sciendo Studia Geotechnica et Mechanica, 2023; 45(2): 89–111

Original Study Open Access

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The new railway hybrid bridge in Dąbrowa Górnicza: innovative concept using new design method and results of load tests

https://doi.org/10.2478/sgem-2023-0002
received August 31, 2022; accepted November 25, 2022.

Abstract: The article presents a prototype steel–concrete bridge with the results of trial load tests. In the design of the structure, new approaches were used, the so-called concept of a hybrid cross section. The obtained results were interpreted against the background of theoretical analysis performed and the experience of the behavior of the existing standard bridge structures. The obtained results are to be the starting point for the development of methods of calculating this type of structure, with particular emphasis on the degree of cracking of the concrete part of the structure. The paper is intended to be a starting point for demonstrating that it is possible to calculate longitudinal shear in the fatigue limit state (of steel dowels) differently than in the fully cracked section. Similarly, it is supposed to be a point of discussion on how to perform a global analysis of hybrid systems.

Keywords: hybrid steel-concrete section; bybrid beam; hybrid bridge; composite dowels; lever arm; shear design.

1 Introduction

History shows that in bridge engineering, practice tends to precede theory; engineers introduce new solutions and then reveal unknown phenomena and develop appropriate scientific theories aimed at their appropriate consideration in design. On the one hand, such a procedure is burdened with risk, and on the other hand, it is an excellent source of new knowledge about new types of structures and enables progress. When creating the bridge in Dąbrowa Górnicza, the authors of the project believed that the previously acquired knowledge [1] made it possible to safely build the bridge in the technology they proposed without additional laboratory tests (which was positively verified by reality later on). The authors decided to design the object in accordance with the emerging concept of a general composite section [3] and then study its behavior on a natural scale. Although the engineering goal was to build an innovative bridge in a new technology, in this particular case, the scientific aspect of the issue is emphasized. A particularly important and main scientific goal (aim 1) was to learn about the phenomenon of concrete cracking of the so-called "combined general composite section" [3] (currently, according to [3] and [42], such a cross section would be called a combined hybrid cross section [CHC]). The purpose of the performed load test was to investigate the stiffness of a new type of structure (and tension-stiffening effect) and to compare the results with the theory for classic reinforced concrete. The second important issue was to draw scientists' attention to the problem of "parallel" application of two types of shear connection in a hybrid structure to transfer the longitudinal shear flow (aim 2). Finally, the third issue was the demonstration of the usefulness, validity, and, at the same time, the necessity to apply a new kind of approach, called the concept of hybrid cross section [45], in the theory of structures. The presented structure is the first engineering structure which used the division of the shear force into the part carried by the steel T-sections and the part carried by the concrete web according to the lever arm concept proposed in [3]. Such an approach will be implemented in the European Technical Specification CEN-TS 1994-1-102 "Design rules for the use of Composite Dowels" design regulations that are just being developed [43,44] (the experience gained during the design of the described bridge was used in its development). The key issue is how to model the type of structure presented in this article. The problem of modeling is presented in [42], and this article shows how modeling with various techniques influences the results of the calculations which were compared to the values of displacements measured during real studies of the object

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on a natural scale (aim 4). In addition to these four main issues, other issues were considered, which are presented later in the text (they are cognitively important and often complicated, but, according to the authors, are of a lesser scientific importance). To clarify, the authors use the term “hybrid” [43,44,45] and not “general composite” [3] in the definition of the cross section type, despite the fact that the former did not exist at the time of designing the bridge structure and was technically designed in accordance with the approach [3]. The introduction of the concept of hybrid [42] is related to the conducted scientific analysis [22,44] which sanctions the approach according to [3] and clarifies certain issues. In fact, the bridge was designed in accordance with the concept currently proposed in [44] as a hybrid. As probably in the future, the term general composite will no longer be used (and it has never been widely known), the authors considered such a procedure to be justified: using the name of the newly introduced concept when presenting a bridge in which it was basically used for the first time. Apart from the quotations cited directly in the manuscript in references, there are other items, not directly cited in the text, which are significantly related to the subject matter presented; when writing this article, the authors used these items, and thus, the knowledge of their existence can help the reader to analyze the text and expand the knowledge.

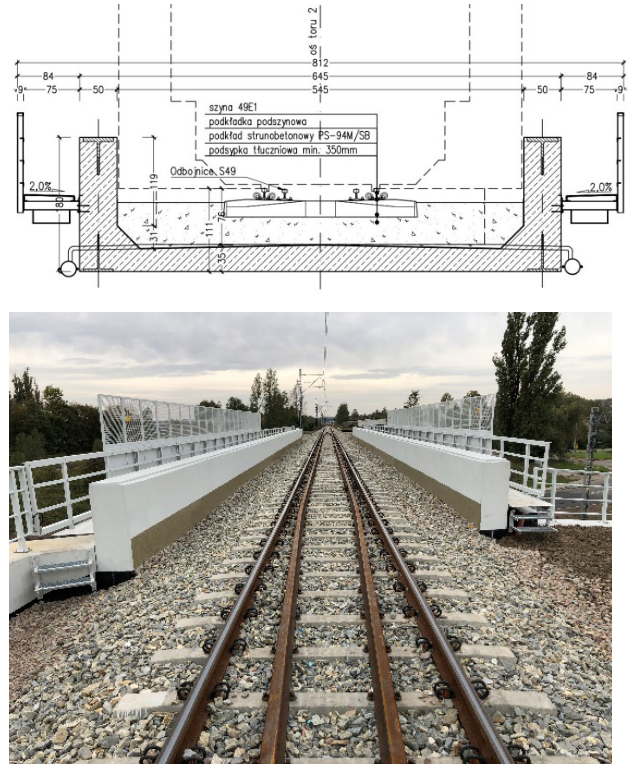

Figure 1: Dąbrowa bridge (picture by Nowak MOSTY).

Figure 2: The idea of external reinforcement by SSF Ingenieure applied in the trough girders (both in the longitudinal and transversal directions of the slab for railway bridges) in the bridge located next to Spergau [9, 1] (picture by Guenter Seidl).

2 The concept used in the bridge

The viaduct in Dąbrowa Górnicza is a prototype structure with a new type of steel–concrete cross section (looking from the point of view of knowledge at present [43] and not during the construction of the object [3]); both the concept used to design the structure and the structure itself are innovative. It is the first structure in the design of which, at the design stage, the proposed new rules for calculating the cross section due to shear force [3] were applied, which, after being verified [22] and established [43], will soon be implemented in the European design rules [44]. The superstructure is a two-span continuous beam with spans 21.30 m + 23.45 m. The cross section of the structure (Fig. 1) is the so-called combined (double) hybrid cross section, that is, one in which the longitudinal shear stream is divided into a steel and a concrete part (and not only goes from the concrete web to the steel web as it is in the case of the simple hybrid cross section). It is emphasized that while the change in the design method is a qualitative change, the structure itself is an evolution of the solution proposed by the SSF Ingenieure and we are dealing here with a quantitative change. But this bridge is a good example of a structural case, when a quantitative change in a design solution (here, a change in the relation of the steel web to the concrete web height) forces a qualitative change (change in the design method) at the design level (see Fig. 4in [3]).

The idea of using external reinforcement in the trough girders for railway bridges (Fig. 2) was proposed by SSF Ingenieure and it was applied for the first time (and only one time) in 2012 in a small bridge next to Spergau (Fig. 2)

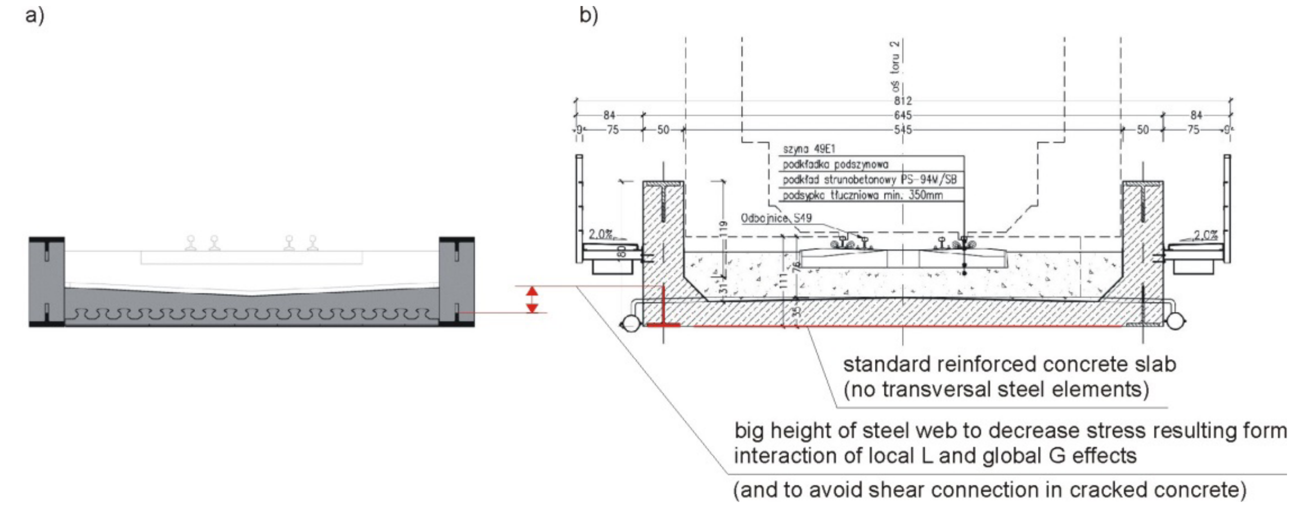

Figure 3: The main differences between 13 m span bridge next to Spergau by "SSF Ingenieure" design office and 23 m bridge in Dąbrowa Górnicza by "FASYS Mosty" design office (reinforced concrete slab and enlarged height of webs of T-sections): a) bridge by SSF using external reinforcement, b) bridge in Dąbrowa Górnicza using general composite section.

in Germany; this bridge is described among the other solutions using composite dowels in [9]. This structure has a small span frame, and it uses external reinforcement in both longitudinal and transversal directions. The cross section of this bridge is shown in Fig. 3a.

The cross section of the bridge in Spergau was a prototype for the bridge in Dąbrowa (the cross section of the bridge in Dąbrowa is shown in Fig. 2), while a significant change was made: the level of the shear connection was changed using higher T-sections. As already mentioned, it is a quantitative change at the level of the cross section structure (it does not change its topology), but it leads to a qualitative change at the design level: for the first time, the cross section was designed as a hybrid cross section according to the concept proposed in [3]. When designing the cross section of the bridge in Dąbrowa, the conservative assumption regarding the lever arm limited by the levels of shear connection (conservative assumption is used in Germany for externally reinforced sections) was abandoned (see Fig. 6-1 in [15]). In engineering considerations, this is a break with the conservative assumption, but in the scientific sense, it is a break with a concept that is generally just incoherent: the shear flow in the reinforced concrete part (and steel webs) depends mainly on the lever arm and not on the position of the shear connection in the cross section [1]. On the other hand, the position of the shear connection and concrete cracking conditions may affect the formation of the S–T mechanism, which is currently the subject of research.

The fundamentals of theory of hybrid cross section (formerly called general composite section) are presented in [3], and this concept is now being implemented to formal design rules at the European level [44,45] after external verification [22]. This is not a scope of the current paper; this is presented in [43]. The problems and experiences resulting from the implementation of a specific structure in the form of a viaduct in Dąbrowa Górnicza are presented here. An important question will be how to carry out a global analysis of such structures, taking into account concrete cracking, rheology, and the shear lag effect. At the beginning, the design and construction of the bridge were described in order to clearly present the background to the further scientific issues. During the design process, new scientific issues were identified, questions requiring answers were asked, and some answers were obtained during the tests of the constructed structure. In particular, it was firstly confirmed experimentally that the behavior of the structure corresponds to the predictions of the new cross section concept. Secondly, it was shown that the tension-stiffening effect and concrete cracking are very important factors that should be taken into account when creating the rules for global analysis. Third, the issue of the behavior of the cross section, which uses two types of shear connection at different heights, was raised. At the same time, engineers and scientists managed to successfully create a new solution for a railway bridge, which served as a prototype for the design of other facilities (bigger span and no T-sections at the transversal direction). It is emphasized that the main difficulty in modeling a structure (using beam elements

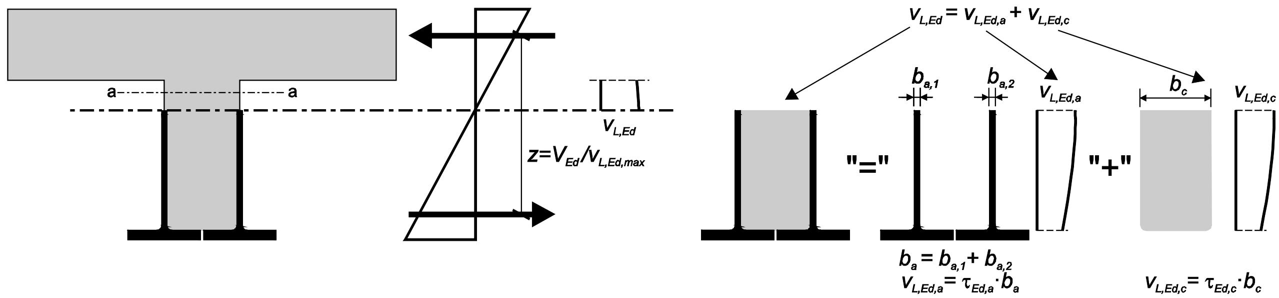

Figure 4: Distribution of longitudinal shear in a combined general composite section [3].

Figure 5: General view of the bridge (picture by Nowak Mosty).

for steel–concrete sections) with CHCs is the appropriate consideration of the division of the shear flow (Fig. 4), taking into account concrete cracking; the simplified rules used in Eurocode 4 [35] for accepting an uncracked plate for the determination of shear flow [41] do not apply.

Moreover, there is a problem of interpretation: which concept should be used to determine the effective width of flanges for shear lag – whether according to Eurocode 4 (as for composite structures) or in accordance with Eurocode 2 (as for concrete structures)? This issue raised in [21] is particularly visible in the case of the structure in question.

3 Design solution and constructed structure

The new solution was used in place of the existing steel truss [10]. The construction of the bridge (the photos and videos) is presented on the website [48]. This article omits the engineering issues related to the construction itself ([10] and [48] provide the necessary information) and focuses on the novelties in the adopted design solution. General view of the bridge is presented in Fig. 5 and upper view is presented in Fig. 1 (right). Only the most important information, as for engineering matters, is given here in order to enable independent verification of the calculations (more is given in [10]): the superstructure has a static scheme of a continuous two-span beam with spans 21.30 m + 23.45 m.

The slab of variable thickness from 31 to 35 cm is designed of C40/50 class concrete (according to Eurocode 2). The oblique angle of the structure is 67°. The axial transverse spacing of the main girders has been adapted to the requirements of the railway gauge and is equal to 5.95 m, and the thickness of their webs is 50 cm. In shaping the cross section of the main girders, sections HLB1100 and HL1100x607 (in the upper zone above the intermediate support) were cut in half of the height and used as T-sections. The height of hybrid sections is 180 cm. Steel grade is S460 (according to Eurocode 3). When shaping the structure, it was decided not to use additional rigid reinforcement of T-sections in the transverse direction (contrary to Fig. 2) due to the fatigue capacity of the connection of these elements with the lower chords of the main girders.

The key issue is the adopted reinforcement layout shown in Fig. 6–9; reinforcement ratios are crucial in the context of cracking of the concrete parts, therefore a detailed design solution is provided. Steel T-sections are produced using HL1100B and HL1100x607 profiles. They have been produced Spring 2019 in Luxembourg by ArcelorMittal (Fig. 10). They have been delivered on site (Fig. 11) where they have been welded to create steel structure of composite span (Fig. 12). Transversal steel elements have been used for edge crossbeams only because no longitudinal tension appears here and it is justified by the values of internal forces (Fig. 13).

It is important that the HL1100x607 cross section was used above the pillar, while the HL1100B was used throughout the rest of the structure. Fig. 13 shows the detailed geometry of the solution above the pillar.

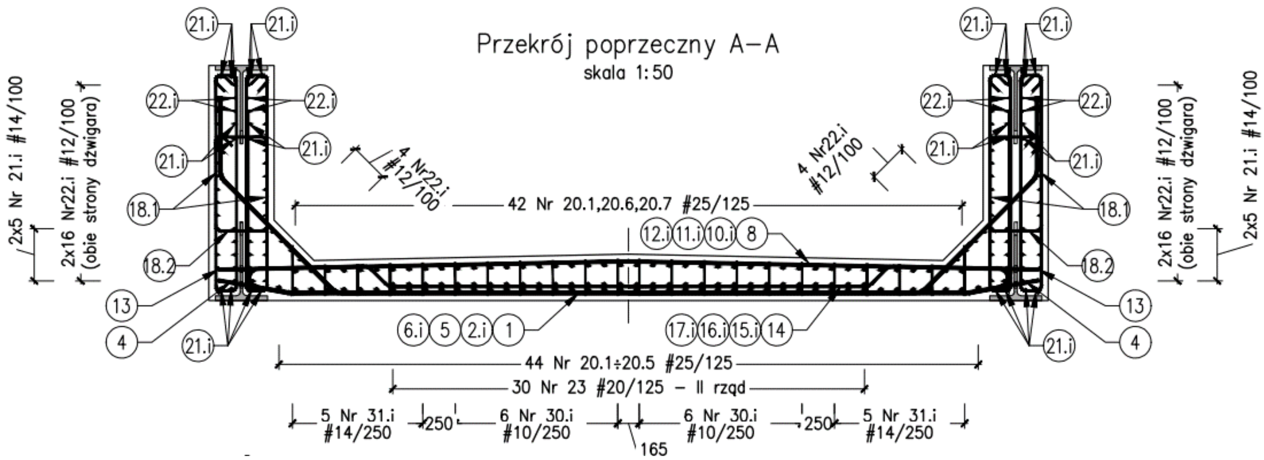

Figure 6: Reinforcing bars in cross section [57].

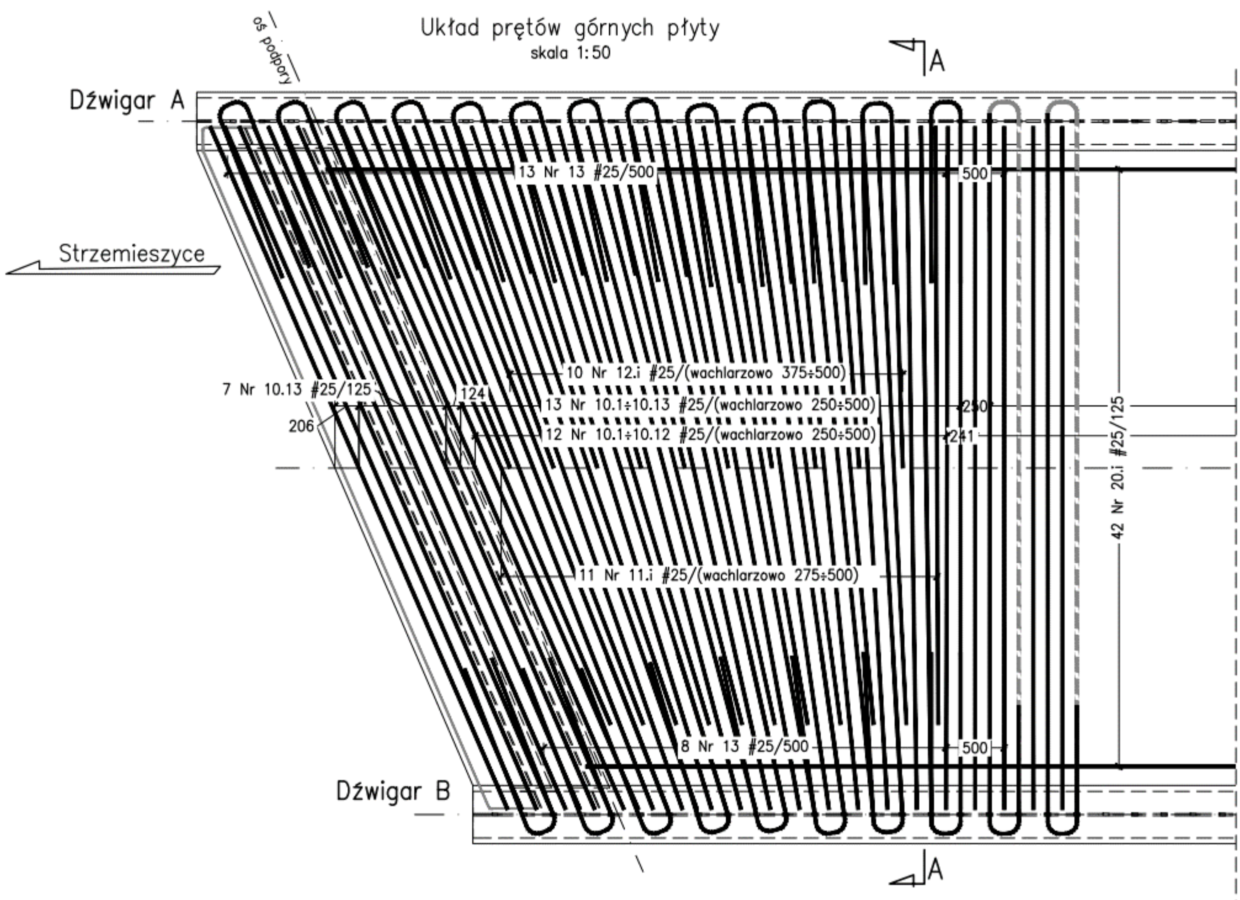

Figure 7: Upper reinforcement in slab [57].

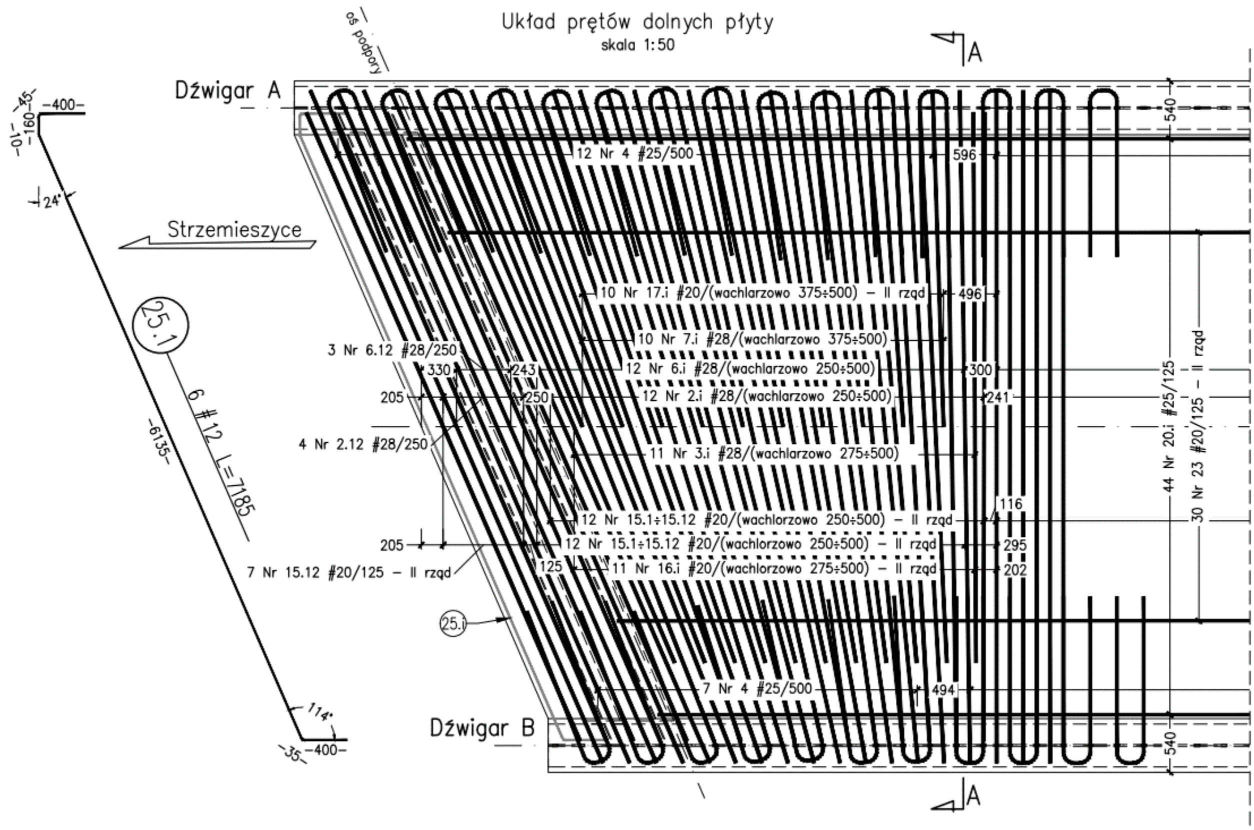

Figure 8: Bottom reinforcement in slab [57].

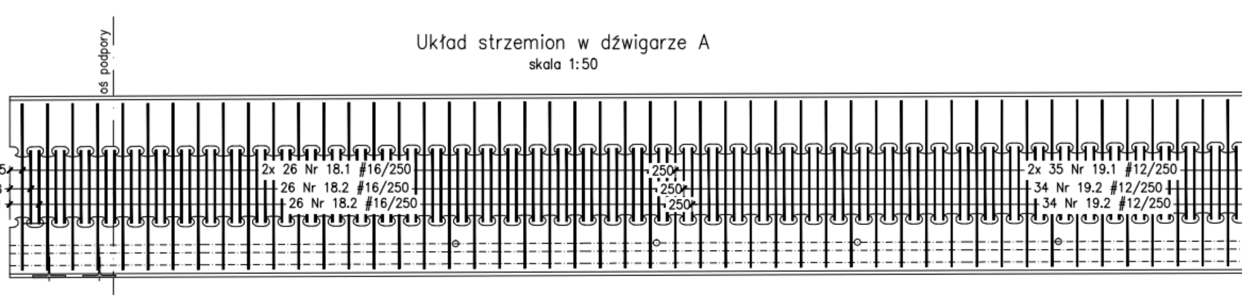

Figure 9: Shear reinforcement of hybrid girders [57].

The hybrid elements in the transverse direction have been used only at the ends of the deck (Fig. 12) where the normal forces in the deck and the stresses in the lower flange of the main girders are low and the deck effort increases significantly due to bending in the transverse direction. Due to the relatively small thickness of the deck slab, a large amount of transverse reinforcement was required (Fig. 6–8), which was shaped to transfer shear in the longitudinal direction from the deck slab to the main girders (Fig. 6). In order to ensure that the lower transverse reinforcement is well anchored in the beams, this reinforcement was led through the holes made in the web of the lower T-sections (Fig. 14). On the other hand, this provides a new and complex mechanism (Fig. 17) for transmitting the longitudinal shear from the concrete slab to the concrete web above the composite dowels' shear connection level. Due to the relatively small thickness of the deck slab, it was necessary to use a large amount of transverse reinforcement (Fig. 14, 15), which was shaped in such a way as to ensure the transfer of shear in the longitudinal direction from the deck slab to the main girders. Attention

Figure 10: Steel T-sections produced in Luxembourg (picture: ArcelorMittal).

Figure 11: Steel T-sections on site next to scaffolding prepared for *in situ* slab (picture by Nowak Mosty).

Figure 12: Steel structure after welding (picture by Nowak Mosty): main elements and transversal elements at the edge of slab.

is drawn to the use of holes (so called PBLs) in the lower T-sections (Fig. 13, 14). Their main purpose is to properly fix the plate in the girders due to its transverse directional action, but at the same time, a separate issue regarding shear flow appears. This is described in later paragraphs.

There is a high degree of concrete reinforcement ration in the tension zones: in the slab, in the spans of the viaduct, and in the upper part of the web girders above the pillar (Fig. 16).

The structure was executed using longitudinal launching technology, which shows an analogy to the technology used in the case of standard solutions in prestressed concrete bridges. These technological and engineering issues are described in detail in [10] and are not mentioned here anymore.

4 New issues and acquired knowledge

4.1 The influence of concrete cracking on the behavior of the structure

In order to understand the problems concerning the applied solution, one can refer to [48] and use the analogy to the tied-arch bridge using composite tie. Concrete cracking is one of the most complicated issues in modeling composite structures. For standard composite sections, the rules given in Eurocode 4 are based on an extensive and a complex theory supported by testing [41]. Contrary to the classic solution according to Eurocode 4, accepting an uncracked cross section is not always safe for longitudinal shear design [3]. In the case of the object in question, the problem of concrete cracking and the effect of tension-stiffening affect the deformation of the system, that is, in practice, vertical displacement under load (1) and the values of longitudinal shear in appropriate limit states (2). In the case of composite dowels, it is especially important in the fatigue limit state (FLS): for this the fatigue limit state actually governed the design of steel parts. Due to the lack of knowledge and experience, two extreme cases in design were conservatively assumed for longitudinal shear design, that is, a fully cracked section and uncracked (Fig. 5). Considering the CHC in the span under sagging bending, it can be seen that the distribution of tensile stresses transferred jointly by T-section and the cracked concrete slab depends strongly on the degree of cracking of this slab, and this influences distribution of longitudinal shear flows (web-T and web-slab). The ration

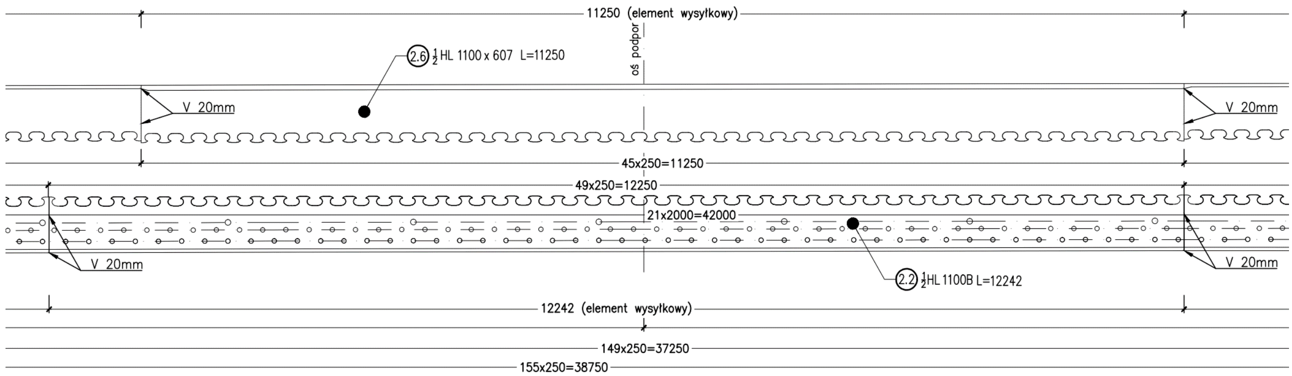

Figure 13: The geometry of the steel T-sections above the pillar [57].

Figure 14: T-sections and reinforcement of bottom part of the superstructure (main stirrups crossing concrete dowels are still missing at this stage, compare Fig. 9).

Figure 15: The general view of slab reinforcement.

of concrete versus steel area in the tensile part of the CHC is greater in the case of span sections than in the case of supports for the structure in question. Therefore, at this stage of the analysis, the focus was on the former, while in the case of the latter, adopting a full crack does not lead to a significant impact on the steel wear in the upper section. The determination of the degree of plate cracking in the FLS is a key issue and the main challenge standing in the way of optimizing the solution. The most reliable measure of the degree of concrete cracking and the first point of reference for further analyses are measurements under the real load of a real object, and so, it was done in this case and this is presented in the paper. Measurements of vertical displacements were made and the obtained results were compared with the predictions from various numerical models to draw first conclusions. In parallel, during the tests, a visual assessment of the degree of cracking was carried out, that is, crack observation (morphology and crack width). The conclusions regarding stiffness of a new type of structure considering tension-stiffening effect based on displacements measured on site have been drawn. The results are presented and discussed in the later paragraphs on the background of the theory for classic reinforced concrete.

4.2 The use of the parallel shear transfer mechanism

The structure uses a new solution that consists of the application of two types of shear connection (CDs and PBLs) in one steel web. This creates parallel “statically indeterminate” shear flows (Fig. 17, right); hence, it is not known what part of the shear flow is transferred to the steel bottom flange via CDs and what part of the shear flow is transferred via PBLs. Fig. 17 explains this task.

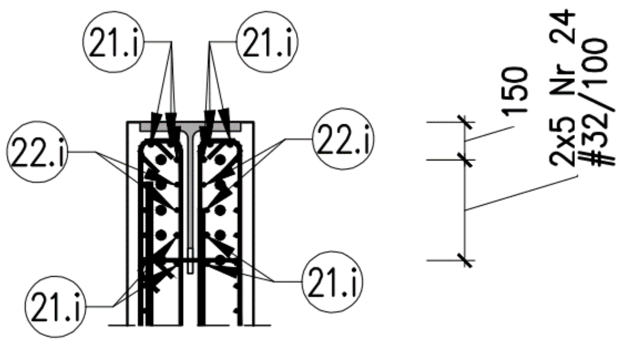

Figure 16: Reinforcing bars in the upper part of the web girders above the pillar [57].

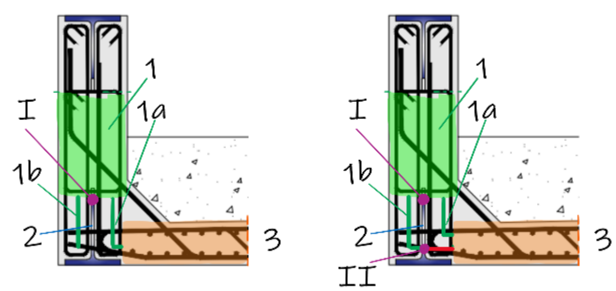

Figure 17: Possible longitudinal shear flow transfers: only via CDs (I) on the left and via CD (I) plus PBLs (II) on the right.

The point of reference here can be filler beam railway bridges, in which holes are used in the lower part of the steel beams to guide the transverse reinforcing bars. For the design of the bridge, the following shear transfer was conservatively assumed (Fig. 17, left): the shear flow from the concrete web (1) passes to the T-section via the CDs (I) and to the plate (3) via the inner part of the web below the CDs' level (1b). The influence of outer web (1a) and PBLs (II) is negligible. In reality (Fig. 17, right), it is difficult to determine what portions of the shear flow are transmitted by the elements 2, 1a, and 1b; it is noted that the shear flow separation depends not only on the shear capacity of elements (webs 2, 1a, and 1b and shear connections I and II), but also on their stiffness, so the model depends on the considered limit state (ultimate limit state ULS, serviceability limit state SLS, fatigue limit state FLS). As a consequence, it is difficult to determine what part of the shear flow is transferred by the PBLs (II), therefore this shear was conservatively omitted in the design analysis for longitudinal shear.

The problem of concrete cracking in the bottom plate described in section 4.1 affects the division of the shear flows (an uncracked plate takes up less force than a cracked one). Bearing in mind the authors' conclusion resulting from the design of the bridge that the assumption of steel sections is essentially determined by the fatigue conditions (FLS), it becomes clear that the stiffness of the cracked plate (adequate to the FLS conditions) and the division of the shear flows are inextricably linked problems. To sum up, Fig. 22 is shown to open a discussion. It is justified to assume that the big part of shear flow goes through PBLs and it would significantly influence the design of shear connection in the FLS (decisive criterion). The use of the parallel shear transfer mechanism (CDs + PBs) in the hybrid cross section is a new issue; such a solution was used in the construction for the first time.

4.3 Application of the hybrid cross section concept for the first time in structural analysis

The presented structure is the first engineering structure which used the division of the shear force into the part carried by the steel T-sections and the part carried by the concrete web according to the lever arm concept proposed in [3]; such an approach will be implemented in the CEN-TS design regulations that are just being developed [44]. It was presented at the SC4 meeting in December 2021 [45]. The theory of hybrid section concept is presented in [43]. (Earlier, an intuitive and engineering version of the method proposed in [3] and sanctioned in [43] was used on the Wierna Rzeka [6] bridge; it is presented in [43].) It is shown below how the concept to be used for simple hybrid cross section [44] has been applied to the combined (double composite) hybrid cross section. According to [15], the reduced effective depth of a cross section d_v (according to Eurocode 2) should be used for calculation of the inner lever arm of internal forces to be used for the determination of required reinforcement and design of the diagonal strut. In practice, in the case of the considered structure, it is about determining the dimensions of the stirrups A_{s1} (Fig. 18) and the application of the concept according to [15] would lead to an irrational and greatly overestimated reinforcement A_{s1}. Moreover, there is no indication as to whether the reinforcement A_{s2} should be taken into account when transferring the transverse force and for the vertical splitting at the same time. Here, the approach was used (formally for the first time) where the lever arm Z_H was used, which was calculated using an effective depth of the hybrid cross section d_H taken as the distance from the top surface of the upper surface of the girder to the centroid of the tensile part of the cross section made of structural steel. As the bridge is conservatively designed in the elastic domain,

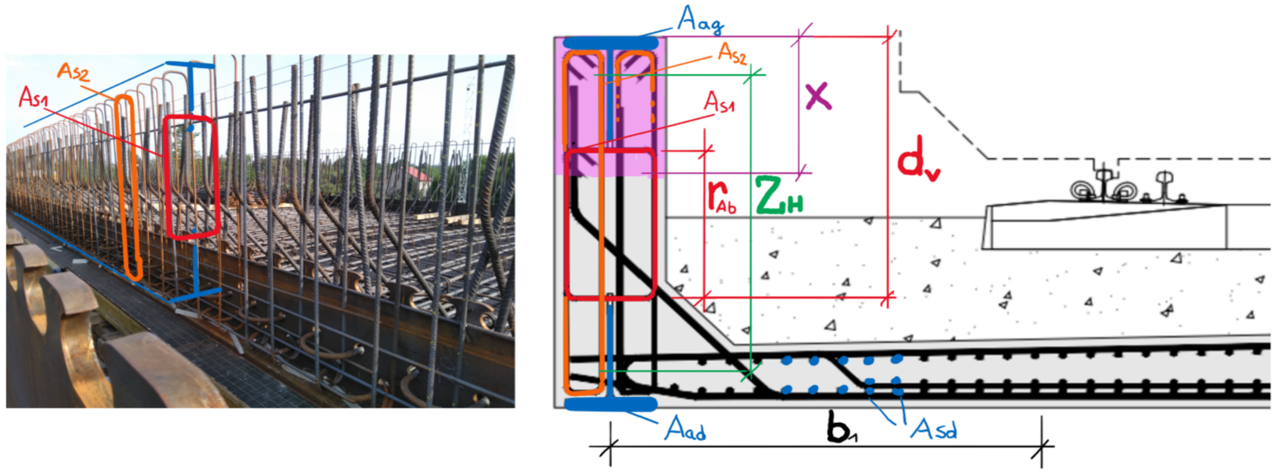

Figure 18: Implementation of the hybrid section concept (double composite) in the design of the bridge.

the lever arm can be determined for such a case from the formula $Z_H = Z = I/S$ [4]. The lever arm of forces Z_H was used to determine the amount of reinforcement needed to transfer the transverse force in the girder and to determine the resistance of the compressed struts in concrete web (the entire procedure is as for a concrete cross section in accordance with Eurocode 2 [43]). The reinforcement A_{s2} was taken into account for transferring the transverse force and for the vertical splitting at the same time. Thus, the total reinforcement assumed to transfer the vertical shear force in one girder is the sum of A_{s1} and A_{s2} (based on the experience from the bridge in Elbląg [1,37]). When determining the cross-sectional characteristics, it was necessary to take the effective width (see b_1 in Fig. 18) of the bottom slab and the premises here were followed as for the standard for concrete structures, that is, Eurocode 2 [21].

Both cracked and uncracked sections were analyzed; x in Fig. 18 is height of uncracked concrete zone for sagging bending (in such a case, reinforcement A_{sd} is considered at b_1 width). As can be seen, the issue of concrete cracking is of key importance in the case of the considered structure, not only for the transmission of the bending moment, but also for the transverse shear. The use of the hybrid cross section concept includes the assumption that the transverse force is divided into parts down: in the case of the design of the described structure, part of the transverse force is transmitted through the upper tee, some through the section of the concrete web connecting the steel T-sections, some through the lower T-section, and some through the inner bottom part of the concrete web (Fig. 17, left). Designing CDs with regard to steel failure (plastic design, at ULS [25,26,44]) provides at the same time a condition for the transfer of a part of the shear force through the webs of the steel T-sections, and the reinforcement of the main part of concrete web (Fig. 17, position 1) has been checked at the level of the hybrid cross section. This is the idea of designing for vertical shear force according to the hybrid cross section concept [43]. Selected design issues with regard to the bending moment and the issues of detailed modeling of the structure using finite element method (FEM) are presented in the subsequent paragraphs. The key issue is how to model the type of structure presented in this article. The problem of modeling (Fig. 19) is presented in [42] and this article shows how modeling with various techniques influences the results of the calculations which were compared to the values of displacements measured during real studies of the object on a natural scale (aim 4). In addition to these four main issues, other issues were considered, which are presented later in the text (they are cognitively important and often complicated, but, according to the authors, are of a lesser scientific importance). At the time of writing the article for printing, the draft of the guidelines introducing the design of hybrid cross sections [31] is basically finished.

4.4 Chosen problems in modeling the bridge structure in Dąbrowa

The issues related to the design and construction of the bridge were first presented at the Wrocławskie Dni Mostowe (WDM) conference [10]. Attention is drawn to the fact that the transverse T-sections used in Spergau were abandoned in order to eliminate the problem of tension acting composite dowels in transversal element [20]. As a result, member characteristics (like the moment of inertia) can be assigned globally to the entire bridge

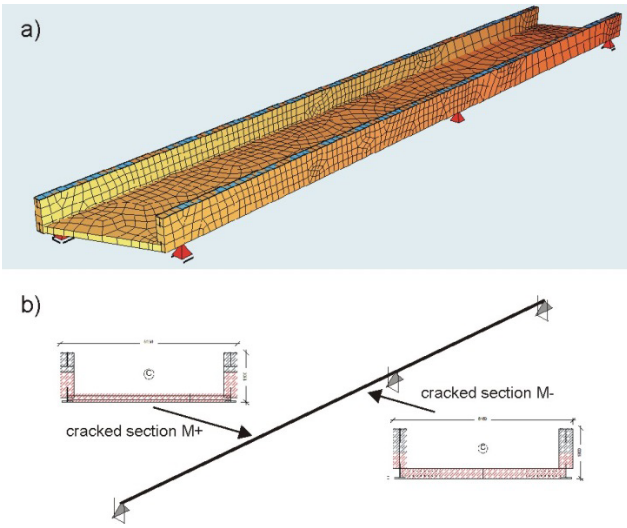

Figure 19: FEM models of the bridge structure made by external consulting [10]: a) mixed shell + beam model with shell elements standing for concrete parts and beam elements standing for structural steel parts; b) beam model using cracked steel–concrete sections.

cross section. Thus, a structure was obtained that can be modeled globally at the element level as a prismatic bar using concrete channel section with reinforced steel parts at the corners. In such a cross section, the effects of both free and restrained torsion play an important role. At the same time, it was necessary to determine whether the global analysis is performed on a cracked model (as for composite structures) or on a uncracked model (as for concrete structures).

The analyzed case is a new type of cross section and this question had to be answered. In the case of a beam model, it was necessary to determine the concept according to which the effective width is assumed [21]; here, it was done according to Eurocode 2 (ad hoc). A different concept is valid for composite structures and a different one for concrete structures, and the convergence of these concepts for the new version of Eurocode 4 was discussed in [21] and this is a complex problem. These two issues already show how important it is from the designer's point of view to qualify a new type of structure: is it a "rather" concrete or "rather" composite structure? As presented already, a key issue is the influence of concrete cracking and the tension-stiffening effect on the longitudinal shear forces in the shear connection when designing this connection on fatigue. Especially in the case of a field section, it is an important issue, where the reinforced bottom plate is an important element of the composite cross section (as opposed to the solution according to [6, 7]). In order to answer the question about modeling issues, a research project was conducted in parallel with the design work, under which a number of analyzes of various models were carried out. Various concepts of shell and beam models, cracked and uncracked, were analyzed and then, after gathering construction experience, mainly concerning the qualitative and quantitative issues of concrete cracking (shrinkage cracks of the new element and cracks resulting from creep effect), models which ensure the best representation of realistic superstructure issues were chosen. The numerical models done by external consulting (next to the models by design office) are shown in [10], and, here, probably, the most cognitively important part of the research work is presented, that is, the results of measurements under real load and their comparison to FEM models are done by the design office.

5 Measurements under static and dynamic load

The most important measurements were made on site, that is, the deflection of the structure under real load and concrete cracking observation. After static loading, the behavior of the structure under dynamic loading was measured.

5.1 Test load parameters

In order to verify design assumptions, regarding the response of the superstructure under static and dynamic loads, test load procedure was performed. During static test load, locomotive SM31 weighing 116.6 t (1143.46 kN) was used. A dynamic test load was carried using locomotive SM31 and ET22 (1177.20 kN) also. (Fig. 20). In a nonlinear analysis, apart from locomotive weight, self-weight of the superstructure and the equipment load were applied to the finite element models. Specific gravity of the superstructure-reinforced concrete (C40/50) was set in programs to 26 kN/m^3. It was assumed that a deck is loaded with weight of a layer of rumble 0.63 m thick, layer of isolation 0.02 m thick, and track weight. Weight of sidewalks of both sides was also applied to main girders as an additional linear load. A sum of the equipment load applied to the whole span is equal 78.06 kN/m (68.67 + 1.53 + 6.00 + 1.86).

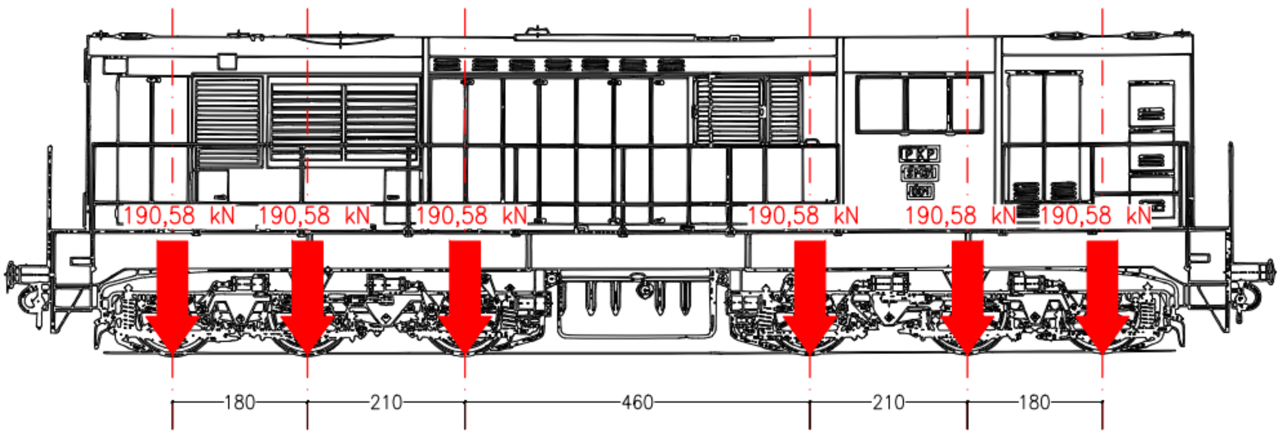

Figure 20: Scheme of the locomotive SM31.

Figure 21: Optical fibers (light wires) on concrete and steel.

5.2 Measurements gauges

Girders' deflection measurements were made using dial gauges, inductive sensors, and precise leveling. During static test load of the structure, strains in steel and concrete were also measured with the use of optical fibers. One optical fiber was attached to the bottom surface of steel flange of T-girder. The second one was attached to the side surface of concrete web of the main girder at the distance circa 5 cm from the bottom surface of the web (Fig. 21). In order to check the dynamic response of the superstructure, two accelerometers were used. Their job was to measure acceleration of two points on the main girders in the middle of the span during dynamic test load.

5.3 Finite element models

In order to analyze and check test load measurements, several finite element models were created. For this purpose, two programs were used: Midas Civil (design office during design process) and SOFiSTiK (external consulting and independent own analysis of B. Bartoszek for purposes of PhD dissertation). Structural modeling problems are presented in [10] where models by external consultancy check are presented. For the purposes of this study, additional models (Fig. 22: S00–S70, M40 UCR/CR) were made. Two models were prepared in Midas Civil for linear analysis of the superstructure in uncracked (M40 UCR) and cracked (M40 CR) conditions. In these models, only T-shaped steel sections are represented by beams. The deck and concrete webs are designed as shell elements. Eight different additional models were prepared in SOFiSTiK. In the first one (S00), the whole span is represented by one beam element with tapered cross section. In the second one (S10), the main girders and the deck are represented by a grid of beams. The whole span is divided into five elements in transversal direction (two girders and three longitudinal deck beams). Because of a vertical offset between the longitudinal deck

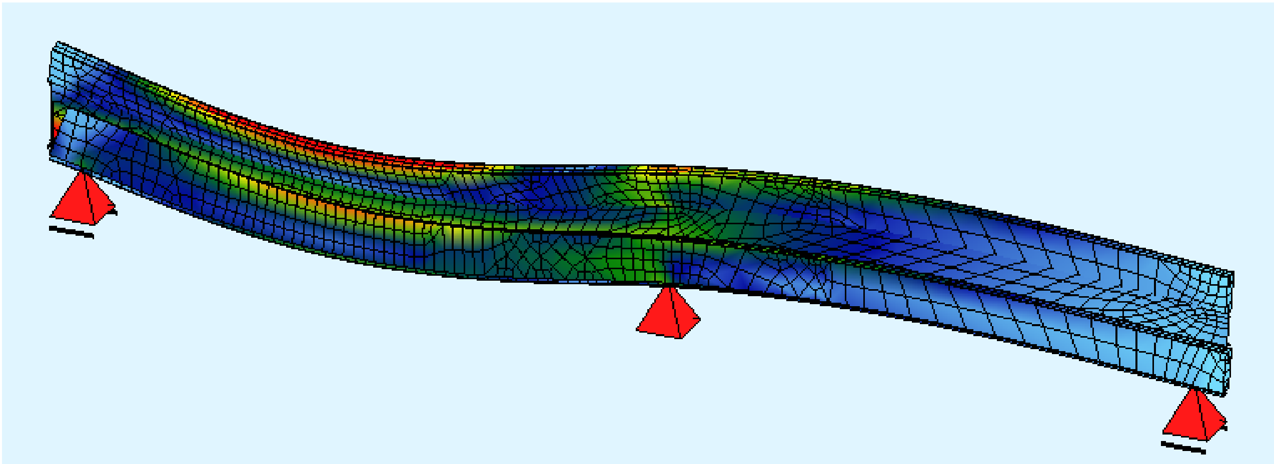

Figure 22: Exemplary deformation (linear analysis) of the finite element model from Fig. 19 under load 80 kN/m on the left span, applied as uniformly distributed to the slab in kPa [10].

bars and the main girders, rigid connections between bars were created. In the third model (S20), the deck was modeled by shell elements and main girders containing steel T-sections and concrete web by beams connected with the deck by rigid links. The fourth model (S30) also used concrete deck modeled as a shell element connected with the main girders introduced as bars. In this case, the T-sections and concrete web were divided into three parts connected via rigid links. In the fifth model (S40), all elements, which means concrete deck and webs, apart from steel T-beams, are implemented as shell elements. Steel beams are connected via rigid links with the top and bottom edges of concrete webs. The next model (S50) uses the same elements, but steel sections are connected with concrete webs at the real level of composite dowels in the middle of the concrete web. In comparison to that, in the sixth model (S60), additional division of concrete webs is implemented on a transversal direction. It means that concrete webs are split into two on both sides of steel T-sections (see Fig. 17, left). This allows to control lateral shear distribution and use or not bars going through steel webs as a composite. The last model (S70) uses beam elements only as the representation of steel flanges of T-sections. All the other elements, concrete deck, steel, and concrete webs, are modeled as shell. In all cases, linear and nonlinear analyses were conducted to get deflection, internal forces, and stress in superstructure in uncracked and cracked conditions. Load application was adjusted individually for all of the models.

5.4 Results of static test

As part of the static test load, one scheme of locomotive load was considered. A location of the locomotive was set to acquire maximum deflection value in the first span, whose length is equal 21.30 m. According to this, many locations along the span were analyzed using finite element models. Span deflection was measured in four points in the middle of each span on the bottom surface of steel T-girders. The location of all measure points is presented in Fig. 23.

Before static load test, the locomotive was set away from the flyover and default values of measurements were set. Next, locomotive SM31 entered the flyover and was set in designated place, which generates the biggest deflection values in span length 21.30 m. Static test load lasted 60 min. Deflection values in the middle of spans were measured every 15 min. Strain values in optical fibers were measured 12 times every 5 min. Acquired measured values were compared to the results of analysis of cracked and uncracked finite element models. Linear results for all models were highly convergent, while differences in nonlinear results were noticeable. Models S00–S30 do not allow to take into account realistic crack zones' range, due to shear transfer in cross section, because steel and concrete are not defined separately. This means that nonlinear results from these models ensure lower accuracy than the models S40–S70 obviously. Results from model S60 and S70 are almost the same because the only difference between them concerns the method of modeling the steel member. The above comparison of deflection values is presented in the graphs for Midas Civil models M40 UCR/CR (Fig. 24-25) and SOFiSTiK models

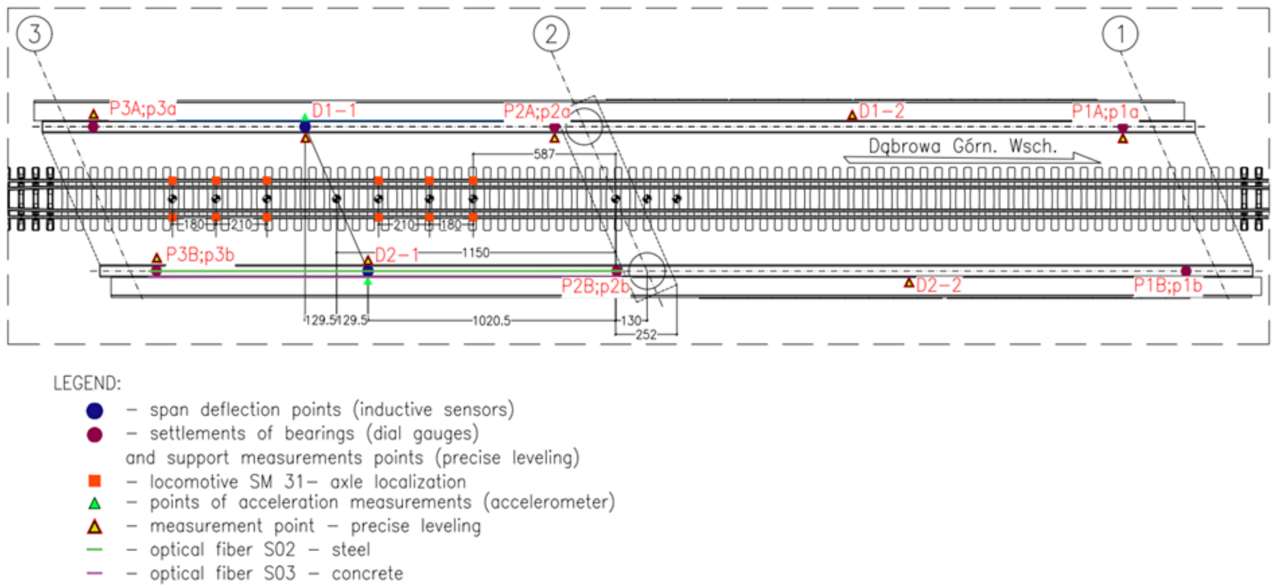

Figure 23: Load scheme and localization of measurements points.

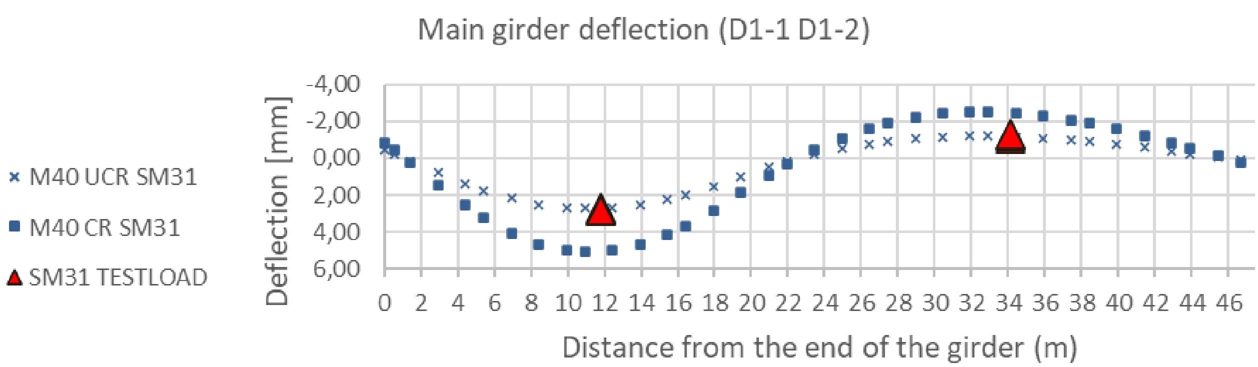

Figure 24: Main girder deflection of points no. D1-1 and D1-2 for models M40 CR and M40 UCR.

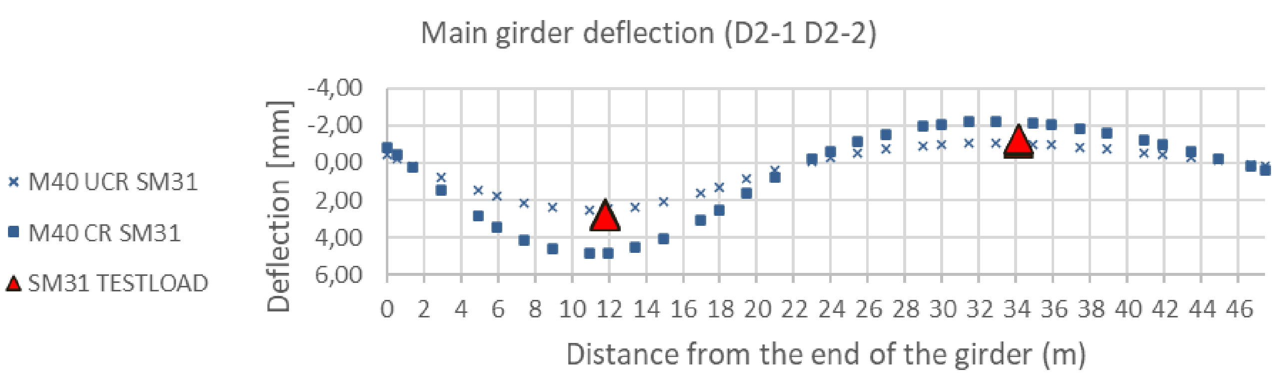

Figure 25: Main girder deflection of points no. D2-1 and D2-2 for models M40 CR and M40 UCR.

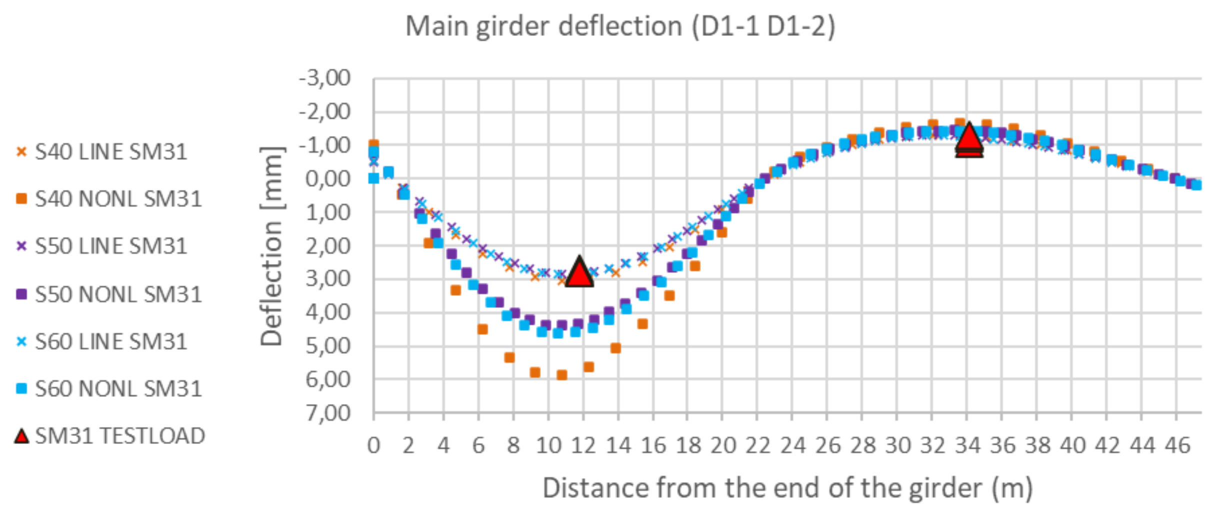

Figure 26: Main girder deflection of points no. D1-1 and D1-2 for models S40–S60 (LINE stands for linear, NONL stands for nonlinear material analysis).

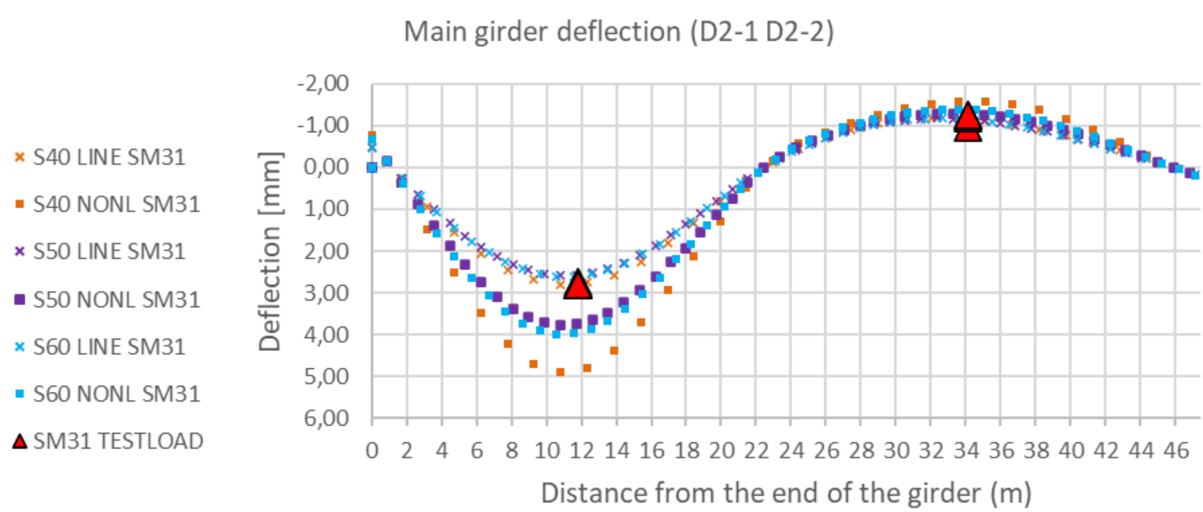

Figure 27: Main girder deflection of points no. D2-1 and D2-2 for models S40–S60.

S40–60 (Fig. 26-27) also. Results from models S00–S30 were also checked and discussed, but in order to ensure better clarity of charts and taking into consideration the complex character of a nonlinear behavior of the superstructure, they were not included in the charts.

The numerical analyses performed showed that the measured values of girder's deflections exceed just slightly the results obtained for uncracked models. Test load values are also much lower than the results obtained according to nonlinear analysis, representing the superstructure in a cracked condition. Maximum values of girder's deflection obtained from the test load measures are equal to 2.81 mm at point D1-1 and 2.82 mm at point D2-1. Deflection values in these points obtained from Midas uncracked model are equal to 2.72 mm (D1-1) and 2.49 mm (D2-1). In comparison, deflection of cracked span obtained from Midas model is equal to 5.07 mm (D1-1) and 4.81 mm (D2-1). It can be seen that the value of girder deflection obtained from nonlinear model S40 is bigger than that obtained from model S50 and even bigger than the value of deflection of model M40 CR. This difference is probably caused mainly by shear deformation of the concrete web (see element 1a in Fig. 17 left) because the primary difference between S40 and S50 FE models is localization of shear connection between the steel girder and the concrete web. Stiffness reduction in model M40 CR assumed axial stiffness reduction of concrete deck and web, but the shear deformation was omitted in this

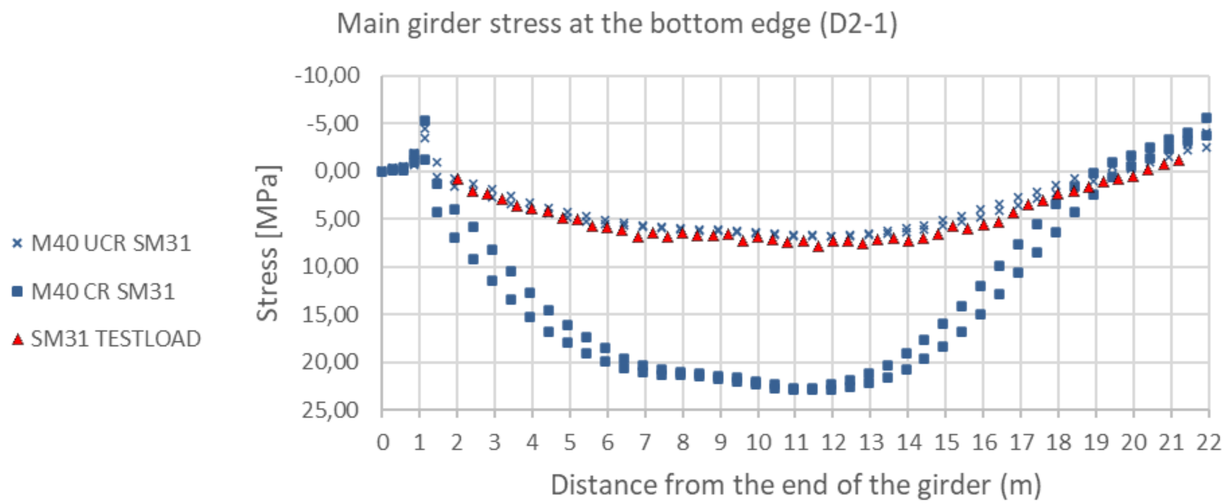

Figure 28: Stress in steel section for optical fibers and for the models M40 UCR/CR.

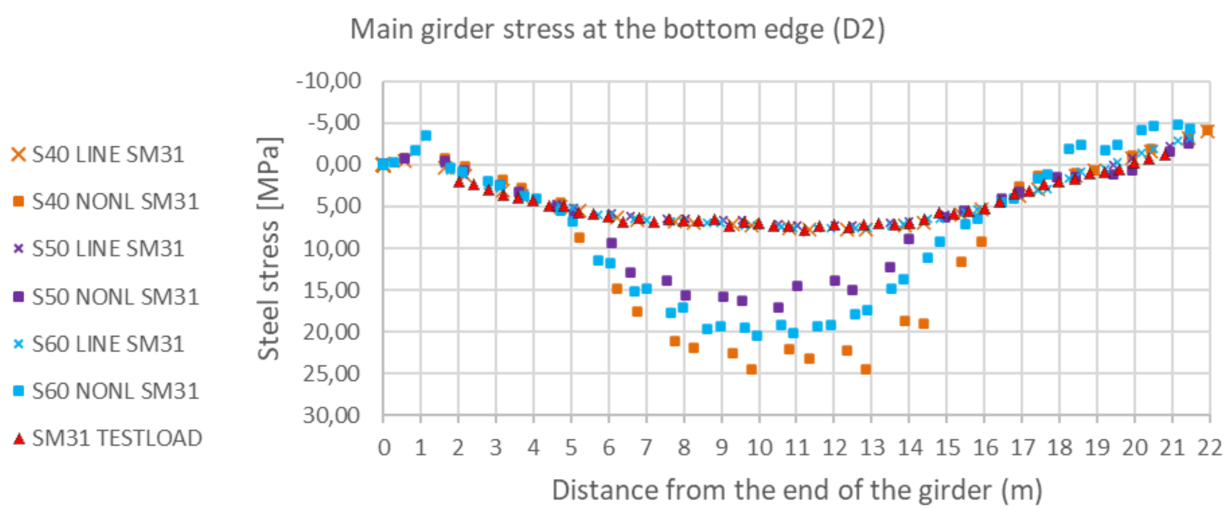

Figure 29: Stress in steel section for optical fibers and for the models S40–S60.

case, so deflection of nonlinear analysis may be more accurate. It is worth noting that deflection models like S40 should be always higher than the real structure because in the real structure, the axial force from steel sections is provided approximately 500 mm (in this particular case) from the concrete edge, and therefore, the real height of shear-deformed web is much lower than 1800 mm. Realistic localization of shear connection is implemented in models S50, S60, and S70. Application of a concrete web split in two pieces by steel web in models S60 and S70, provides realistic representation of shear transfer in the cross section. So, these models should ensure the highest accuracy of results. The next step is to analyze the strain and stress values measured by optic fibers. In this case, the test load values are very close to the results obtained from uncracked Midas models. Comparison of values from Midas models (design office models) and optic fibers results is presented in Fig. 28-29.

5.5 Results of dynamic test

In order to check the dynamic response of superstructure, dynamic test load was designed. Regarding this, the acceleration and deflection of points (D1-1 and D2-1) in the middle of the span of 21.30 m were measured during passage of the locomotive in both directions.

Table 1: Dynamic factors for SOFiSTiK models.

	Speed		Dynamic factors								Units
Model no.	-		S00	S10	S20	S30	S40	S50	S60	S70	-
Maintenance	-		Standard								-
Span length	L_Φ		26.85								m
Frequency	n_0		**4.83**	**4.45**	**4.68**	**4.70**	**4.57**	**4.83**	**4.72**	**4.71**	Hz
ϕ	10	2.78	**1.02**	**1.01**	**1.01**	**1.01**	**1.01**	**1.01**	**1.01**	**1.01**	-
	20	5.56	**1.03**	**1.03**	**1.03**	**1.03**	**1.03**	**1.03**	**1.03**	**1.03**	
	30	8.33	**1.04**	**1.04**	**1.04**	**1.04**	**1.04**	**1.04**	**1.04**	**1.04**	
	40	11.11	**1.06**	**1.06**	**1.06**	**1.06**	**1.06**	**1.06**	**1.06**	**1.06**	
	50	13.89	**1.07**	**1.07**	**1.07**	**1.07**	**1.07**	**1.07**	**1.07**	**1.07**	
	60	16.67	**1.09**	**1.09**	**1.09**	**1.09**	**1.09**	**1.09**	**1.09**	**1.09**	
	70	19.44	**1.10**	**1.11**	**1.10**	**1.10**	**1.11**	**1.10**	**1.10**	**1.10**	
	80	22.22	**1.12**	**1.12**	**1.12**	**1.12**	**1.12**	**1.12**	**1.12**	**1.12**	
	90	25.00	**1.13**	**1.14**	**1.13**	**1.13**	**1.14**	**1.13**	**1.13**	**1.13**	
	100	27.78	**1.15**	**1.15**	**1.15**	**1.15**	**1.15**	**1.15**	**1.15**	**1.15**	
	110	30.56	**1.16**	**1.17**	**1.16**	**1.16**	**1.16**	**1.16**	**1.16**	**1.16**	
	120	33.33	**1.17**	**1.18**	**1.18**	**1.18**	**1.18**	**1.17**	**1.18**	**1.18**	
	Speed		**Dynamic factors**								Units

Table 2: Dynamic factors for Midas models, PN-EN 1991-2, and test values.

	Speed	Dynamic factors						Units
Model no.	-		M40 UCR	M40 CR	6.4.5.2	D1	D2	-
Maintenance	-	Standard				-	-	-
Span length	L_Φ	26.85				-	-	m
Frequency	n_0		**4.90**	**3.76**	-	**5.43**		Hz
ϕ	10		**1.01**	**1.02**	**1.16**	**1.00**	**1.00**	-
	20		**1.03**	**1.03**	**1.16**	**1.01**	**1.00**	
	30		**1.04**	**1.05**	**1.16**	**1.00**	**1.00**	
	40		**1.06**	**1.06**	**1.16**	**1.02**	**1.02**	
	50		**1.07**	**1.08**	**1.16**	**1.01**	**1.01**	
	60		**1.09**	**1.10**	**1.16**	**1.03**	**1.03**	
	70		**1.10**	**1.12**	**1.16**	**1.02**	**1.02**	
	80		**1.12**	**1.13**	**1.16**	-	-	
	90		**1.13**	**1.15**	**1.16**	**1.06**	**1.03**	
	100		**1.14**	**1.17**	**1.16**	-	-	
	110		**1.16**	**1.19**	**1.16**	-	-	
	120		**1.17**	**1.21**	**1.16**	**1.08**	**1.06**	

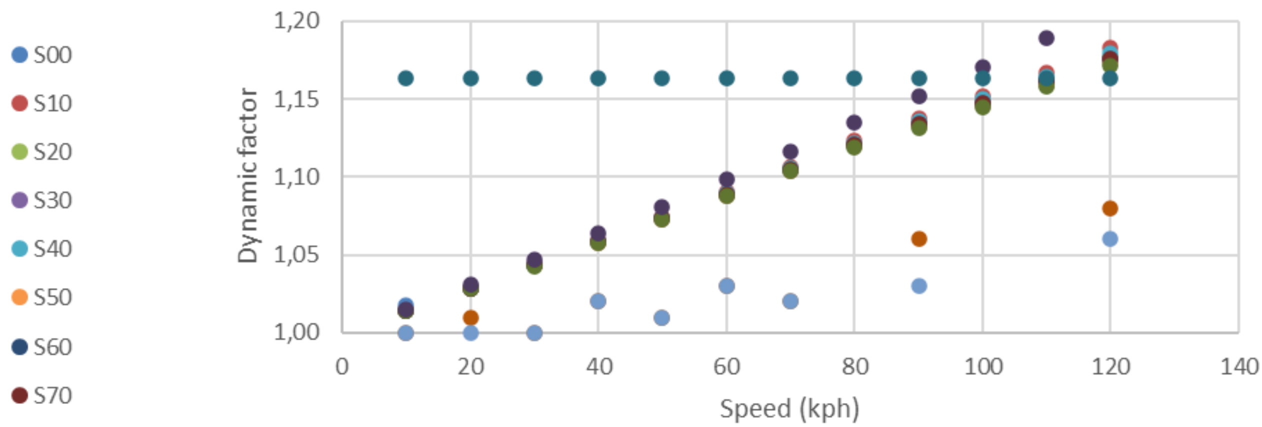

Figure 30: Comparison of dynamic factors for different FE models and the test load results.

In this case, two different locomotives were used. First, on 05.08.2019, locomotive SM31 drove over the flyover with speeds 10, 20, 40, and 70 kph. In the second test, on 05.11.2019, locomotive ET22 drove over the span with speeds 10, 30, 50, 70, 90, and 120 kph (design speed). The maximum value of acceleration reached during tests was equal to 0.39 m/s^2 for speed 120 kph. This number is much lower than the maximum required due to pt. A 2.4.4.3.2 PN-EN 1990 A1 (Table A2.9), which is 1.0 m/s^2. So, the superstructure ensures very good comfort for the passengers. Dynamic factors of span in design analysis were calculated due to pt. 6.4.5.2 PN-EN 1991-2 and Annex C of PN-EN 1991-2 based on results of the first natural bending frequency of span. Results of these analysis for Midas Civil and SOFiSTiK are presented in Tables 1 and 2. Real dynamic factors reached during dynamic test load were estimated based on static and measured deflection of girders, while the locomotives were going over the span. Comparison of analytically calculated and test values is presented in Fig. 30. Based on dynamic test load, the value of the first natural frequency was established at a level 5.43 Hz, which is within the range (3.36–8.09 Hz) defined according to pt. 6.4.4 PN-EN 1991-2.

This value is slightly lower than any frequency obtained from numerical analysis, where the biggest frequencies were reached for M40 UCR (4.90 Hz), S50, and S00 (4.83 Hz) models. This difference may be caused by smaller real values of dead load applied to the superstructure than it was assumed in numerical analyses. Additionally, stiffness of rumble, track, and friction between them and the superstructure were neglected in the analyzed FE models. In reality, these phenomena may have influenced the global dynamic response of the structure.

5.6 Optical fibers measurement results

Additional measurements using optical fibers were performed and comparison of steel and concrete bottom surfaces strains can be seen in Fig. 31. High picks of strains in optical fibers attached to concrete surface (S03) mean that discontinuity appears and should be interpreted as cracks in concrete. These results led authors to estimate crack localization on concrete webs. In Fig. 36, strains for steel bottom surface and concrete web are compared, which means that the mean values of strain in optical fibers are presented.

5.7 Conclusions from the first load test of the structure

It was initially explained why the stress level in the lower chord is small under the test load. It is generally difficult to achieve a high degree of effort under the actual service load of short span railway structures. The lower chord effort level (about 200 MPa in the span at the ultimate limit state) results not only from the designers' conservatism in their approach to this innovative structure: the bottom steel flange area is correlated with the thickness of the steel web (rolled HL sections) and the thickness of the steel web in railway bridges using composite dowels [6,9,12] usually results from the fatigue limit conditions (FLS) for steel dowels [56], and the same was true for the structure in question. Thus, assuming a cracked state in the dimensioning both in terms of bending and longitudinal shear resulted in oversizing of the structure due to bending in the span (relatively big steel flanges). In addition, a much lower operational load does not even significantly induce the concrete cracking in the bottom

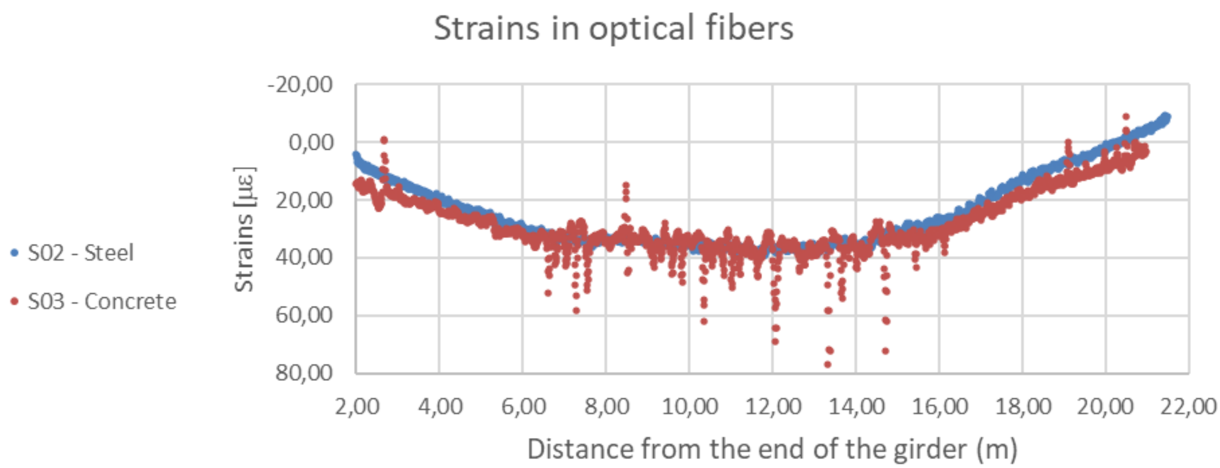

Figure 31: Strain in optical fibers along the span.

Figure 32: General view of the structure during test. L1 – light wire on steel flange, L2 – light wire on concrete web, E – measuring equipment under the bridge (displacements), C – car with computer for computing light wire recordings.

plate and results in a very low level of stress in the steel bottom chord (but also with low longitudinal shear). With the present state of knowledge [3, 43, 44], it simply could not be designed less conservatively and the test load results show a far-reaching conservatism, and this is the crux of the influence of concrete cracking on dimensioning that the authors wanted to show. In order to draw conclusions as to the behavior of an innovative structure, one should have a point of reference. In this respect, reference is made to the results obtained during measurements of standard bridge structures. On the other hand, the slight effort of the structure is justified. As test loads in Poland are performed as standard for new bridges, this valuable reference point was used. The reference point is the bridges made of reinforced concrete as well as those made of filler beam decks (in the case of steel structures, there is a high compliance of the measurement results with the theoretical models and standard composite solutions are not a good reference point). The point of reference is the conclusions and experiences of the person having big experience in first load tests in Poland [50, 51]. In the case of filler beam decks in railway bridges, the obtained deflections are smaller [51] or close to those obtained for the uncracked cross section [50]. The method of modeling elements other than the girders themselves plays an important role, and in the case of a relatively accurate model, values close to those calculated for the uncracked model are usually obtained [50]. Railway bridges with a reinforced concrete structure generally have a small span, up to 10 m, and are stressed during loading at the level of about 50% of the characteristic load according to [51]. Statistically speaking, the displacements obtained correspond to about 70% of the theoretical displacements obtained for the uncracked model. Thus, not only do they "work as if they are not cracked," but also the influence of equipment elements not included in the model is also visible (attention is paid to the small span). Reinforced concrete road structures are usually structures with a span of less than 20 m (prestressed concrete is usually used above). The level of their effort during the test loads is higher than the railway bridges, and usually about 75% of the characteristic load [51] is achieved. Usually, about 90%–100% of the displacement values calculated as for the uncracked model are obtained [51]. Since equipment elements (such as sidewalk slabs) are not taken into account in the calculations and cracks in concrete are observed, it can be concluded that these objects reach the level of effort corresponding to the beginning of the cracked phase, but, in general, it should be summarized that their stiffness corresponds to the uncracked phase (note that a similar result was obtained for the structure in question and its span exceeds that used in reinforced concrete railway bridges and corresponds to the upper limit for road structures). The deflections obtained in road bridges are essentially elastic. Against this background, a specific conclusion can be drawn: the behavior of the structure corresponds to that of reinforced concrete bridges; it works like almost uncracked, but due to the relatively low level of stress in the steel bottom flange of the structure, one should be careful with drawing far-reaching conclusions. The values of the measured deflections under static load are significantly lower than the analytically determined values for the cracked models and only slightly higher than for the uncracked models. This allows us to conclude that the structure exhibits greater stiffness than conservatively assumed (cracked) at the design stage. The results constitute an important premise to indicate the justification for modeling hybrid structures in an uncracked state for FLS. The value of the natural vibration frequency determined from the tests is higher than for uncracked models, which may be caused by the cooperation of the span with the railway track surface. The values of the dynamic coefficients determined on the basis of the tests are lower than the values determined analytically according to standard design procedures; thus, it is confirmed that the assumptions made in the design calculations concerning the dynamic coefficient allow for the safe design of this type of structure. The acceleration values are significantly lower than the maximum values allowed by [52], thanks to which the structure ensures adequate comfort for passengers. Summarizing the first load test, relatively small cracking of the concrete structure and relatively high stiffness of the innovative hybrid superstructure as well as very good dynamic properties were observed. The obtained unique results constitute the first and basic point of reference for further analyses.

6 Conclusions

The article presents a prototype steel–concrete bridge with the results of trial load tests. In the design of the structure, new approaches were used, the so-called concept of a hybrid cross section. The obtained results were interpreted against the background of theoretical analysis performed and the experience of the behavior of the existing standard bridge structures. The obtained results are to be the starting point for the development of methods of calculating this type of structure, with particular emphasis on the degree of cracking of the concrete part of the structure. In general, a very small crack width under service load ensures the durability of the solution, but in the case of the considered structure, it is more important in the context of the use of appropriate calculation models. The confirmed very limited cracking of the concrete superstructure (under service loads) is a very important information because 1) it influences the structure's stiffness, which is crucial from the point of view of the design of railway bridges and 2) it reduces the value of longitudinal shear forces from the fatigue load, which is the basic criterion for selecting T-sections from rolled sections (the steel web thickness is decisive). This would allow for the thesis that designing in a cracked stage for fatigue and for passenger comfort may be far too conservative and it is possible to take into account a significant share of (cracked) concrete.

Especially the first aspect is crucial – in the case of the designed bridge, the fatigue capacity of the steel part of the composite dowels' shear connection [56] was decisive for the selection of the steel shape and for the final steel consumption for this bridge. As the structure includes a rigid reinforcement (T-sections) connected by a shear connection using composite dowels with concrete, and classic reinforcement (bars), it should be considered whether the current modeling of the tension-stiffening effect (in reinforced concrete and composite girders) is applicable in the considered cross section. This will be the subject of further analyses, but the present results indicate that fatigue of composite dowels could be analyzed in an "almost" uncracked state for this kind of hybrid girders. It is to be compared to other types of hybrid railway bridges like "Simmerbach" bridge [12], "Wierna Rzeka" bridge, and "Tczew" bridge [6,9]; cracked conditions in hybrid beams are being currently widely investigated by Kożuch [47,54] and this bridge is to be one of the reference points in studies of this topic. The simulations carried out by the authors [55] have shown that in the case of using the available HL sections, the solution is rational for simply supported spans with a span of 20–30 m and for continuous spans up to about 40 m (the analysis was performed with a fixed construction height of 1.8 m, and the load capacity in the elastic area and the limit deflections were considered conservatively, as in the case of the bridge). It is noted that the conclusion that it is possible to model the structure as uncracked (or not fully cracked) for the needs of the FLS cannot be clearly drawn on the basis of the presented implementation and tests because a low level of stress was obtained in steel lower flange, and so, greater effort of the structure was required under the operational load. This can only be achieved in the big-scale laboratory test. Moreover, the deflections of the structure of the bridge in question will be checked again in the future a few years after the start of its operation; after this time, the influence of rheology on the pattern of cracks in concrete will be visible. It should be noted, however, that the behavior of the structure may change over time after applying a load greater than during testing. Summarizing the above, this paper is intended to be a starting point for demonstrating that it may be possible to calculate longitudinal shear in FLS (the fatigue of steel dowels) differently than in the fully cracked section. Similarly, it is supposed to be a point of discussion on how to perform a global analysis of hybrid systems (whether as uncracked in accordance with Eurocode 2 [33] or as cracked in accordance with Eurocode 4 [35]); this question was raised in [42]. Summarizing the whole, an analysis of the implementation of an innovative structure designed with the use of an innovative design approach [3, 43, 44] is presented and the obtained test load results under real service load are interpreted.

Acknowledgments: Financial support of ArcelorMittal (Luxembourg) is acknowledged. Commitment and cooperation with FASYS Mosty (Poland) and Nowak-Mosty (Poland) is acknowledged.

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