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DOI: 10.1515/ace-2015-0104

ARCHIVES OF CIVIL ENGINEERING
Vol. LXII ISSUE 4, Part 1 2016

ŁAZIENKOWSKI BRIDGE FIRE IN WARSAW – STRUCTURAL DAMAGE AND RESTORATION METHOD

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On the 14th of February, 2015, a huge fire broke out on Łazienkowski Bridge; a five span bridge, 423 m long and 28 m wide, built in the years 1972-74. It was a fully steel structure with four plate girders and orthotropic deck. The fire started under the first span during the replacement of wooden service decks. The next day, the Department of Bridges of the Warsaw University of Technology was designated to conduct an expertise material investigation, geometrical verification, and FEM model analysis. The subject of this paper concentrates on geometrical issues. The main difficulty of this task was the lack of full reference data regarding the bridge's original structure. The old design was incomplete and there was no actual surveying results for the undamaged structure. As a conclusion, some remarks focused on surveying measurements and on the final decision regarding this bridge are given. It was eventually exchanged into a brand new one and put into public use on the 28th of October, 2015.

Keywords: fire, bridge, orthotropic deck, surveying methods, formation line, girder verticality

1. THE BRIDGE STRUCTURE

Łazienkowski Bridge is one of the most important bridges in the center of Warsaw, the capital city of Poland with almost 2 million inhabitants. More than 100,000 vehicles use the bridge daily on

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average, among them many public transport buses (the highest number in comparison to other Warsaw bridges).

The continuous beam superstructure of the destroyed bridge consisted of five spans 76,5+90,0+90,0+90,0+76,5=423,0 m (Fig. 1). The structure was steel, mostly welded (assembly joints were riveted), with four plate girders and an orthotropic deck. The spacing between the girders was 6,114 m + 9,560 m + 6,114 m. The total width of the bridge was 27,76 m. It covered two three-traffic lane roadways, 10,50 m each, separated by a 1,00 m wide security strip. Additionally, two cantilevers for pedestrian use were installed – each 2,88 m wide (Fig. 2). Three service decks situated between girders were covered with wooden boards.

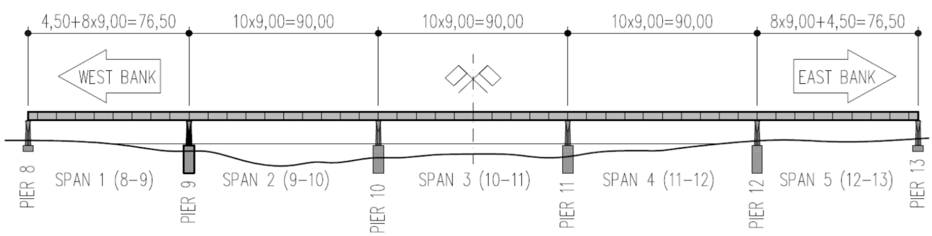


Fig. 1. Side view of Łazienkowski Bridge

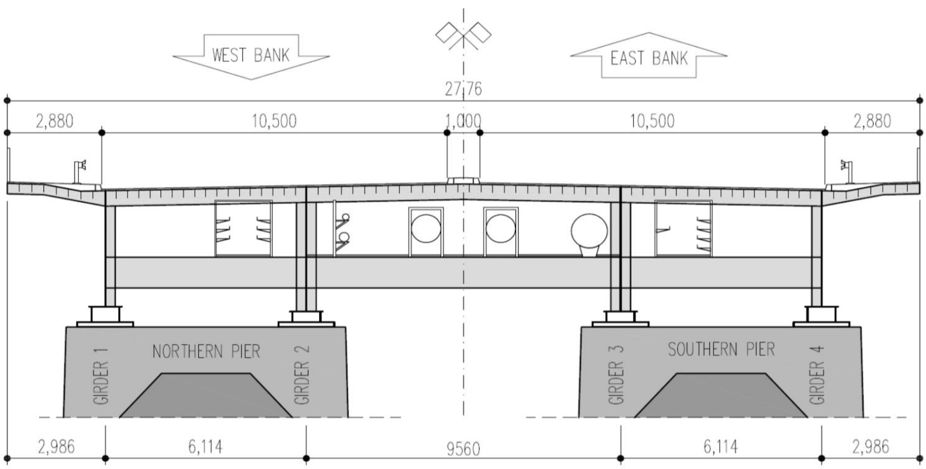


Fig. 2. Cross section of Łazienkowski Bridge

The depth of the girders was constant along whole structure; 3800 mm for external and 3892 mm for internal girders. The thickness of the webs was 12, 14, 16, or 20 mm. Lower flanges were 700 mm wide and 24 or 30 mm thick. Some parts of those flanges had cover plates with a thickness of 20, 24, and 30 mm. The orthotropic deck consisted of a plate, cross-beams, and longitudinal ribs. The deck plate was 12 mm thick in span, 18 mm on the supports and 10 mm in the cantilevers. Cross-beams had webs with dimensions of 400 mm x 8 mm and lower flanges with 200 mm x 24 mm. The spacing of the cross-beams was 1500 mm. Longitudinal ribs were of the open type (180 mm x 12 mm) with distance between them 344 mm. There were also lower cross-beams in the structure. Their spacing along the longitudinal axis of the bridge was 9,00 m. Every second beam had double webs. Their size was 1222 mm x 12 mm, and the lower flange 300 mm x 20 mm and 200 mm x 20 mm. The spacing of the double webs was 600 mm. The orthotropic deck was build using St3M (S235) steel, girders, and lower cross-beams of 18G2A (S355) steel .

The insulation was bituminous and the pavement was asphalt with a depth 5,0 + 4,0 cm. The superstructure was supported by two concrete piers standing on one common foundation (caissons and piles). There were no abutments in this bridge; two approach viaducts (one from each side) connected the bridge with the streets of Warsaw.

Inside the bridge superstructure, between internal girders, were installed: a cold water, two hot water, and gas pipelines, and about 20 different telecommunication cables in plastic covers were located between the two pairs of girders.

2. Course of fire

The fire started under bank span 12-13, during a replacement of wooden elements of the service decks. They were stored on the ground exactly under the structure. The fire started in the evening and was extinguished by morning the next day. During that time, consequently, three spans (10-11, 11-12 and 12-13) were on fire. The actions of the fire brigade were provided from the bank, from the water below, and along the bridge structure starting from pier 8. It is necessary to add that it was winter, with temperatures of -5°C on this Valentine's Day.

The presence of pipelines and cables also had an influence on the expansion of the fire. At the beginning, various kinds of insulation accelerated the fire, and then kept elevated temperature up for several hours.

3. Effects of fire

3.1. General

Very extensive investigation and analysis was done over the following weeks including material investigation, geometrical verification, and FEM model analysis to determine the behaviour of the bridge during thermal overloads. In this paper are presented results of: local deformation of the deck due to results of surveying measurements and FEM analysis, local deformation of the web of the selected girder, global rotation of the outer girders, and global displacement of all four girders measured during the investigation and during further disassembly of the burnt bridge.

The area of damage covered the two fastest lanes of traffic, half of the middle lanes, and part of the bus lanes on the northern side of the bridge along three spans (10-11, 11-12, 12-13).The fire caused a lot of damage to the steel superstructure, piers, and equipment.

3.2. Orthotropic deck

First, there was enormous noticeable damage to the deck; extensive, deep bulges of the plate of the orthotropic deck in the central part of the traffic lanes (Fig. 3 a). In the span 12-13, values of local differences were up to 30 cm, mostly around 15-20 cm. From the bottom view of the deck there was a visible loss of stability of the longitudinal ribs. This kind of damage was observed in the area of the deep bulges of the orthotropic deck as well as in the area without those bulges. The plate on the bus lanes was locally flat and no damage of insulation and asphalt pavement was observed. Only upon further surveying the measurements showed that this part of the structure was also deformed as noted by the shape of the strong deflected girders (Fig. 3 b).

FEM analysis (with simplified boundary conditions – only local actions) was done to determine the internal forces which could cause observed displacements. The results showed different forms of local failure (Fig. 5 a, b c), which could be found on the real structure (Fig. 5 d).

Deformations of the plate and of the longitudinal ribs of the orthotropic plate „pulled" the cross-beams alternately; first one up, and the next one down.

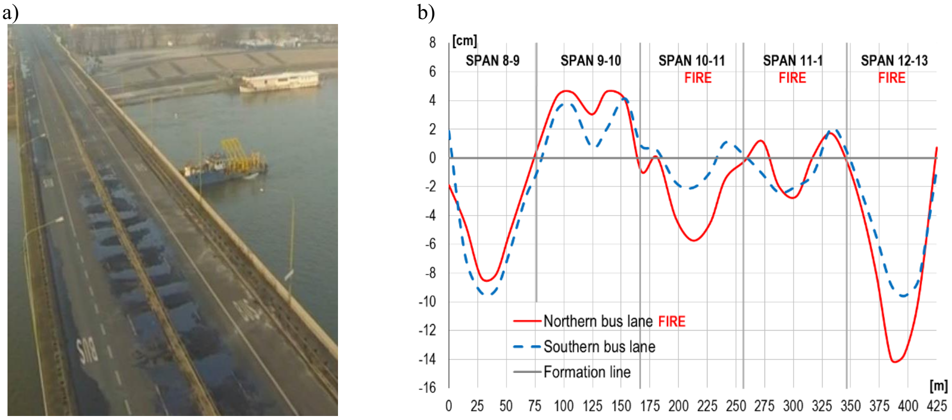


Fig. 3. First view of the damaged structure: a) top view of destroyed central part of traffic lanes and deceptively undestroyed bus lanes [www.gazeta.pl], b) comparison of the bus lane formation lines

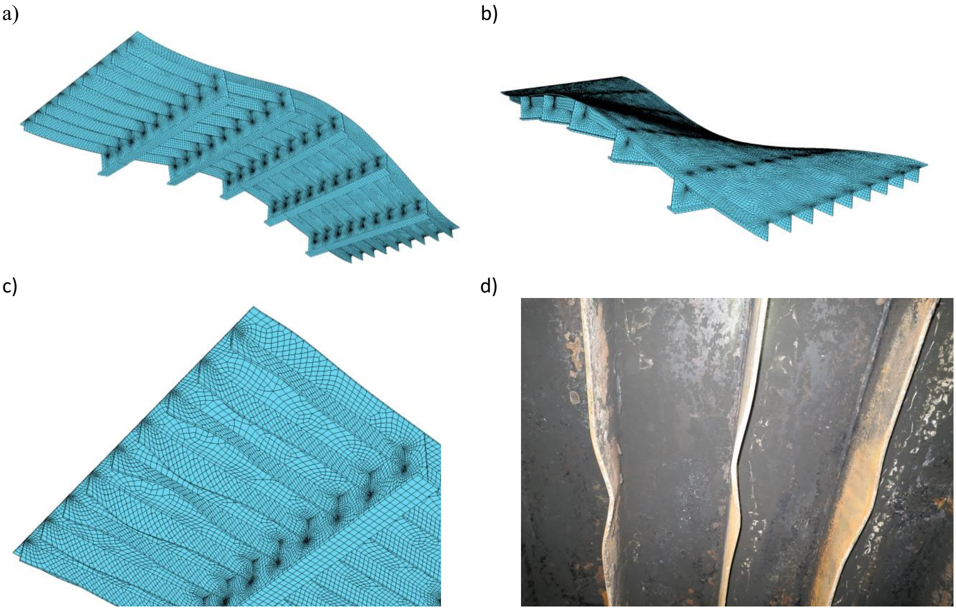


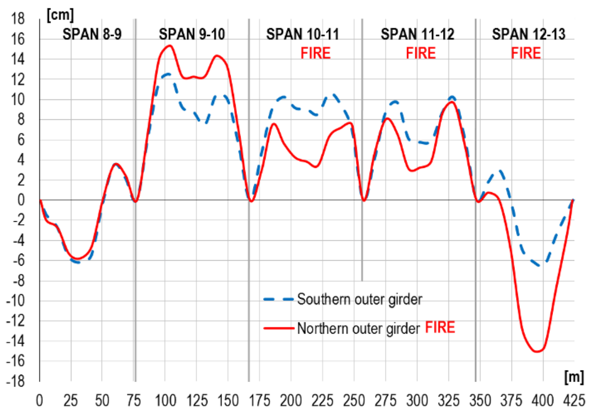
Fig. 4. Loss of stability of the orthotropic deck: a) FEM analysis in temperature 100°C-115°C – bending form of loss of stability, b) FEM analysis in temperature 115°C-125°C – bending-torsion form of loss of stability, c) FEM analysis in temperature 150°C-175°C – warping of longitudinal ribs, d) real warping of longitudinal ribs of bus lane part

3.3. Girders

Many surveying measurements were done, unfortunately, surveying the bridge before the fire was not possible. Therefore, an analysis of geometric changes to the structure was done, comparing the damaged areas to spans where the fire did not reach, i.e. 8-9 and 9-10.

Excessive deflection of the two northern girders was observed. The northern external girder was lower comparing to the southern one by 58 mm, 32 mm, and 81 mm, for spans 10-11, 11-12, and 12-13, respectively (Fig. 5 a). The northern internal girder was lower comparing to the southern one by 46 mm, 22 mm, and 33 mm for spans 10-11, 11-12, and 12-13, respectively (Fig. 5 b).

a)



b)

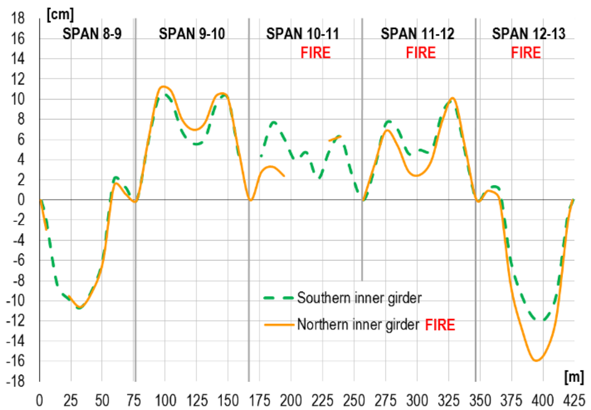


Fig. 5. Comparison of girders deflection: a) external , b) internal (some data was impossible to collect because of impeded visibility of bottom flange)

As a result of the fire there was a torsional deformation of the whole cross section of the bridge in spans 10-11, 11-12, and 12-13, a result of the shrinkage of the total steel structure in the transverse direction, and deflections or bulges of particular elements of the orthotropic deck. The initial shape of the cross-section was symmetrical, and both pairs of girders were slightly inclined to the longitudinal axis of the bridge (which showed the shape of two undamaged spans) after years of work. Surveying after the fire showed that the southern girders rotated in the direction toward the longitudinal axis of the bridge, and the northern girders rotated in the direction opposite to the longitudinal axis of the bridge. Finally, the northern girders in all three spans affected by the fire remained straight, with the southern more inclined (Fig. 6). The total cross-section of the bridge was asymmetrical – "skewed" in the northern direction – and as a consequence the loss of stability of the whole structure was very possible.

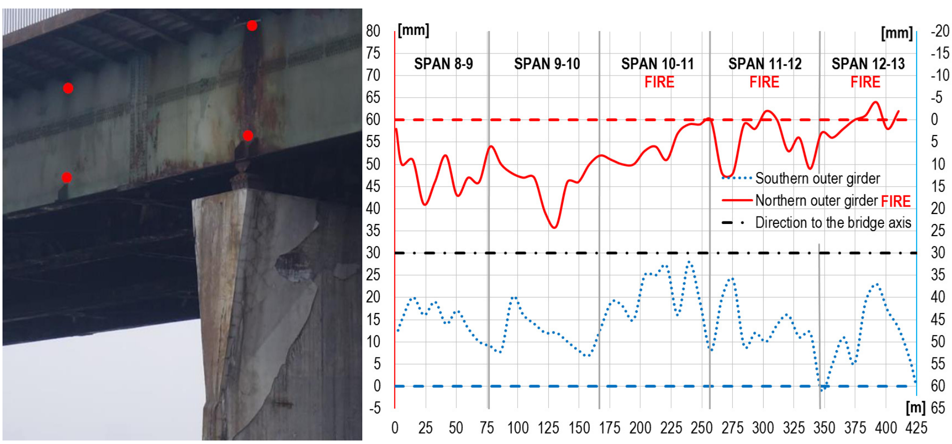


Fig. 6. Verticality checking of outer girders: a) example of the measurements points, b) results for the whole structure

More serious damage to the girders was discovered when it was possible to go under the deck in the close proximity of the girder web. Extensive local bulges of the webs in the internal girders were noticed and later measured. For the northern one (which was most exposed to fire), the curvature was bigger than 40 mm at a distance of 1500 mm (Fig. 7 b).The maximal measured bulges of the northern internal girder were 43,1 mm at span 10-11 and 41,2 mm at span 11-12.

Surveying was done within the fields between the vertical ribs of the girders. According to observations, a mutual location of reference points for the measurements of deformations of longitudinal ribs was also not linear. The web of the girder lost stability for more than 1500 mm of distance. A comparison of values of standard permissible deformations against measured ones showed that the steel structure was in a state of emergency. Large deformations („waving") of horizontal ribs stiffening in the upper part of the webs of the internal girders (Fig. 7 a) were also observed.

Horizontal and vertical ribs did not prevent a change of the geometry of the web. High temperatures caused „waving", thereby losing stability. The damage was much more extensive in the northern girder. An adjacent cold water pipeline secured the southern girder, consuming quite an amount of heat generated by the fire.

In situ and laboratory (each measured sample) hardness tests were conducted, similar to those verifying changes in the steel microstructure as a result of very high temperatures.

It was found that in many places the steel structure was of lower hardness than the nominal. Also, changes in the microstructure were noticed relating to the presence of elevated temperatures. The scatter of results is relatively high which confirms the expectation that the influence of elevated temperatures was not uniform. Therefore, it was practically impossible to point out precise locations of such places.

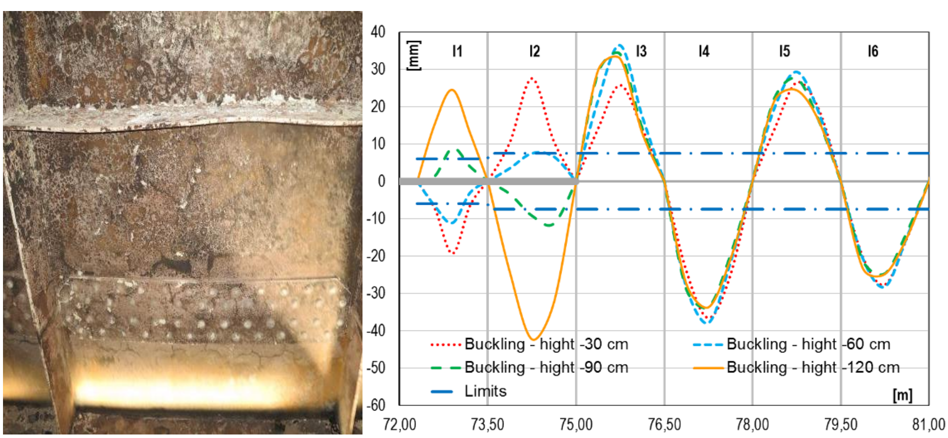


Fig. 7. Failure mode of the girders webs: a) picture of local bulges of the web and deformations of the horizontal ribs, b) local bulges of the 9 m long part of the girder 10-11 – values for level of measurements 30/60/90/120 cm below the deck

3.4. Bearings

Serious damage was observed with bearings as well; there had been excessive rotation of bearings on three piers (11, 12, and 13).

The bearings on pier 13 reached the maximal permissible angle of rotation (Fig. 8) caused by elevated temperatures, which changed the geometry of the steel superstructure. In the case of extreme low temperatures, there was a risk that span 12-13 could fall down. It was winter and the probability of occurrence of temperatures of -25ºC is high.

Fig. 8. Maximum permissible rotation of the bearing on the pier 13

3.5. Other damages

There was a lot of other damage observed, mostly damage of equipment. But there was also very local structural damage such as cracks of welds or shearing of rivets. For example, the investigation found several cracks of welds connected to the vertical ribs with cross-beams of the orthotropic deck (Fig. 9) and with the horizontal rib stiffening web of the internal girders. Many of the rivets were sheared in assembly joints between the cross-beams and the internal girders and in the joints of secondary elements in all three spans. Inventorying all this damage was impossible in reasonable time. Fortunately it would be necessary only in the case of maintenance, which was finally rejected.

The list of other damage includes also:

- damage of elongation joints on the last pier on the right bank (Pier 13),
- loss of stability of structural elements of service decks,

- bulges and cracks of asphalt pavement on traffic lanes and pedestrian cantilever,
- melting of bituminous insulation,
- demolition of barriers as a result of damage of orthotropic deck,
- total damage of anti-corrosion cover,
- damage of insulation of pipelines,
- vertical and horizontal bending of gas pipeline,
- total damage of cables,
- falling of concrete cover of one pier (shotcrete was applied a few years ago to cover surfaces of the piers. Elevated temperatures caused it to fall off, especially in pier 12).

Fig. 9. Typical crack of weld

3.6. General

Generally, the whole steel superstructure which was touched by fire had been damaged. It is necessary to point out that area of damage from an internal point of view was much larger than what could be seen from the outside.A very important phenomenon which should be considered during the determination of the effects of the fire is the shrinkage of steel. The main issue is product grade and the method of fixation of particular elements of the structure or the degree of freedom of strains.
During the cooling period, the value of shrinkage will be the same in all directions. The final result shows that residua deformations remain in the structures, therefore dimensions of elements measured

at distances with limited freedom of deformations will be smaller than the original one, and those measured at distances with not limited freedom of displacement will be larger.
An additional factor which influences the value of deformation of changes of the steel microstructure is the duration of time of the presence of high temperatures which generate damage. If exposure is long enough, then both mentioned above factors will occur, but it could happen that the duration of exposure to high temperatures was so short that the pavement was not damaged (the plate of deck did not “bulge”), but the longitudinal ribs of the orthotropic deck had lost stability.This means that a combination of two factors determined the level of damage of the steel superstructure: the value of deformations and the changes of the steel microstructure. In the case of Łazienkowski Bridge, the more important factor had been the value of residual deformations of the girder webs, deck plate, and longitudinal ribs, aiding in the decision to close and disassemble the bridge.

4. FINAL DECISION ABOUT THE BRIDGE

From the structural and material point of view, the burnt bridge was damaged at a high level. But, as always in such cases, the decision should be made taking into consideration not only structural and material aspects of the problem, but also economic factors, durability of the structure, and influence on public life in the city over short and long periods of time.As with any solution, in such a location (center of Warsaw with heavy traffic – 110,000 vehicles per business day) the social cost of closing the bridge down and the known time of construction of a new structure, it was decided that the city would build a completely new steel superstructure and repair the piers.

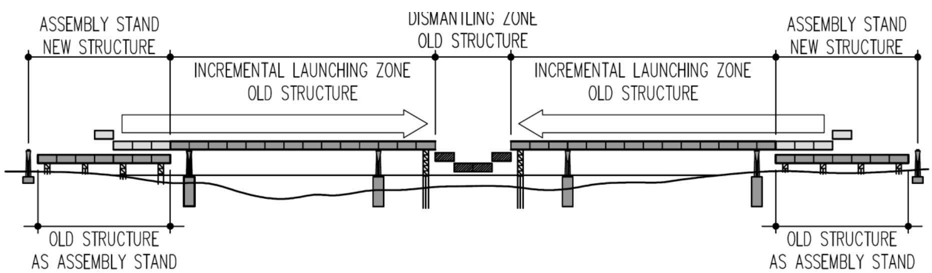


Fig. 10. Replacement method of steel part of Łazienkowski Bridge. During the operation additional temporary support in the middle of right part of the structure was added

The contractor decided to use the modified incremental launching method to construct the new superstructure (Fig. 10). Because of the use of some parts of the old structure in the process of launching, it was possible to avoid temporary supports (except for two) and disassemble the old structure while assembling the new one. This modification required cutting and lowering the outer spans (they consisted of assembly stands) and cutting the rest of the structure in the middle of the middle span. The bridge was divided into two parts. In the beginning, both were one-span, simply supported beams with cantilevers. In such a moment it was easy to compare the deflections of each part. Results of this comparison confirmed the poor condition of the burnt part: it had 10 cm of deflection, comparing to the non-burnt, 22 cm (Fig. 11). Finally, additional temporary support in the middle of span 4 was used.

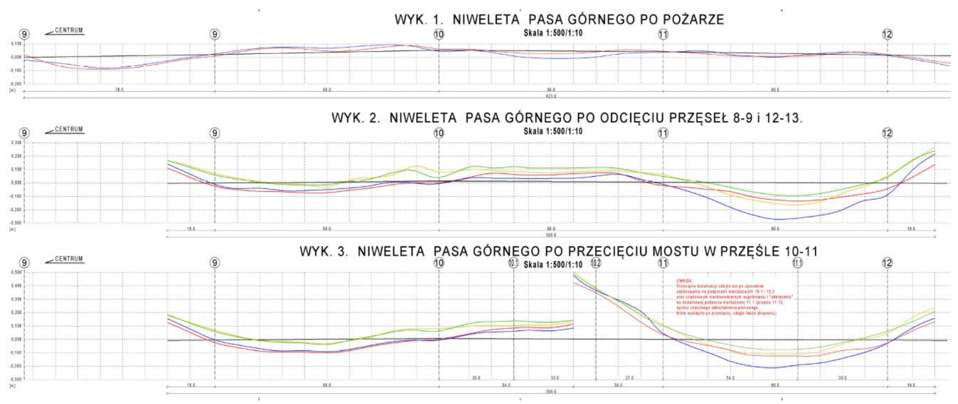


Fig. 11. Original surveying report from contractor of replacement of Łazienkowski Bridge. The third graph shows the huge difference between deflection of burnt and non-burnt spans of the bridge

Other final surveying methods helped to assess the condition of the old Łazienkowski Bridge. It was possible even despite of the incompleteness of data about the old structure.

The owner of the structure had decided to not order such measurements during the years of bridge exploitation, as there is no such requirement in any regulations. Of course, it can be part of the Quality Controls Reports, but usually, because of generating additional costs, it is not included.

In comparison to other methods it is cheap way of assessment and gives clear results.

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Received 03.10.2016
Revised 10.12.2016

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POŻAR NA MOŚCIE ŁAZIENKOWSKIM - USZKODZENIA KONSTRUKCJI I ICH KONSEKWENCJE

Słowa kluczowe: pożar, most, pomost ortotropowy, metody geodezyjne, niweleta, pionowość dźwigarów

STRESZCZENIE:

14 lutego 2015 roku miał miejsce pożar na Moście Łazienkowskim – pięcioprzęsłowym obiekcie mostowym o schemacie statycznym belki ciągłej i rozpiętościach 76,5+90,0+90,0+90,0+76,5=423,0 m oraz szerokości 27,76 m, wybudowanym w latach 1972-1974, mającym bardzo duże znaczenie dla struktury komunikacyjnej Warszawy – miasta o ponad 2 mln mieszkańców (codziennie przez most przejeżdża ponad 100 tys. pojazdów). Pierwotny obiekt to konstrukcja stalowa z czterema blachownicowymi dźwigarami o wysokości 3800 mm (dźwigary zewnętrzne) i 3892 mm (wewnętrzne) w rozstawie 6114 mm+9560 mm+6114 mm i pomostem ortotropowym o żebrach wiotkich o wysokości 180 mm i rozstawie 344 mm. Elementy konstrukcji były w większości spawane lub nitowane (m.in. połączenia montażowe). Konstrukcja była wykonana ze stali St3M (pomost ortotropowy wraz z górnymi poprzecznicami) oraz 18G2A (dźwigary i dolne poprzecznice). W konstrukcji dodatkowo były zlokalizowane trzy rurociągi z wodą (dwa z zimną i jeden z ciepłą), gazociąg oraz ponad 20 kabli telekomunikacyjnych. Dostęp do nich oraz do konstrukcji od spodu, umożliwiały trzy kładki robocze o drewnianych pomostach zlokalizowane pomiędzy dźwigarami.

Pożar rozpoczął się podczas wymiany tych pomostów od miejsca składowania zdemontowanych elementów drewnianych pod przęsłem 12-13. Pożar rozpoczął się wieczorem i został ugaszony nazajutrz rano. Do tego czasu objął swoim zasięgiem trzy przęsła.

Na drugi dzień po tym zdarzeniu Instytut Dróg i Mostów Politechniki Warszawskiej został poproszony o wykonanie ekspertyzy uwzględniającej: badania materiałowe, weryfikację geometrii konstrukcji oraz analizę wytrzymałościową MES.

Tematem niniejszej publikacji są zagadnienia związane z weryfikacją geometrii konstrukcji, która, choć najszybsza do przeprowadzenia, była utrudniona z powodu braku danych odniesienia (dokumentacja archiwalna była niekompletna i nie zawierała informacji o niwelecie obiektu sprzed pożaru).

Jako pierwszą zweryfikowano geometrię pomostu, gdzie zniszczenia były najbardziej widoczne w postaci licznych wybrzuszeń nawierzchni w obrębie pasów ruchu znajdujących się pomiędzy środkowymi dźwigarami. Największe wybrzuszenia zaobserwowano w przypadku przęsła skrajnego 12-13. Kontrola wizualna pomostu od spodu ujawniła liczne deformacje żeber podłużnych pomostu ortotropowego - uszkodzenie to występowało zarówno w przypadku pasów zdeformowanych, jak i pozornie nienaruszonych pasów dla autobusów. Analiza geodezyjna niwelet w obrębie pasów dla autobusów wykazała, że niweleta obiektu w ich osiach wykazuje znaczne odstępstwa od niwelety spodziewanej. Wykonano lokalna analizę zachowania się fragmentu konstrukcji pod wpływem podwyższonej temperatury. Mimo przyjęcia mocno uproszczonych warunków brzegowych otrzymano charakter uszkodzeń podobny rzeczywistych odkształceń konstrukcji.

Następnie wykonano pomiary geodezyjne niwelet dolnych półek dźwigarów. Wykazały one, że na odcinkach trzech spalonych przęseł przemieszczenia pionowe były znacznie większe od przemieszczeń pionowych analogicznych fragmentów części niespalonej. Ponadto wzajemne porównanie symetrii przekroju wykazało, że dźwigary północne mają znacznie większe przemieszczenia pionowe niż dźwigary południowe, mniej narażone na działanie temperatury w związku z sąsiedztwem rurociągu oraz północnym kierunkiem wiatru podczas pożaru. Dodatkowe pomiary dotyczące pionowości dźwigarów zewnętrznych wykazały z kolei, że na odcinku trzech spalonych przęseł przekrój poprzeczny

konstrukcji uległ skręceniu w kierunku północnym. Przy wszystkich tych pomiarach odnoszono się do dwóch niespalonych dźwigarów.

Pomiary lokalnej geometrii wybranego do analizy środnika dźwigara wykazały wiele wybrzuszeń lokalnych w polach pomiędzy żebrami pionowymi środnika. Wartości tych przemieszczeń poziomych znacznie przekraczały wszelkie wytyczne dla tego typu konstrukcji.

Zmierzono przemieszczenia konstrukcji na łożyskach. Wykazano, że łożyska przesuwne na skrajnej podporze – nr 13 – są wychylone maksymalnie i istnieje zagrożenie, że w przypadku spadku temperatury konstrukcja może spaść z łożysk.

Poza powyższymi, ekspertyza uwzględniała również występowanie wielu innych uszkodzeń, takich jak: uszkodzenia dylatacji, zniszczenia konstrukcji stalowej pomostu roboczego, zniszczenia nawierzchni, barier ochronnych, powłok malarskich, instalacji obcych itd.

Generalnie należy stwierdzić, że konstrukcja mostu została bardzo mocno uszkodzona na skutek pożaru i główną przyczyną był tu skurcz termiczny stali. W związku z różnymi warunkami brzegowymi dla różnych elementów konstrukcji mostu w momencie stygnięcia konstrukcji pojawiły się liczne deformacje resztkowe, eliminując wybrane elementy z dalszego użytkowania.

Ostatecznie podjęto decyzje o wymianie konstrukcji stalowej przęseł. Wykonawca zdecydował się na zastosowanie zmodyfikowanej metody nasuwania podłużnego. Metoda ta uwzględniała wykorzystanie istniejącej konstrukcji przy montażu nowych elementów, pozwalając uniknąć schematu wspornikowego podczas nasuwania konstrukcji i tym samym pozwalając na zmniejszenie liczy podpór tymczasowych. Podczas jednego z etapów budowy schemat statyczny istniejącej konstrukcji – w związku z odcięciem przęseł skrajnych i rozcięciem konstrukcji w przęśle środkowym – sprowadził się do schematu belek jednoprzęsłowych ze wspornikami. Jedną z nich stanowiło przęsło niespalone, drugą przęsło spalone. Porównanie ugięć obydwu przęseł pokazało słuszność podjętej decyzji o rozbiórce części stalowej obiektu. Ugięcia w przęśle spalonym były dwukrotnie większe od ugięć w przęśle niespalonym. Spowodowało to dodatkowe komplikacje dla wykonawcy – należało wprowadzić dodatkową podporę tymczasową. Ostatecznie most po wymianie oddano do użytku w dniu 28 października 2015 roku.