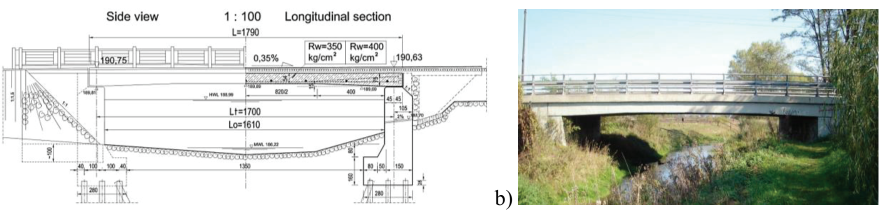
a)  b)

Fig. 2. Bridge in Biłgoraj a) side view and the longitudinal section b) a photo from the outlet perspective.

The abutments were placed on three rows of wooden piles, 11+10+11 pieces in each row. The height of the slab structure at the support is equal to h_0=67 cm, and at the span h=54 cm. In cross-sections the panel height is constant. 2% roadway cross slopes were obtained by means of the variable height of the concrete overlay on a waterproofing layer.

For the purposes of prestressing, the 12ϕ5 longitudinal tendons made of R_r=16500 kg/cm^2 steel (described as piano steel in the design project), each of the bearing capacity of 20 T, were used. 80 tendons were laid as horizontal straight lines, and 79 were vertically deflected parabolically at the supports. Altogether 157 cables of the total tension P=314T, transversely spaced every 6 cm. The plate was transversely prestressed by means of nine 12 ϕ 5 cables. The arm of the action of a transverse cable, measured from the surface of the top plate to the cable axis, equalled a=40 cm. Freyssinet’s conical anchoring system were used. The reinforcement against the stretching of concrete in the zones behind the anchoring blocks was a set of 3 nets made of rods of the diameter of ϕ 8 mm, in the spacing of 12 cm. In the general list the material characteristics were included: n_0=h/L_t=1/31, n_1=(piano steel)/(concrete volume) = 6.952/106=0.066 t/m3, n_2=(ordinary steel)/(concrete volume) =(1.592+3.092+2*0.75)/106=0.058 t/m^3. The ordinary steel mass contains: distribution rods, the rods of the parabolic tendon routing grids and cable casing sheets.

In 2005, the bridge was renovated with only vehicle traffic introduced afterwards. Its width which is 2x3.5 = 7 m. A separate footbridge for pedestrians and cyclists was constructed next to the bridge, on the inflow side. The bridge is located in the meadows surrounding the river. Aesthetically, a side view of its bright body with its quasi-arc transom gives the impression of a minor, non-aggressive dominant. The bridge functions faultlessly.

4. THE BRIDGE IN SOSNOWICA

The bridge design bears the date 24 August 1957, [29]. It was designed by Z. Jaworski and verified by W. Barzykowski. The verification study contains the information that statical calculations were conducted according to the 1956 standard, as well as using equations from Kaufman's book [19].

The existing bridge is placed on two rows of 5 m long Contractor piles, 22 items under each of the abutments in total. The superstructure length L=13.3 m, the plate width B=9.19 m. The slab thickness is constant and equals h=38 cm, where the plate is chevron-shaped in the cross-section, in accordance with the roadway cross slopes. The pedestrian slab is b=1.77 m, while its height changes from 30 cm at the outer edge to 31.5 cm at the edge on the bridge side. A left-hand bridge bevel, the angle value equal to 70°.

a)  b) 

Fig. 3. The bridge in Sosnowica a) side view b) view of the underside.

The first variant of the bridge was a concrete beam structure post-tensioned with tendons. In Fig. 3.b two bridge elements are visible: the road bridge superstructure slab and – 60 cm away from it - an independent pedestrian slab. Fortunately, a drawing of the slab in question is preserved among the technical documents. The slab is called "working footbridge" there, Fig. 4. The use of two separate slabs allowed for segregation of vehicle and pedestrian traffic. 12ϕ5 mm prestressing tendons were used - 8 of them at the bottom layer and 7 over them. The bearing capacity of a single cable was 20 T, Freyssinet anchoring blocks were used. ϕ12 transverse separation rods were put in-between the top and bottom slabs at intervals of 50 cm.

In 2006, the technical documentation for the purposes of the bridge renovation was drawn up. The conducted inventory did not reveal any structural defects.

a) b)

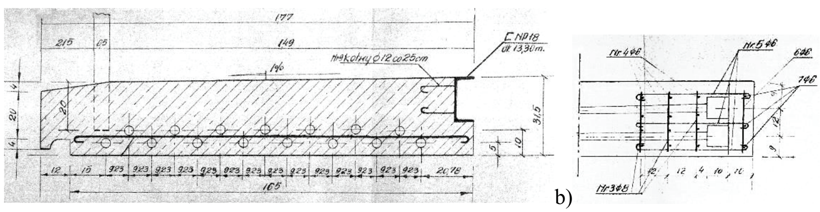


Fig. 4. Sosnowica – working footbridge a) cross-section of the slab with the arrangement of post-tensioned tendons b) detail of the slab reinforcement at the anchorage.

The bridge is aesthetically interesting, which means something entirely different than “this bridge is nice”.

5. WOODEN BRIDGES IN PAWIA STREET IN LUBLIN

Initially, the bridges over Czerniejówka river were wooden, then a PC slab was built. It is not always possible to find images of the past of the bridge. In the case of the bridge in question, the documented history goes 100 years back. Fig. 5.a shows a footbridge in the course of a gravel path for pedestrians stretching from Kunicki Street to the district of Kośminek. It is a single span footbridge of the length of approx. 8 m, with stairs to access the platform on both sides. A significant elevation of the superstructure results from the high ordinate of the designed water level (H.W.L. = 192.348 MASL), at the water column height of 2.19 m. Fig. 5.c shows a photograph taken from a wooden bridge in the direction of the river flow. It illustrates the level and area of the Czerniejówka River floodwater.

In 1940, the construction of a road bridge commenced, Fig. 5.b, in the place of the former footbridge. It was a classic three span wooden bridge of a continuous beam structure. The continuity maintained by saddle beams (without struts) placed above the yokes.

Fig. 5. The wooden bridges in Pawia Street before World War 2 a) the footbridge b) road embankment for the bridge, 1940 c) Czerniejówka waters rising, a view from the bridge, 1942; ([2]).

The beams consist of two wooden logs, located one over the other, joined by means of wooden blocks and steel anchors. The bridge wall wings in Fig. 5.b are to be covered with ground cones according to the design.

6. POST-TENSIONED CONCRETE SLAB BRIDGE IN PAWIA STREET

In 1949, the construction of ZOR[3] Zachód, ZOR Bronowice and ZOR Tatary residential districts commenced in Lublin. In Lublin, they were equivalents of Nowa Huta in Cracow.

Lublin grew which called for further investment in the city's communication network. The erection of the ZOR Bronowice residential district forced the extension of Fabryczna Street. At the beginning of 1960s, the construction of today's Droga Męczenników Majdanka[4] Street was started. It allowed for fast and easy access to the city centre. The street was planned to be constructed in a deep trench (max. 7 m) over the distance of approx. 750 m, taking into consideration the viaduct to be erected over it, on the Warsaw-Kowel railway line. The construction work was assumed to take four years. During that time, the access to the city centre via Pawia Street was significantly hampered. Due to that fact, Lotnicza Street and Pawia Street with a new bridge over the Czerniejówka River should have been constructed first. Here, we have an example of engineers' actions in the living tissue of a city, very similar to planning a road and bridge investments in contemporary Lublin.

The technical design stage was preceded by an expert study conducted by Z. Krogulec, assessing the advantages and disadvantages of the competing technologies, taking into consideration grade line ordinates, the quantity of materials, the foundation method and the possibility of fast construction.

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[2] Old black and white photos were sourced from https://www.biblioteka.teatrnn.pldlibradlibrasresultsaction =SearchSimilarAction&eid=147

[3] ZOR stands for Factory Workers' Residential District

[4] The Way of Martyrs Majdanek Street

The reinforced concrete superstructure would be twice as high as the prestressed bridge which, in turn, would necessitate elevation of the road gradeline and reconstruction of the sections of the approaches of the total length of approx. 150 m. Therefore, it was decided to construct a post-tensioned concrete slab bridge, designed by Z. Krogulec and M. Pyszniak, while the shallow foundation of the abutments was designed by W. Barzykowski, [30]. In the conceptual design, the bridge length amounted to 17 m, but it was shortened to 16 m in the technical design, Fig. 6.

a)  b) 

Fig. 6. Present view of the bridge in Pawia Street, April 2019 a) side view b) Pawia Street view.

In the descriptive documentation of the bridge, there are several literal references to the design of the bridge in Biłgoraj, while the only change is having to take into consideration the right-hand bevel of the bridge of the angle 68°. Unfortunately, the prestressing calculations are missing.

The structure characteristics: PC slab length – 16 m, slab width – 7 m, slab height in the middle of the bridge – 54 cm, at the support – 67 cm, in the side view the length of the slab underside slope at the supports is equal to 3.90 m, slab concrete class - C30/37, w/c = 0.35 ÷ 0.4, bridge width 7 + 2 x 1.25 = 9.5 m.

91 18ϕ5 mm longitudinal prestressing tendons made of 1st class high-grade steel of the strength 16500 kg/cm^2 = 1620 MPa were designed. Every other cable is bent upwards, cable spacing on the transverse axis of the bridge equals 8.25 cm, Fig. 7, 18xϕ5 mm transverse tendons at intervals from 1.5 m to 2.25 m, post-tensioned after 21 days of setting the concrete mix. The tendons were installed in sheet casings, the reinforcement of the anchoring zone behind the blocks: three rows of ϕ8 mm grid, distribution reinforcement on the top: 1x1 m mesh grid of ϕ8 rods.

Fig. 7.a-b shows the anchoring blocks of longitudinal prestressing tendons. Just over the bottom cables, the course of a transverse cable with its anchor is marked with a dotted line. The prestressed slab is not horizontal, but chevron-shaped, of the same drop as the two-way crossfall of the surface, i.e. 1.5%. Waterproofing on the top surface of the slab is protected by the pressuring layer of concrete of 5 cm thickness.

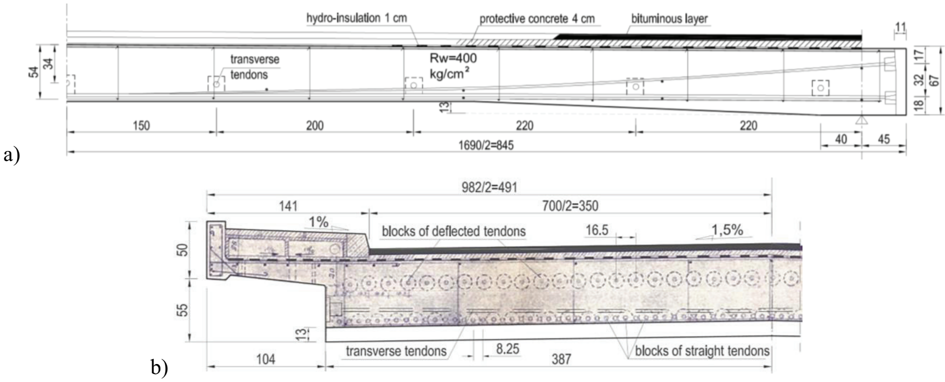


Fig. 7. Prestressing tendons a) longitudinal cross-section, tendon tracks b) slab end view, tendon anchors.

Sidewalk cantilevers were concreted after prestressing; transverse dilatation gaps in the pavements. Double crashed basalt aggregate was used. The dilatation gaps of the visible pavement cantilever are filled with chipboards. According to the technology description quoted above, the pavements should have been constructed after the slab prestressing to prevent them from affecting the functioning of the plate girder of significant bevel.

7. EXPERT STRUCTURAL STUDIES

The first expert study consisted in a dynamic examination of the bridge conducted by the employees of Structural Mechanics Department of the Lublin University of Technology under the direction of prof. Andrzej Flaga in 1997. The aim of the numerical study was the comparison of the initially designed slab and the actual state where some cables were corroded. The assumed hypothesis, confirmed by the analysis results, proposed that as a result of transverse prestressing, asymmetric loads and a significant slab bevel, the strain in the top area of the concrete slab was greater than the limit strains for concrete cracking. As a consequence, water infiltrated through the cracking, which, in turn, caused the corrosion of prestressing tendon rods, Fig. 8.a. This rational hypothesis was not confirmed in situ e.g. by uncovering the upper slab surface.

a) b) c)

Fig. 8. Bridge corrosion a) prestressing tendon rods view b) stalactites of calcium salt infiltrations c) ASR effect on concrete.

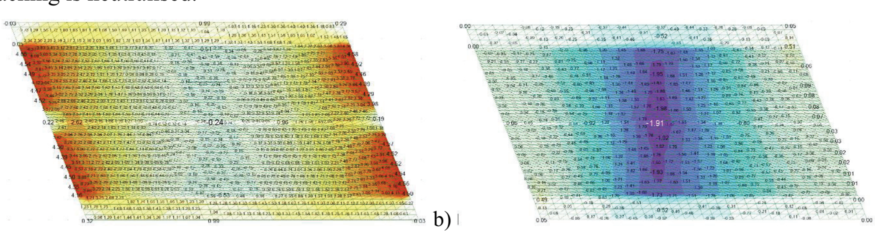
In 2014, an expert study was conducted by the team of DrogMost Lubelski. The author of the paper was among the members. In that case, the fundamental purpose was to identify the condition of the structure materials, especially the degree of the prestressed slab degradation. Chemical tests of concrete for the content of sulphate and chlorine ions, as the most harmful in corrosion processes, were performed. The pH number was determined. The results of all the tests were negative.

Non-destructive concrete surface hardness tests indicated the concrete degradation degree from the design strength class of C30/40[5] to the present one of C25/30.

Numerically, the crack was modelled by means of introducing a lower value of the concrete elastic modulus (respective to C12/15 class), in the middle part of the longitudinal symmetry axis of the length of 8.20 m. 11 longitudinal prestressing tendons corresponding to the 1 m breadth of the underside strip of the slab were disregarded in the numerical model, as well as 2 middle transverse tendons. In Fig. 9, two maps of normal longitudinal stresses on the bottom slab surface are shown. Adding two tensile stress components, one obtains the value of longitudinal stresses

$$\sigma = \sigma_{d.l.+P} + \sigma_{E150} = -0.24 - 1.92 \cdot 1.27 = -2.68\, MPa\,, \qquad |\sigma| = 2.68\ MPa > f_{ctk,\,0.05} = 1.8\, MPa\,,$$

which means that a compression state due to prestressing is absent in both stress components, and the cracking is neutralised.



a) b)

Fig. 9. Longitudinal stresses on the slab bottom surface in the middle axis a) action of dead loads and prestressing b) service load, 2 heavy vehicles of 2 x 150 = 300 kN weight.

[5] C30/40 - formally is not written down in standards.

However, decreasing the number of active tendons, the load capacity of the slab is reduced. The determined load capacity does not meet the criteria of the minimal load class E, [31]. The basic conclusion indicated the urgency of commencing new bridge design works, as an element of the construction process preparation. The conclusion results from the choice between strengthening the existing structure and building a new one. In this particular case, a very poor condition of the structure was decisive, Fig. 8. Further observations of the functioning of the structure were recommended. It was suggested to fit plaster seals, as well as to conduct periodic geodesic measurements of the plate deflection value. It was also proposed to reduce the permissible vehicle loads to the maximum weight of 10 T and to limit the vehicle speed to up to 30 km/h, see Fig. 6.b. The necessity of strengthening, renovating and systematic observation of the structure was emphasised. In contrast to the first expert study, the cause of the bridge degradation was identified, namely, the incorrectly conducted technical inspections to date had not revealed developing concrete corrosion. Its symptoms were not observed in time. An adequate counteraction, i.e. the waterproofing replacement, might have prevented the bridge destruction.

Fig. 8.c shows a corrosion image of the Alkali Silica Reaction (ASR) type. The ASR process is qualitatively well understood in terms of chemistry, however, its quantitative identification, which would allow for its efficient prevention and minimisation, is still lacking, [31].

During the summer time of 2019 the prestressed bridge was demolished and a new one was erected. In the Fig. 10.a the demolishing of the old bridge is shown. The lack of injection of sheet casings was the reason of cables corrosion.

Fig. 10.b shows the new bridge in Pawia Street designed by Krzysztof Gnyp. It is a massive reinforced concrete frame structure, placed on piles of the diameters of 1.20 m. Vehicles of the weight not exceeding 500 kN, which corresponds to Load Class A according to [32], are allowed on the bridge.

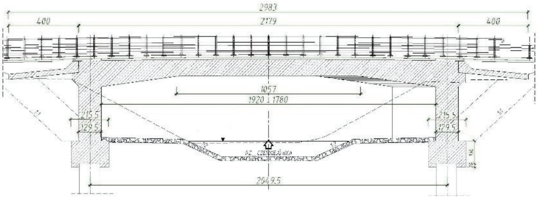
a)  b) 

Fig. 10. Year 2019 a) prestressed bridge demolishing b) a design of a new bridge; longitudinal section.

8. CONCLUSIONS

In 1950s, three modern road bridges of post-tensioned concrete slab superstructures were designed and erected in Lublin Voivodeship. After having been renovated, two of them will continue to serve for many years, perhaps reaching the life span of 100 years, prescribed by the relevant regulations.

Looking for the causes of the limit degradation of the bridge in Pawia street, one can list several reasons. The system of the periodic control of the bridge failed – the potential threat had not been identified in time. In terms of design, the theory of skew angle plates was not applied, especially the knowledge of the actual strains of obtuse corners. In technological terms, the error consisted in not having filled the cable hoses with sealing injection. During the demolition of the bridge in summer 2019, a dry cracking appeared on the top surface of the bridge plate. In the author's view, the cause of leaks and, as a result, the accumulation of water in the cable hoses, could have been the inadequate securing of the anchor blocks.

The determined level of the dependable water flow corresponds to the level of the wooden footbridge from the interwar period.

All three prestressed concrete slab bridges are "reinforced" by means of prestressing steel. The quantity of supplementary reinforcement is smaller than 0.2 %. To put it simply, the slabs are concrete blocks prestressed with tendons in the tension zone, are very similar to Jackson's concept. In comparison to contemporary requirements with regard to ordinary reinforcement design, it is an original, pure concept of PC.

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PIERWSZE PŁYTOWE MOSTY KABLOBETONOWE W POLSCE, MOST NA ULICY PAWIEJ W LUBLINIE

*Keywords***:** utrzymanie mostów, pierwsze polskie betonowe mosty kablobetonowe, mosty na ulicy Pawiej w Lublinie

STRESZCZENIE:

Przedmiotem artykułu są pierwsze polskie mosty kablobetonowe, przy czym w szczególności są opisane mosty na Lubelszczyźnie. Pretekstem do podjęcia tej tematyki jest historia mostów przez rzekę Czerniejówkę na ulicy Pawiej w Lublinie.

We wstępie zostały przypomniane podstawowe publikacje z okresu rozwoju koncepcji sprężania. Przegląd bibliograficzny posłużył do ukazania powodzeń i niepowodzeń sprężania belek i płyt betonowych. Początkowo stosowano kable sprężające wykonane ze zwykłej stali. Pierwszym udanym sprężeniem było naśladownictwo łuku ze ściągiem przeprowadzone przez Jacksona, który połączył bloki betonowe przy użyciu pręta metalowego. Dziś brzmi to dziwnie, ale największym osiągnięciem w rozwoju teorii betonu była konstatacja E. Freyssineta, że kable sprężające muszą być wykonane ze stali o wysokich wytrzymałościach ($f_u \geq 1500$ MPa), przy jednocześnie niskiej relaksacji tj. nie większej niż 2.5%. W przypadku betonu konieczny był rozwój teorii pełzania betonu do stopnia, który zapewniał występowanie efektywnej siły sprężającej po uwzględnieniu odkształceń sprężystych i lepkosprężystych ściskanego betonu. Początkowa teorię Dischingera rozwinął Trost i Zerna. Wzór Trosta ma znaczenie podstawowe podstawowy i jest na tyle sprawny, że obecnie został zapisany w Eurokodzie. Spośród monografii dotyczących teorii sprężania należy wyróżnić książki Guyona i Jakubowskiego, które miały w Polsce duży wpływ na środowiska inżynierskie. Pierwszą monografią w języku polskim była praca Kaufmana pt. Mosty sprężone.

W roku 1952 zostały wybudowane dwa mosty kablobetonowe płytowe. Tomasz Kluz był projektantem mostu w Końskich o długości 12.6 m, a Czesław Eimer zaprojektował most o długości 18.9 m. Mosty te uznawane są za pierwsze polskie mosty sprężone.

W latach pięćdziesiątych na Lubelszczyźnie zbudowano 3 mosty kablobetonowe płytowe. Chronologicznie pierwszym z nich był most w Biłgoraju (Puszcza Solska) zaprojektowany w roku 1952. Jego długość to 17,8 m, szerokość 10.6 m. W planie most był prostokątny. Drugim obiektem był most w Sosnowicy zaprojektowany w 1957 r. Most w Sosnowicy jest w rzeczywistości dwuczęściowy. Składa się z połączonych niezależnych elementów: płyty przenoszącej ruch samochodowy oraz płyty będącej kładką dla pieszych. Długość obiektu wynosi 13.3 m, ukos mostu 70°. Na szczególną uwagę zasługuje kładka dla pieszych . Jej wysokość konstrukcyjna wynosi 30 do 31.5 cm. Kładka została sprężona piętnastoma kablami o naciągu 200 kN każdy.

Oba wymienione mosty zostały w ostatnim dziesięcioleciu wyremontowane i funkcjonują bezawaryjnie.

Trzecim obiektem rozpatrywanym w artykule jest most w Lublinie na ulicy Pawiej. Rzeka Czerniejówka jest jedną z trzech rzek przepływających przez Lublin. Położenie ulicy Pawiej w dawnej dzielnicy przemysłowej sprawiało, że podczas kwerendy zbiorów archiwalnych udało się odnaleźć kilka zdjęć z okresu międzywojennego oraz z czasów II wojny światowej. Mosty były lokowane w jednym miejscu. Pierwszym była drewniana kładka dla pieszych wysoko wyniesiona ponad tereny zalewowe rzeki. Dostęp do kładki z obu stron był za pomocą schodów, które podczas powodzi były zalewane. Most drogowy został wzniesiony w 1940 roku i funkcjonował do budowy mostu sprężonego w roku 1960.

Projektowanie każdego z opisywanych mostów polegało na wykonaniu dwóch koncepcji. Pierwsza z koncepcji była w zakresie mostów żelbetowych, podczas gdy druga była na ówczesne lata nowoczesna i dotyczyła technologii sprężania. Most żelbetowy na ulicy Pawiej miał wysokość konstrukcyjną pomostu belkowego dwa razy większą niż most płytowy sprężony. To przesądziło o jego wyborze. Alternatywnie przy koncepcji mostu żelbetowego konieczne by były przebudowy dojazdów w celu uzyskanie wyższych rzędnych niwelety. W wyniku analiz zdecydowano o posadowieniu bezpośrednim przyczółków. Odniesieniem do projektowania płyty sprężonej był most w Biłgoraju, przy czym w tym przypadku ukos mostu był znaczny i wynosił 68^{o}. Ostatecznie zdecydowano, że długość mostu będzie równa 16 m. Wysokość płyty pomostu była zmienna, przy przyczółkach było 67 cm, podczas gdy w osi 54 cm. W projekcie mostu uwzględniono występowanie wsporników chodnikowy, przy czym ich budowę należało prowadzić po wprowadzeniu sprężenia. Most został sprężony za pomocą 91 kabli podłużnych 18ϕ5 mm. Zastosowano stal pierwszej klasy o wytrzymałości 1650 MPa. Budowa przebiegała bez zakłóceń.

Zupełnie nieoczekiwanie w moście w jego osi podłużnej od spodu płyty pojawiły się oznaki korozji betonu , a w miejscach ubytków otuliny były widoczne efekty korozji osłon kabli i drutów sprężających. W roku 1997 zespół pod kierownictwem prof. Andrzeja Flagi przeprowadził ekspertyzę mostu. Przyjęto hipotezę o istnieniu zarysowania w osi mostu od góry płyty, powstałego na skutek przekroczenia odkształceń granicznych w złożonym stanie rozciągania. Przeprowadzone analizy statyczne i dynamiczne MES potwierdziły hipotezę. Ze studiów dokumentacji eksperckiej wynika, że nie potwierdzono istnienia rysy przez lokalne odsłonięcie płyty.

W roku 2014 kolejną ekspertyzę prowadził zespół DrogMostu Lubelskiego. Przeprowadzone badania betonu wskazywały na jego degradacje przynajmniej o jedną klasę wytrzymałości betonu. Analizy numeryczne przy uwzględnieniu wyłączenia skorodowanych kabli wykazały na górnej powierzchni płyty występowanie naprężeń ściskających, co oznaczało neutralizację przyczyny zarysowania.

Niestety na, tym etapie funkcjonowania mostu zalecono przygotowanie dokumentacji technicznej nowego obiektu i wymianę istniejącego mostu na nowy. W okresie przejściowym wnioskowano o zmniejszenie dopuszczalnych ciężarów pojazdów do 10 T i ograniczenie prędkości do 30 km/h. Zarząd Dróg i Mostów w Lublinie zrealizował przedłożone wnioski.

Szukając przyczyn granicznej degradacji ustroju nośnego można wymienić kilka elementów, które sumarycznie przyczyniły się do utraty nośności. Zawiódł system okresowych kontroli stanu technicznego mostu, Nie zidentyfikowano na czas potencjalnego zagrożenia. W zakresie projektowym, na podstawie dostępnych dokumentów technicznych dostrzega się brak wykorzystania teorii płyt z ukosem, w szczególności wiedzy o stanie wytężenia tzw. naroży rozwartych. Ten most jak i most w Sosnowicy projektowano na zasadzie analogii do istniejących mostów o podobnej geometrii.

W zakresie technologicznym błędem było pozostawienie osłon kabli niewypełnionych uszczelniającym iniektem.

Podczas rozbiórki mostu przeprowadzonej latem 2019 r. stwierdzono, że warstwa hydroizolacji w osi mostu była sprawna, bez śladów zarysowań. Natomiast podczas kruszenia betonu pojawiło się suche zarysowanie na górnej powierzchni płyty mostowej. Zdaniem autora przyczyną przecieków i w konsekwencji gromadzeniu wody w osłonach kabli mogło być niewłaściwe zabezpieczenie bloków kotwiących.

Wyznaczony poziom wody miarodajnej odpowiada poziomowi drewnianej kładki z lat międzywojennych.

Opisywane mosty płytowe sprężone charakteryzuje stopień zbrojenia zbrojeniem zwykłym o wartości poniżej minimalnego zbrojenia. To oznacza, że płyty ustrojów nośnych są betonowe zbrojone i jednocześnie sprężone kablami.

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