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Research paper

Analysis of the impact of widening the bridge deck on the redistribution of the internal forces in the structure on the example of the bridge in Opole, Poland

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Abstract: Redistribution of internal forces in existing bridge structures occurs very often in practice, especially when they are functionally modernized (e.g. by widening the deck), as well as as a result of changing the static system of the structure. The most telling and relatively common example of the latter situation is the elimination of hinges (e.g. hinges in Gerber systems or hinges in the middle of the span of concrete bridges erected using the cantilever concreting method) and thus creating a continuous hingeless system. As a result of this change, internal forces are redistributed, and insufficiently thorough consideration of it can lead to serious, negative consequences. The article analyzes a real bridge structure located in Opole. The structure accepted for testing has a complicated structural system and has Gerber hinges. The lower flanges of the girders have a variable width along the entire length of the spans – therefore almost every span cross-section of the structure differs from each other. The bridge structure was modelled in the numerical programme SOFiSTiK, which managed to reproduce the diversity of the main girders (variable cross-sections). Work on the model was divided into several stages. The first one was a comparison of internal forces for the state before 2015, obtained from loads according to the standard from the time of construction of the structure and Eurocode. The second stage was a change of the static scheme (widening the usable width of the roadway) and a comparison of internal forces and as above. The third part included the elimination of hinges. Based on the analyses performed, three cases were compared and conclusions were drawn. Looking at the results from loads according to the old standard (and the bridge construction process), we see no increase in the value of forces in variant A and B. The increase occurs only when the hinge is eliminated. Analyzing the results from PN-EN loads, we see that they cause an increase in the value of forces, which is related to a different load setting. Modern utility loads, according to Eurocode, generate greater internal forces, an increase of 70–100%. As can be seen in the example of a real object from Opole, even a small interference in the bridge construction system can have a significant impact on changing the arrangement of forces.

Keywords: bridge structures, bridge widening, numerical analysis, redistribution of internal forces

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1. Introduction

Many bridges both at home and abroad are in urgent need of modernisation and repair. These structures are often unsuitable for the increased traffic, and the load class for which they were once designed, as well as the dimensions of the vehicles. The load class for which they were once designed, as well as the dimensions of the vehicles and their speed, are now inadequate. There is therefore a need to adapt these facilities to the new traffic conditions, which often involves structural (mainly reinforcement) and functional modernisation (mainly changes to the geometric parameters of the facilities, e.g. widening of bridges). The consequence of such technical measures is always a greater or lesser redistribution of internal forces in the structure. However, as is well known, it is not only in the case of retrofitting that we are confronted with this phenomenon. The redistribution of internal forces in bridge structures may also have many other sources, such as transitions from assembly states to operational states during the construction of the structures, as well as rheological effects during their use.

2. Literature review

The subject of the redistribution of internal forces in bridge structures is a very broad, multi-faceted topic. The redistribution itself has its genesis in various aspects related to both the construction of bridges and their operation, repair, as well as failures and even – in extreme cases – catastrophes. A number of interesting studies can be found in the world literature. Many of them describe the topic of redistribution of forces in bridges in the event of a sudden failure, related, for example, to the impact of a vehicle on the underside of the structure, or intermediate supports. A Chinese team of authors [1, 2] has presented an engineering method for calculating the impact force between a high-rise truck and a bridge structure. The research is based on a simplified numerical model that can help engineers at the engineering design stage. Another team of researchers [3] undertook to determine the correlation between the design of bridges (mainly pillars) taking seismic problems (earthquakes in China) into account and their resistance to heavy vehicle impacts. Construction disasters are also a topic for research. For example, Naser M.Z., author of the [4] publication, made an in-depth analysis of bridge disasters and failures in the United States and major cases from around the world. He created an algorithm to predict the vulnerability of bridge structures to failure and an algorithm for the expected degree of hazard/damage to bridges following extreme loading.

Redistribution of internal forces also occurs in bridge structures as a result of retrofitting or repairs. In this case, it is very important to analyse the new force distribution beforehand so that the respective retrofitting, which also involves a change in the static arrangement of the structure, is successful. A good, and very spectacular, example of neglect in this respect is the Koror bridge over the Pacific Koror and Babekthuap in the Palau archipelago, built in 1978. The span of the longest span was 241 m, with an articulation formed in the middle. This joint declined over time, reaching 1.3 m after 18 years of operation of the structure, largely as a result of underestimating the effects of concrete creep. Concerned that the vertical displacement of the main span would continue to increase, it was decided to remove the hinge in the middle of the span and sectionally prestress the middle section of the long span. This led to a change in the original static scheme of the bridge structure, resulting in a redistribution of internal forces. Shortly after the aforementioned work was carried out, on 27 September 1996, the middle span of the bridge suddenly collapsed. As a result of investigations carried out by experts [5–7], it was proven that the very stiff bank spans were not designed to redistribute the forces generated by the renovation. Excessive concrete creep and shrinkage also contributed to the catastrophe, namely the underestimation of deflections and stress losses. The authors of publications [5–7] state that the bridge could probably have served for 100 years if repairs had not been undertaken. The old bridge was completely demolished and a new bridge with a suspended structure was built in its place, which was put into service on 11 January 2002.

There are many outdated bridge structures in the world that are not adapted to increased traffic volume and higher loads than they were originally designed for. It is important to assess the technical condition of the structures and determine which of them can be adapted to new conditions by widening the usable width of the roadway. The authors of the study [8] focus on monitoring the safety of bridges in order to significantly reduce the risk of unexpected collapse, as well as on effective maintenance of bridges using integrated management systems. The proposed classification of Bridge Management Systems indicates the expected directions of development, taking into account changing challenges and integrating new developing technologies, including automation of decision-making processes using elements of augmented reality (AR) and virtual reality (VR) [9] In the study [10], the team of scientists presents parametric models for preliminary estimation of investment costs, analyzing 5 bridge structures in their work. Parametric models are to provide greater accuracy and flexibility in estimating expenses, by taking into account variables specific to each project, such as the type of bridge, geometric parameters and applied technical solutions. In the world literature, we can find examples of structures where the roadway width was widened, and the authors present aspects related to the technology of construction, additional reinforcement of the structure, and problems that arose during the implementation of the task [11–14]. In the article [15], the structural aspects related to the design of the structure widening and the challenges facing the process of widening the existing bridge structure were discussed.

2.1. Importance of research

This paper presents an example of a bridge structure whose bridge deck has been widened from two lanes to three lanes, and presents numerical analyses to show what changes take place in the force distribution of its structure as a result. The influence of this widening on the redistribution of the internal forces in the structures is of generally important problem. The relevant analyses are rather seldom published so far. On the other hand, however, they seem to be of fundamental meaning for functional modernization of bridge infrastructure, bridge maintenance as well as brigde service. Therefore, this problem requires more reseach. It should be also emphasized that ine spite of the general theoretical analyses, each case should be considered individually.

3. The analysed bridge

3.1. Description of facility

The analysed bridge is located in Opole, in the course of Wrocławska Street (DW 414 at km 180+858) and runs over the so-called Relief Canal. It is a reinforced concrete bridge built in 1938 (Fig. 1). The superstructure is a four-span, three-beam, gerbered superstructure located at an oblique angle of 68 to the obstacle. The main beams have variable height and width of the webs; in the area of the pillars, they are reinforced with a lower plate passing towards the hinges into a flange of variable width [16, 17]. The beam spans in the bearing axis are shown in Fig. 2.

Fig. 1. View of the bridge [own archive]

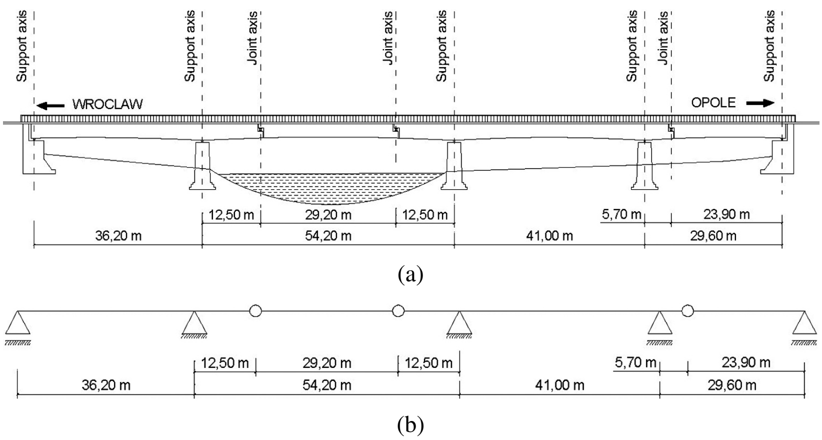

Fig. 2. Road bridge over Wrocławska Street in Opole: (a) view of the structure from the downstream side, (b) static scheme of the bridge [own elaboration]

The height of the beams varies from 2.05 m to 3.45 m, and the thickness of their webs varies from 0.60 m and 0.78 m in the centre of the spans to 1.50 m above the pillars and at the hinges [16, 17]. The top flange of the main beams is a 0.25 m thick deck slab that is constant along the entire length of the bridge. The transverse beams are 0.30 m thick. They are partly located perpendicular to the main beams with the exception that the support cross-bars and in the hinges are diagonal, parallel to the axis of the obstacle, and all expansion joints are located in this way. All cross-bars have rectangular ribs. The stringers are 75 cm high and 30 cm thick.

The reinforced concrete slab of the carriageway is 25 cm thick and has pavement supports extending beyond the outer girders with a 2.00 m overhang (Fig. 3). Three ribs were made within the pavement to support the pavement surface elements, namely: a handrail rib, a rib where the kerb separating the cycle path from the pavement is fixed and a rib at the stone kerb of the carriageway (Fig. 3).

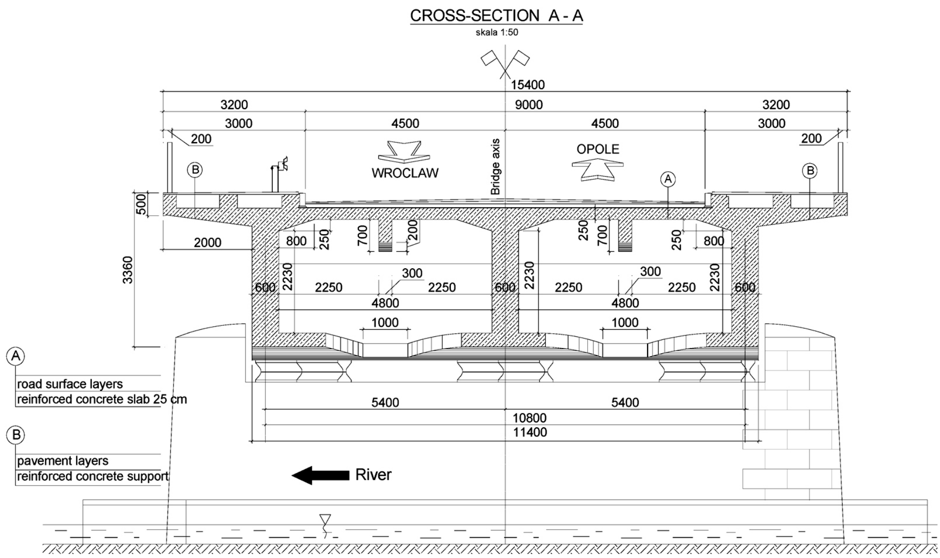

Fig. 3. Cross-section

The span was supported on massive reinforced concrete abutments, the body of which was laid parallel to the obstacle. The piers of massive construction were founded in sheet pile walls. The length of the pillar relative to its own axis is 16m and the thickness at the base is 3.23 m. The main girders of the bridge are supported on steel bearings.

The remaining geometric features are as follows [16, 17]:
- theoretical span length L_t: 48.7 + 29.2 + 59.2 + 23.9 m,
- support span: 36.2 + 54.2 + 41.0 + 29.6 m,
- total length of superstructure/facility: 166.5 m/ 180 m,
- structural/foundation/supporting structure height: 3.91 m/ 4.4 m/ 4.17 m,
- bridge height above ground/ clear height: 10.5 m/ 3.6 m,
- overall width of the structure: 15.4 m,
- width of carriageway/sidewalks: 9.0 m/ 3.00 + 3.00 m.

3.2. General technical condition of the bridge

The span structure, i.e. the main beams, crossbeams and deck slab showed many cracks ranging from 0.08 mm to 0.66 mm along the entire length of the bridge. The course of the cracks generally corresponded to the direction of the main tensile stresses, but many cracks ran in different arbitrary directions. In addition, a few areas where there was insufficient lagging were noted and corrosion of the rebar was observed. The slab was assessed as satisfactory. Few concrete voids and rebar corrosion were observed.

Significant cracking was observed on the abutments near the wing expansion joints. No damage indicative of foundation failure or overloading was found. Water leaking through the expansion joint caused significant cracking and loss of concrete in the foundation footings. In addition, microcracks and cracks were found on the surface of the pillar body. The bearings on which the girders rest are in satisfactory condition. Minor corrosion of the steel was observed.

3.3. Concrete strength tests on site

In order to identify the basic properties of the concrete of the Ulga Canal bridge, material tests had to be carried out. After a thorough study of the technical documentation of the bridge over the Ulga Canal in the course of Wrocławska Street, the results of tests on the structural elements of the structure, carried out by MOSTOPOL, were made known [18]. Tests were carried out using the non-destructive Schmidt hammer method. Measurements were taken in places where the concrete did not show signs of damage. On the basis of the Schmidt hammer sclerometric tests and the tests of the core samples taken, it was concluded that the concrete of the load-bearing structure corresponds to class C40/50 [18].

Another concrete test was carried out while the bridge was undergoing an upgrade, which involved widening the carriageway by 0.5 metres at the expense of the pavement on the left side. Cylindrical samples were taken for testing from the elements that formed the footing for the pavement slabs in the pavement structure. The samples were taken at the three locations shown in Fig. 4. The surface of the samples was adjusted using the pen overlay method (according to PN-EN 12390-3:2011 appendix A – by applying a levelling layer [19]) and then tested using Method A (according to PN-EN 12390-13:2014 [20]). The results of the tests obtained are summarised in Tables 1 and 2. Taking into account the values obtained and the results of the concrete tests which were available in the technical documentation of the object, and the fact that the concrete in the studies of the author of the work came out showing a class in the range of C35/45÷C30/37 – concrete C35/45 was assumed for further numerical analyses.

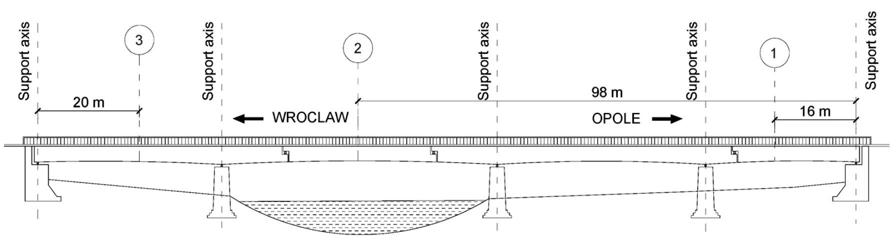

Fig. 4. Schematic side view with three core intake sites marked

Table 1. Concrete laboratory test results: compressive strength test results

Sample destination	Date of examination	F [N]	fc [MPa]	Destruction type
LMB 308- 1 /2015	19-06-2015	241400	34.8	correct type of destruction*
LMB 308- 2 /2015	19-06-2015	286400	41.3	
LMB 308- 3 /2015	19-06-2015	283900	40.9	
LMB 308- 4 /2015	19-06-2015	271600	27.2	
LMB 308- 5 /2015	19-06-2015	237800	23.8	
LMB 308- 6 /2015	19-06-2015	238300	34.3	
LMB 308- 7 /2015	19-06-2015	287900	41.5	
LMB 308- 8 /2015	19-06-2015	281300	40.5	
LMB 308- 9 /2015	19-06-2015	258800	37.3	
LMB 308- 10 /2015	19-06-2015	288400	41.6	
LMB 308- 11 /2015	19-06-2015	301600	43.5	
LMB 308- 12 /2015	19-06-2015	306000	44.1	

Note 1: All destructions of the tested samples indicate that the test was carried out correctlu, the types of destruction are in accordance with PN-12390-3:2011
Note 2: Notations used in the table: F – maximum load at failure,
fc – compressive strenght of the sample
Note 3: Compressive strenght was calculated based on the actual dimensions of the sample.
* correct type of destruction according to PN-12390-3:2011

Table 2. Concrete laboratory test results: secant modulus of elasticity of concreto

Sample designation	E_{C0}[GPa]	E_{CS}[GPa]
LMB 308- 2 / 2015	20.5	26.0
LMB 308- 3 / 2015	25.8	33.3
LMB 308- 4 / 2015	25.3	34.5
LMB 308- 6 / 2015	21.5	28.2
LMB 308- 7 / 2015	27.2	33.6
LMB 308- 8 / 2015	26.5	33.1
LMB 308- 10 / 2015	23.3	29.2
LMB 308- 11 / 2015	24.0	33.3
LMB 308- 12 / 2015	24.0	31.5

Note 1. The following designation have been adopted:
E_{C0} – initial modulus of elasticity
E_{CS} – stabilized modulus of elasticity
Note 2. Compressive strength of samples

4. Widening the usable width of the bridge

The bridge required changes to widen its usable width and eliminate the bottleneck that formed on the bridge during rush hour. Analysing the documentation available at the Municipal Road Administration in Opole concerning the analysed bridge, a document [18] was consulted, in which an analysis of the structure was carried out together with proposed renovation concepts, taking into account a significant widening of the usable width of the bridge (up to 4 lanes, among others). The concepts show that it is possible to rebuild the structure improving the bridge's performance. However, this would require a major reconstruction of the span together with a thorough overhaul of the Gerber joints, which carries considerable costs. The technical report [18] also points out the possibility of leaving the structure in its existing traffic condition and dispensing with reconstruction, but with the overhaul of many components of the structure carried out.

The bridge was finally repaired in 2015, which did not use any of the proposed concepts, and included injecting cracks in the main girders, repairing the top layer of concrete of the supports by shotcreting, cleaning the sub-base benches of the supports, making a new expansion cover and repairing the bridge equipment including replacing the lanterns on the structure. In addition, an extra lane was provided on the bridge, namely the carriageway was widened by 0.5 metres at the expense of the pavement on the left-hand side (as seen from the city centre) (Fig. 5). The girders of the bridge have not been reinforced and the Gerber joints have not been repaired either.

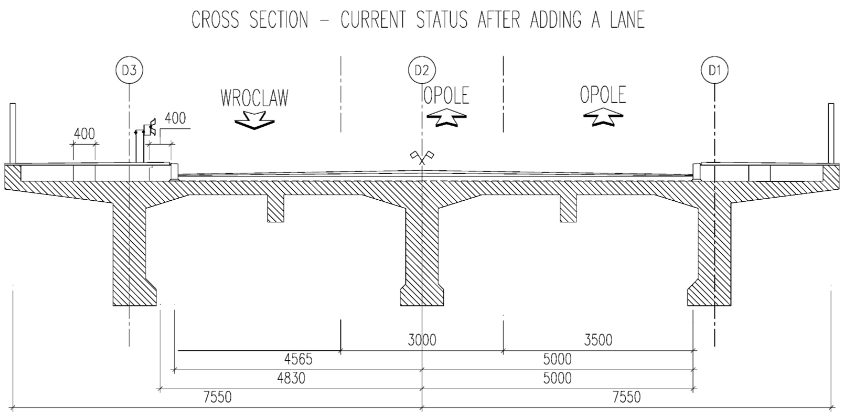

Fig. 5. Schematic cross-section of the bridge – current state with three lanes

Several years after the renovation and the addition of an extra lane, the structure still bears traces of overloading, new cracks and defects are visible. This is understandable, as the structure, which was not load-bearing at the time of the earlier analyses and expert reports (there was even talk of reducing its load-bearing capacity), was additionally overloaded by the new lane and thus a higher volume of vehicles. This immediately raises the question of the distribution of internal forces in the bridge structure, which have certainly changed and, in the new configuration, affect the operation of the individual girders, which were designed for a different load distribution. This and other relationships will be addressed and described further on.

5. Numerical analysis

A numerical analysis was carried out in which models were created using SOFiSTiK's professional bridge structure analysis software. The basis for these calculations are proprietary text algorithms based on the programme's environment library. The models were created using the Teddy editor, which makes full use of the CADINP language [21]. The models were made using the finite element method (FEM) as e1 p3 models [22]. Models of the structure were developed by maintaining ordered node numbering. The models were discretised in the longitudinal section at an average of 1 m intervals, while in the cross-section they were discretised according to the characteristic points (min. 0.5–1.0 m). In the case of the real object, the cross-sections adopted represent its real stiffness, and also take into account the structure's haunch in plan, as well as the variable geometry of the lower flanges and main beams. The hinges were designed as 'suspended', using a 'KINE' module with the possibility of longitudinal rotation, but without a torsion option [21].

The work was divided into three phases, each with three numerical models. The first stage of the analysis reflects the structural layout from the time of construction of the facility and in use until 2015 (hereafter referred to as Variant A – without widening). The second model contains the changes to the lane layout made after the upgrade (hereafter referred to as Variant B – with widening). The third stage assumes the same lane layout as in model No. 2, however with the assumption of the elimination of the Gerber joints (hereafter: Variant C – with widening, without joints).

Each model was loaded with two load systems. Firstly, the load applicable at the time of construction of the structure was set according to [23]. The models take into account assumptions in which internal forces, displacements and reactions are calculated in such a way that each beam carries 100% of the imposed load according to [23] (Fig. 6) (the transverse co-operation of the spans was then not taken into account in the present understanding, so that in this case analysed at present almost the entire weight of the cylinder fell on one beam) [17]. In each variant, the loading according to [23] was distributed in the same way (Fig. 6). For the second loading arrangement according to PN-EN 1991-2 [15], a lateral distribution according to the rigid cross-beam method was adopted, which results in approximately 50% of the live load falling on the girder. The distribution of the loads on each girder in variant A is shown in Figs. 7– 9. For the other variants, the second set of loads was distributed according to the principles of PN- EN 1991-2. For the remaining variants B and C (three traffic lanes) the second set of loads was distributed in accordance with the principles of PN-EN 1991-2. For comparison, the Fig. 10 shows the load distribution for the end girder D1.

It was decided to separate the girders into separate sub-models, as this was how the bridge was designed according to the 1920 standard. Attempts to load the entire spatial structure with three rollers (each girder was supposed to carry 100% of the road roller load according to the old standard) resulted in questionable results, and the structure would not be able to carry

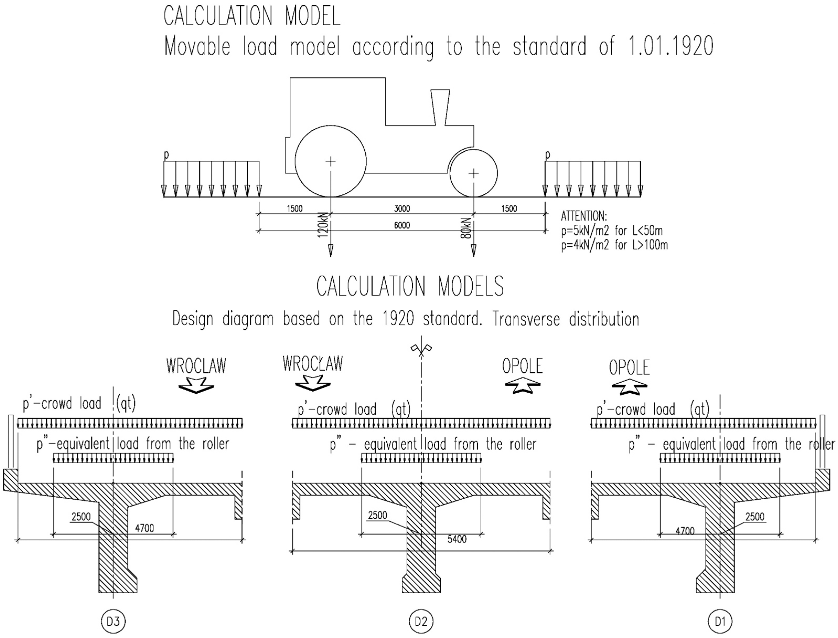

Fig. 6. Road roller loading scheme according to [23]

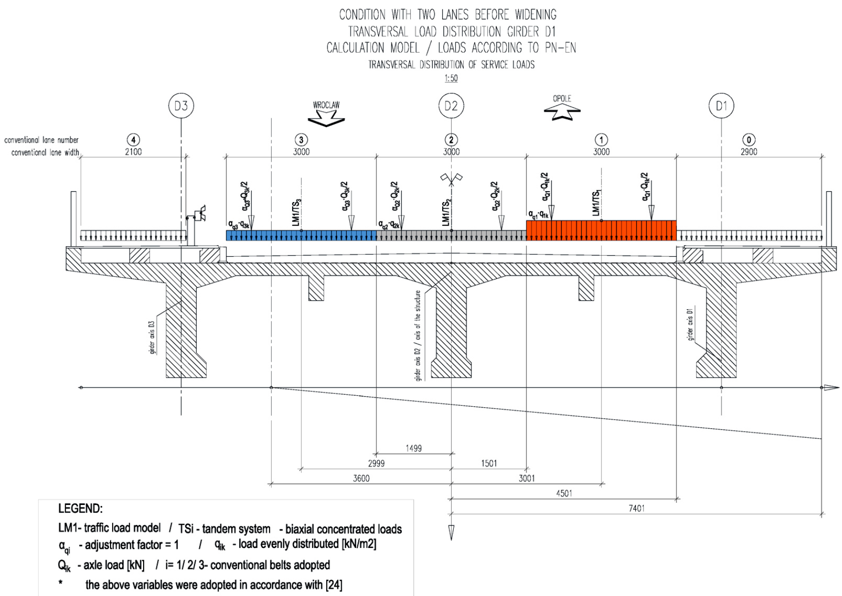

Fig. 7. Load distribution diagram according to PN-EN 1991-2 for edge girder No. 1 in variant A

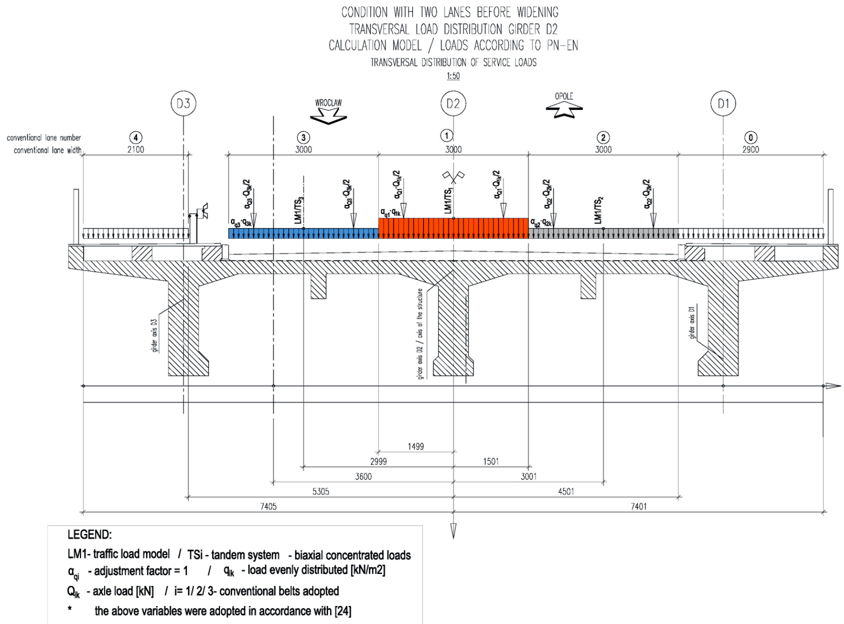

Fig. 8. Load distribution diagram according to PN-EN 1991-2 for edge girder No. 2 in variant A

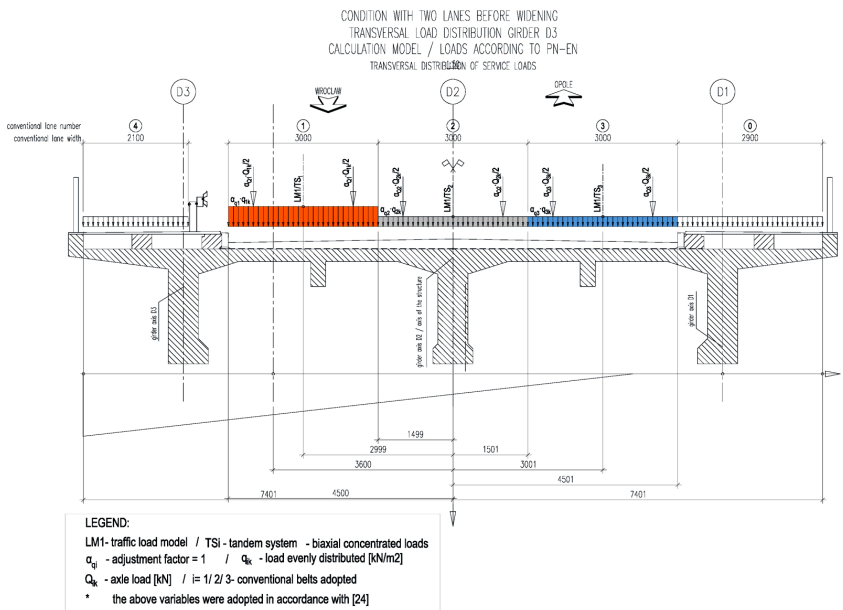

Fig. 9. Load distribution diagram according to PN- EN 1991-2 for edge girder No. 3 in variant A

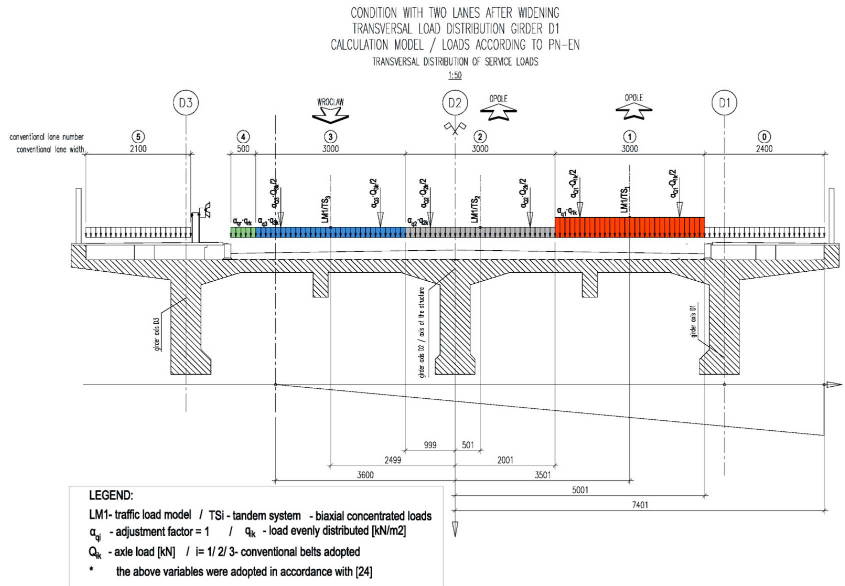

Fig. 10. Load distribution diagram according to PN-EN 1991-2 for edge beam no. 1 in variants B and C

the accumulated load. In the numerical analyses, three girders were therefore considered in each step, and the results of the calculation of the internal forces and displacements were then compared and described in the form of conclusions.

The computer calculation approach, in which each girder is considered separately, is correct and the decomposition according to the rigid cross-beam method is used in engineering practice. However, it should be noted that the values adopted in this way may or may not be higher. However, the methodology adopted here for the numerical analyses is correct and allows the analysis of the internal forces in the individual girders of the structure [22].

In the first stage, a structural layout was modelled (Variant A) which represents the condition of the structure during construction and in use until 2015. The bridge has two lanes in opposite directions each 4.5 m wide, and two 3.0 m pavements on both sides. The load distribution on each girder is shown in Figures 7–9.

In the second stage, the structural layout (variant B) was modelled, which represents the state of the structure after the 2015 upgrade. The bridge has three lanes (two towards Opole) (Fig. 5) each 3.0 m wide and two pavements on both sides (2.4 and 3.0 m).

In the third stage of work, the model was modified to make the structure tighter by removing the Gerber joints (variant C). The loads remain the same, only the static scheme of the structure changes (Fig. 10). The changes were made in the TEDDY text editor, using the SOFiMSHA tab and KINE commands (Fig. 11).

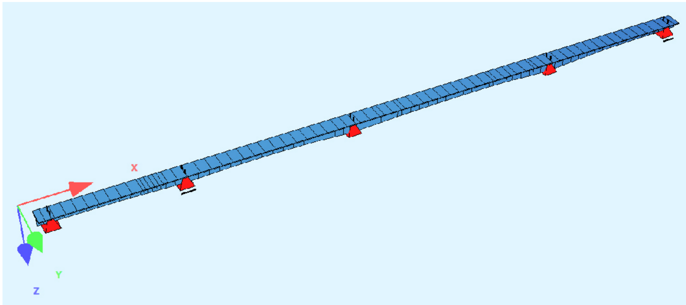

Fig. 11. Model of the extreme girder no. 1 in the SOFiSTiK program after eliminating the joints

6. Results of numerical analyses

Tables 3–5 show the summarised results of the internal forces obtained from the SOFiSTiK reports. Looking at the results from the loads according to the old standard, we can see that there is no increase in the values of the forces in variants A and B. An increase only occurs when the hinge is removed. When analysing the results from PN-EN 1991-2 loads, we see that they cause an increase in force values, which is related to a different load setting. Modern service loads, according to Eurocode, generate internal forces that are higher, an increase of 70–100%. As can be seen from the example of the real object from Opole, even a small interference in the structural system of the bridge can have a significant impact on the change in the arrangement of forces. In the case of the analysed structure, no structural reinforcement was decided during the previously mentioned 2015 upgrade, and no numerical studies were

Table 3. Summary of results for the three models – Outrigger No. 1

	End girder no.1								
	***S* – shear for ces [kN]**					***M* – bending moments [kN·m]**			
	values	Standard from 1920	PN-EN 1991-2	difference	%	Standard from 1920	PN-EN 1991-2	difference	%
Variant A	max	717.3	1209	491.7	69	5930	10495	4565	77
	min	–712.2	–1202	–489.8	69	–7019	–12390	–5371	77
Variant B	max	717.3	1347	629.7	88	5930	11729	5899	98
	min	–712.2	–1339	–626.8	88	–7019	–13843	–6824	97
Variant C	max	762.4	1442	679.6	89	4570	9021	4451	97
	min	–766.6	–1439	–672.4	88	–8044	–14331	–6287	78

Table 4. Summary of results for the three models – Centre beam No. 2

	Middle girder no.2								
	***S* – shear for ces [kN]**					***M* – bending moments [kN·m]**			
	values	Standard from 1920	PN-EN 1991-2	difference	%	Standard from 1920	PN-EN 1991-2	difference	%
Variant A	max	789.7	843.2	53.5	7	6515	7432	917	14
	min	–788.8	–842.2	–53.4	7	–7683	–8740	–1057	14
Variant B	max	789.7	843.2	136.9	17	6515	8125	1610	25
	min	–788.8	–842.2	–136.9	17	–7683	–9558	–1875	24
Variant C	max	849.3	986.8	137.5	16	4994	6134	1140	23
	min	–852.2	–983.7	–131.5	15	–9176	–9924	–748	8

Table 5. Summary of results for the three models – Outrigger No. 3

	Middle girder no.3								
	***S* – shear forces [kN]**					***M* – bending moments [kN·m]**			
	values	Standard from 1920	PN-EN 1991-2	difference	%	Standard from 1920	PN-EN 1991-2	difference	%
Variant A	max	711.9	1165	453.1	64	5928	10198	4270	72
	min	–715.4	–1170	–454.6	64	–7091	– 12128	–5037	71
Variant B	max	711.9	1213	501.1	70	5928	10607	4679	79
	min	–715.4	–1218	–502.6	70	–7091	– 12615	–5524	78
Variant C	max	762.5	1296	533.5	70	4659	8263	3604	77
	min	–764	–1291	–527	69	–7961	– 12773	–4812	60

carried out to determine the impact of this change. In the technical documentation, only expert reports of the facility dated 2012 (i.e. before the widening) were available, as well as proposals for widening the carriageway width to 4 lanes with additional external structural reinforcement. It is therefore necessary to consider whether it would be possible to widen the facility to 4 lanes in the future and whether the design of external structural reinforcement would allow the transfer of more vehicle traffic. The structure has been in operation for eight years after the upgrade and while its condition is satisfactory, it is uncertain for how long. Already, additional scratches and efflorescence can be seen on the facility, which has appeared over the last few years. The concrete defects in the lower zone visible in Fig. 12 relate to girder

No. 1, which takes the heaviest loads (the carriageway has been widened to 0.5 m above it). Continuous real-time monitoring of the structure should be considered, and further numerical studies of the structure should be carried out at the same time for comparison purposes.

Fig. 12. View of girder 1 in the strand bay. Concrete defects and numerous cracks in the bottom zone of the girder

7. Conclusions and concluding remarks

As engineering practice shows, the phenomenon of redistribution of internal forces, being a consequence of the modernisation of structures, is not always sufficiently analysed by designers and implemented with due care and skill by contractors. This negligence can lead to serious consequences, even catastrophes. There are unfortunately quite a few examples of this. Looking at the results of the numerical analyses presented here, we can see that even a relatively small interference with the bridge structure affects the redistribution of internal forces. This change can be a positive change that improves the performance of the structure, but it can also be a change that unexpectedly causes a threat to the technical condition of the structure. It is very important to compare the changes taking place in order to determine the nature of the changes and their subsequent impact on the structures.

As can be seen from the example of the Opole structure, even a slight widening of the carriageway has an impact on the load distribution and thus on the internal forces in the structure, which can affect the technical condition of the structure in the later years of its life. In the case of old bridges where Gerber hinges have been designed, their removal should be considered in detail, as it is with this procedure that the greatest changes in internal forces occur.

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Analiza wpływu poszerzenia płyty pomostowej na redystrybucję sił wewnętrznych w konstrukcji na przykładzie mostu w Opolu

Słowa kluczowe: analiza numeryczna, konstrukcje mostowe, poszerzenie mostu, redystrybucja siłwewnętrznych

Streszczenie:

Do redystrybucji sił wewnętrznych w istniejących obiektach mostowych dochodzi w praktyce bardzo często, przede wszystkim gdy są modernizowane w sposób funkcjonalny (np. przez poszerzenie pomostu), a także wskutek zmiany układu statycznego konstrukcji. Najbardziej wymownym stosunkowo najczęstszym przykładem tej ostatniej z wymienionych sytuacji jest likwidacja przegubów (np. przegubów w układach Gerbera lub przegubów w środku rozpiętości przęseł mostów betonowych wznoszonych metodą betonowania nawisowego) i utworzenie w ten sposób ciągłego układu bezprzegubowego. W wyniku tej zmiany dochodzi do redystrybucji siłwewnętrznych, a niedostatecznie wnikliwe jej uwzględnienie może prowadzić do poważnych, negatywnych następstw. W artykule dokonano analizy rzeczywistego obiektu mostowego zlokalizowanego w Opolu. Obiekt przyjęty do badań ma skomplikowany układ konstrukcyjny i posiada przeguby Gerbera. Dolne półki dźwigarów mają zmienną szerokość na całej długości przęseł – także prawie każdy przekrój przęsłowy obiektu różni się względem siebie. Wykonano ocenę stanu konstrukcji, a także przedstawiono wyniki badań betonu, które zostały wykonane w trakcie remontu obiektu przeprowadzonego w roku 2015. W trakcie wspomnianej modernizacji utw orzono dodatkowy pas ruchu dla pojazdów, zmniejszając przy tym szerokość jednego z chodników. W nowym układzie most posiada trzy pasy ruchu, dwa w kierunku centrum Opola, jeden w kierunku Wrocławia. Konstrukcję mostu zamodelowano w programie numerycznym SOFiSTiK, w którym udało się odwzorować zróżnicowanie dźwigarów głównych (zmienne przekroje). Prace podzielono na trzy etapy, a każdy z nich zawiera trzy modele numeryczne. Pierwszy etap analizy, to odwierciedlenie układu konstrukcyjnego z czasu budowy obiektu oraz użytkowania do 2015 roku (wariant A – bez poszerzenia). Drugi model zawiera zmiany w układzie pasów ruchu dokonanych po modernizacji (wariant B – z poszerzeniem). Trzeci zakłada układ pasów jak w modelu nr 2 jednak z założeniem likwidacji przegubów Gerbera (wariant B – z poszerzeniem, bez przegubów). Każdy model obciążono dwoma

układami obciążeń. W pierwszej kolejności zadano obciążenie obowiązujące w czasie wznoszenia obiektu zgodnie z *Betonkalender 1931r. Taschenbuch fur Beton und Eisenbetonbau sowie die verwandten Facher*. Modele uwzględniają założenia, w których siły wewnętrzne, przemieszczenia i reakcje są liczone w taki sposób, że każdy z dźwigarów niesie 100% obciążenia użytkowego- nie uwzględniono wówczas w obecnym rozumieniu współpracy poprzecznej przęsełtak, że w tym analizowanych obecnie przypadku prawie cały ciężar walca przypadałna jedną belkę. W przypadku drugiego układu obciążeń wg PN-EN 1991-2 przyjęto poprzeczny rozkład według metody sztywnej poprzecznicy, co powoduje, że na dźwigar przypada około 50% obciążenia użytkowego.Na podstawie przeprowadzonych analiz dokonano porównania trzech przypadków i wyciągnięto wnioski. Patrząc na wyniki od obciążeń według starej normy widzimy brak wzrostu wartości siłw wariancie A i B. Wzrost następuje dopiero przy likwidacji przegubu. Analizując wyniki od obciążeń PN-EN widzimy, że powodują one wzrost wartości sił, co związane jest z innym ustawieniem obciążenia. Obciążenia współczesne użytkowe, wg Eurokodu, generują siły wewnętrzne większe, wzrost o 70–100%. Jak widać na przykładzie rzeczywistego obiektu z Opola, nawet niewielka ingerencja w system konstrukcyjny mostu może mieć istotny wpływ na zmianę układu sił. W przypadku analizowanego obiektu, w czasie poprzednio wzmiankowanej modernizacji z 2015 roku, nie zdecydowano się na wzmocnienie konstrukcji, a także nie przeprowadzono badań numerycznych mających na celu określenie wpływu tej zmiany. Obiekt po modernizacji funkcjonuje 8 lat i co prawda jego stan jest zadowalający, jednak nie ma pewności na jak długi okres czasu. Już teraz widać na obiekcie dodatkowe rysy i wykwity, które pojawiły się na przestrzeni ostatnich lat. Należy zatem zastanowić się czy możliwe byłoby w przyszłości poszerzenie tego obiektu do 4 pasów ruchu i czy zaprojektowanie zewnętrznego wzmocnienia konstrukcji pozwoliłoby przenieść większy ruch pojazdów.

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