Stress State and Several Problems of Estimating the Actual Operation of Effective Bridge Superstructures

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Abstract. The paper presents the results of investigating the under-load behaviour and some estimates of the mutual influence of the stress state of structural elements during the actual operation of effective bridge superstructures with orthotropic deck plates. A brief description of the design solution of orthotropic deck plates is presented. It is stated that, from a structural point of view, significant saving of steel in the case of orthotropic deck plates is due to the fact that the longitudinal stiffeners and decking of such plate are included in joint work with the main girders and, thus, are included in the upper flange cross section. Therefore, the normal stresses in the plate near the webs are always greater than at any distance from them. Estimating this irregularity is very important for determining the actual operating conditions of the orthotropic deck as a whole and is carried out in each case by performing calculations of the superstructure. The results of calculating the real bridge deck of the carriageway at the Dnieper Hydroelectric Power Station dam in Zaporozhye are described. Relevant recommendations are provided.

Keywords: Bridge structure · Superstructure · Orthotropic deck plate · Computational model · Reduction factor · Stress

1 Introduction

Recently, efficient orthotropic steel plate systems (by "orthotropic" we mean deck plates which consist of longitudinal stiffeners and crossbeams welded to the decking plate, intersecting with each other; since the rigidity of such a plate is different in perpendicular directions, it is called orthotropic, or orthogonally anisotropic one) have become widespread in many bridge structures as one of the main load-bearing structural elements of the carriageway (Fig. 1). The almost widespread use of this design solution is due not only to its exceptional technical characteristics [24–26, 31, 33], but also to many other important advantages over commonly used solutions. First, orthotropic deck plates can increase the bearing capacity and reliability of the carriageway when resisting transport

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(static and dynamic) loads. Secondly, it can increase rigidity of thin plated structural elements under compression, and finally thirdly, probably the most important thing, orthotropic deck plates can achieve significant saving of steel and reduce construction time, what, of course, leads to lower costs of bridge structures.

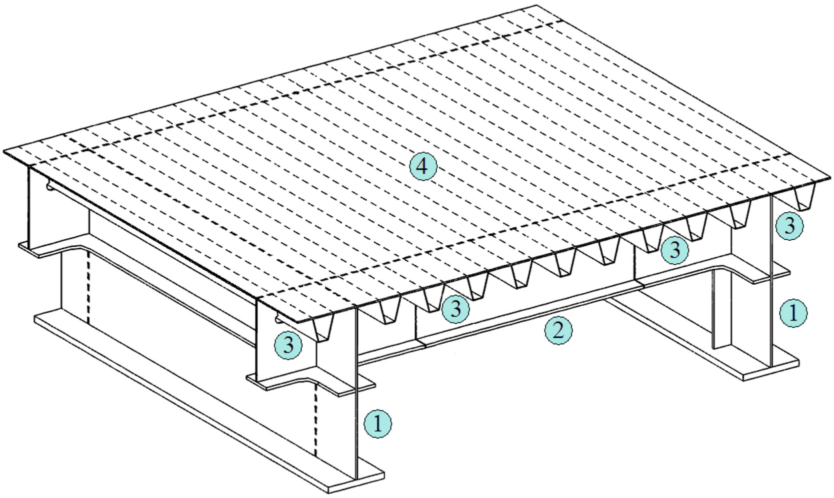

Fig. 1. Standard structural element of the orthotropic steel plate system. General view: 1 – main girder; 2 – crossbeam; 3 – longitudinal stiffener; 4 – steel plate.

Let's pay attention to the fact that initially the cases of using orthotropic steel deck plates in bridge construction were sporadic, but further they are becoming more common. In confirmation of the above, we note that the design of the first bridge using orthotropic deck plates was developed by German engineers in the 1930s, and the first such bridge was built by them in 1936 [26]. German engineers are also ascribed creating the very term "orthotropic" and registering the corresponding patent in 1948 [30].

It is also obvious that similar work was simultaneously carried out in other countries. For example, in the United States in the State of California, across the San Francisco Bay between the cities of San Francisco and Oakland, the Bay Bridge was built in 1936, which is an example of the first suspension bridges using orthotropic steel deck plates. By the way, American engineers introduced the term "battle deck", which differs from the European one, to name orthotropic deck plates, apparently by analogy with the name of an armed capital ship (in a more common abbreviated version – a battleship) that has long been used in shipbuilding – "battleship" [26].

Orthotropic deck plates were especially widely used in bridge construction in Germany after World War II due to the fact that almost all bridges in the country at that time were destroyed and the problems of saving steel became very urgent. In addition, the expanded introduction of orthotropic deck plates was additionally facilitated by two significant circumstances: the improvement of welding technology and the development of the calculation theory [4, 26, 28, 30].

Subsequently, in many countries around the world, a large number (hundreds and thousands) of various bridge structures were built using orthotropic steel deck plates. Therefore, it makes no sense to dwell on the description of all these bridges in this paper due to their large number. However, we would like to mention only one of a lot of these wonderful structures of the world's transport infrastructure using orthotropic deck plates, which is rightfully among the most famous bridge structures. It is about the cable-stayed bridge (more precisely, the viaduct) of Millau, passing through the valley of the Tarn River near the city of Millau, located in the south of France in the Aveyron department (Fig. 2).

Fig. 2. Panoramic views of the Millau viaduct from the eastern (*a*), north-eastern (*b*), south-western (*c*) and south-eastern (*d*) sides.

The mentioned bridge attracts attention by the fact that at commissioning in 2004 it held three world records in bridge construction (for the height of reinforced concrete supports, for the height of reinforced concrete supports with a crowning metal pylon and for the carriageway height). However, in order to save space, we will not give more detailed information about this remarkable engineering structure, since it can be easily obtained using its official website capabilities Le Viaduc de Millau [27].

As follows from the above, the history of using orthotropic steel deck plates in bridge construction goes back almost a century. During this time, these structural elements of the bridge carriageway have found the widest application. Moreover, the latter was facilitated not only by their naturally inherent numerous positive properties (efficiency, reliability, increased bearing capacity, steel economy and much more), but also by constant improvement of both the theory and research methods, as well as engineering solutions.

If we talk about improving the theory and methods of studying orthotropic deck plates, it should be noted that at first, analytical methods for their calculation were actively developed. In 1947, German engineer W. Cornelius [14] proposed to apply the well-known theory of M.T. Huber for calculation of plates [20–22] with some clarification. Moreover, the refinement was based on the fact that the thickness of the plate was determined as a result of taking into account the metal plate and additional “smearing” on it of the longitudinal stiffeners and crossbeams, proceeding from equivalent cylindrical rigidity of the orthotropic deck plate and a smooth thin plate. The theory of calculating orthotropic deck plates was further developed in 1957, when engineers V. Pelikan and M. Esslinger [29] proposed to consider longitudinal stiffeners as continuous beams on elastically subsiding supports, which made it possible to somewhat simplify the calculated ratios due to neglected tensional rigidity of open stiffeners.

In the early 1960s, G. Gomberg and K. Trenks [19] recommended using the girder grillage method for calculating orthotropic deck plates, based on the orthogonalization of unknown quantities with presentation of external load and internal forces in the form of group factors, the change of which is described by trigonometric functions. As for the girder grillage design model, it was adopted in the form of a freely supported contour grillage made as a system of crossbeams resting on infinitely large number of elastic yielding rotating supports, or, in other words, a decking plate with longitudinal stiffeners.

In the 1990s, the era for applying computer technology and numerical calculation methods began, which were implemented in high-performance software and computing systems, as a rule, on the basis of the finite element method. The use of these complexes removed almost all the difficulties inherent for analytical methods in terms of setting, detailing and dimensioning the research problem, the introduction of assumptions and simplifications in creating design models of structures, adequate presentation of existing static and dynamic loads, and much more. Moreover, it became possible to obtain reliable results not only by creating design models that fully reflect the design features of real structures, but also by performing calculations (geometrically and physically) both in a linear and nonlinear setting.

The latter made it possible to assess the stress–strain state (SSS) of orthotropic steel deck plates of complex configuration with almost maximum detailing, taking into account the features of the design solution, various formulations of the integral mode for material state and random external loads and influences. And as a confirmation of what has been said, let us mention only a few of the most indicative publications from a very large number of available examples. In particular, the papers [15, 16] describe the design model of an orthotropic deck with detailed modeling of cuts in the webs of crossbeams for passing longitudinal stiffeners, the results of determining the stresses in the beam webs at the cuts and changes in their values depending on the fatigue strength are given, which allowed to reveal the reasons for growth of fatigue cracks.

Moreover, a visual method for calculating stresses in the webs at the cuts is proposed, which not only allows us to understand the origin of all stress components, but also simplifies the comparison of results for cases of cuts of various geometry. By the way, the authors of the paper [32] draw attention to the rather thorough development of structural models of all elements of orthotropic deck plates in the analysis of secondary bending stresses in webs of enclosed trapezoidal stiffeners. The principles and features of the design model are described, and the calculation results are illustrated by a numerical example. It is noted that secondary stresses are caused by the transverse expansion of the lower parts of the stiffeners, and in combination with primary bending stresses, they can contribute to the growth of fatigue cracks. Recommendations are given for decreasing the values of these stresses.

The authors of a very interesting paper [12] proposed an idealized design model of an orthotropic deck plate, the equivalent physical and mechanical properties of which in perpendicular directions are established by taking into account the stiffeners by analogy with reinforcing fiber beams in composite materials. The study results are presented concerning the joint behavior of stiffeners, beams and the decking plate under transverse load, based on which the authors formulated a fairly simple analytical and engineering approach based on Ritz method for determining the deformations of an orthotropic deck plate.

In the article of the journal [13], in order to improve the design of bridge superstructures, several options for calculating the effective width of beam flanges b_{eff} in reinforced concrete single-span and multi-span bridges were considered, depending on three main parameters: the distance between the beams, the span length and the inclination angle, as well as more universal criteria for determining the effective width of the flanges in comparison with those used in engineering practice. The studies were carried out using the finite element method, the results of which were not only compared, but also confirmed by experimental data on test samples in scale 1/4 and 1/2, and the criteria were extended to the areas of positive and negative bending moments.

In terms of improving the engineering and technical solutions for orthotropic steel deck plates, we only point out that this improvement, in fact, has always been the dominant component of their use. This is evidenced by the fact that even the first experience of using orthotropic deck plates revealed two disadvantages in them: low fatigue resistance (cracking at the intersection of longitudinal stiffeners and crossbeams) and destruction of the asphalt concrete pavement on the deck plate. Further, a lot of studies were devoted to this issue, and at present a number of recommendations have been developed that can significantly improve the situation. In a similar way, the parameters of longitudinal stiffeners were improved, such as: construction (open, closed), geometry (flat, corner, tee, strip etc.), configuration (trapezoidal, V-shaped, circular), with or without diaphragms, and much more. The results of all the above long-term and numerous studies have been implemented in the norms and guidelines in many countries specifying the design of orthotropic steel deck plates for bridge superstructures. Among them are the most well-known norms and manuals of USA [8–10, 18], Europe [17], United Kingdom [11], Japan [23], and Ukraine [1–3].

It should also be emphasized that these regulatory documents, in addition to the norms themselves, guidelines and manuals for design of orthotropic deck plates, contain extensive bibliographies and large-scale analyzes of the problem state, what, in the opinion of the authors of this paper, is very useful for acquainting the reader with the topics under consideration herein.

Thus, summarizing the above analysis of literature sources, we can conclude that over a relatively short historical period of using orthotropic steel deck plates in bridge construction, the theory and methods of their research, as well as improvement of structural and technical solutions, have got really impressive development.

However, both in the works discussed above and in other more numerous articles and monographs, due attention was not paid to the study of the operation of orthotropic steel deck plates under conditions of long-term (decades of years, and especially many decades) use under conditions of development and accumulation of various kinds of operational damage. Let's note that with regard to bridge structures, the nature and consequences of these damages are as follows.

First, most often they are caused by temperature actions, seismic and aggressive influences, collisions of vehicles and accidental impacts of extraneous objects, and secondly, this leads to distortion of structural elements, occurrence of fatigue cracks, destruction of protective paint coatings and subsequent corrosion of metal and decreasing the design thickness. Meanwhile, it seems quite obvious and absolutely indisputable that only a correct understanding of the actual under-load operation of orthotropic steel deck plates of bridge superstructures, when there are operational damages, provides the ability to maintain the required level of their strength and reliability as a result of timely implementation of the necessary repair measures. If, for one reason or another, the repair work cannot be completed on time, then only knowledge about the actual technical condition of orthotropic deck plates can help the matter, on the basis of which schemes for organizing temporary traffic on the bridge superstructure are to be developed in order to ensure its limited operation (as a result reducing the load on the bridge superstructure) until the completion of the repair work. However, it should be noted once again that this traffic schema is temporary one and is used exclusively to facilitate the operation of the bridge superstructure. And it is for this reason that it contains a set of interrelated proposals aimed both at direct reduction of the load on the bridge structure (especially from oversized, freight and public transport), and at redistribution of this load over the carriageway area (taking into account the places of damage) due to changes of the number and width of traffic lanes, as well as the traffic intensity and speed of vehicles.

Finally, we note that the study of the actual technical condition of bridge superstructures with the roadway orthotropic steel plates, the operation of which lasts a long time, is an important and at the same time complex engineering problem. The solution to this problem is of great importance and contributes to timely extension of operation of bridge superstructures and ensuring the traffic on them without restrictions.

As an example of solving the indicated problem, let us consider the existing driveway at the Dnieper Hydroelectric Power Station dam, which was built in 1977 with use of the driveway orthotropic steel deck plates and which is one component of the complex of bridge crossing structures over the hydraulic structures of the said HPS.

2 General Characteristics of the Bridge Superstructure of the Driveway at the Dnieper Hydroelectric Power Station Dam

It is known that the Dnieper Hydroelectric Power Station dam in the city of Zaporozhye was commissioned in 1932. In addition to actual generation of electricity, its distinctive feature is that it simultaneously served and now serves as an important traffic link between the right bank and left bank of the Dnieper river due to the arrangement of a two-lane highway crossing on its entire length (Fig. 3).

The specified highway crossing contains several structures (from left to right): the bridge across the fore chamber of 319.76 m length (section I–II); the driveway on the dam structures of 666.00 m length, which includes superstructures executed in 1932 and 1977 (section II–III, Fig. 4); 111.50 m long overpass connecting the dam with the left bank (section III–IV), 136.57 m long soil insert-section (section IV–V) and 352.00 m long overpass over gateways (section V–VI). Thus, the total length of the highway crossing is 1,585.82 m.

The driveway located on piers of the dam with a curvature radius of about 600 m before the reconstruction of 1977 had a width of 7.5 m. Its load-bearing structures were split spans that overlapped the span girders between the dam piers and consisted of four steel riveted solid beams and reinforced concrete carriageway slabs placed on them. And now, running a little ahead, we will point out that these spans were used during reconstruction of the driveway, as a result of which the width of the latter increased to 15.9 m (Fig. 5). At the same time, a 2.25 m wide sidewalk was installed on the downstream side of the dam and a 0.75 m wide inspection gangway on the upstream side.

The reason for the reconstruction was quite simple and obvious: it was caused by a significant increase in traffic intensity in 1977. That is why the reconstruction of the driveway was carried out, one of the main objectives of which was the expansion of the carriageway and the arrangement of four traffic lanes in both directions instead of the existed two lanes. At the same time, two traffic lanes remained along the spans of 1932, and the other two lanes were organized along the spans of 1977 (Fig. 6).

Fig. 3. Highway crossing on the superstructures at the Dnieper HPS dam. General view.

During the reconstruction, steel cantilever girders were brought under the existing spans (as shown in Fig. 6). Then they were installed on the dam piers and attached to the dam by means of anchoring tie rods made from wire ropes Ø 68 mm passed in the openings drilled in advance, which were injected with cement mortar after pre-tensioning the wire ropes. The box-section steel girders are taken without field joints.

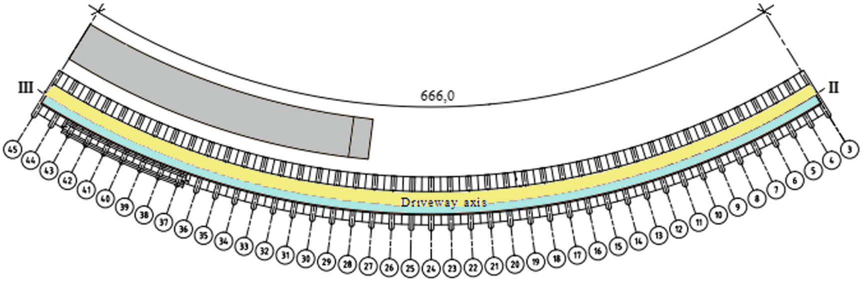

Fig. 4. Highway crossing on the superstructures at the Dnieper HPS dam. Layout: blue color – spans executed in 1932; yellow color – spans executed in 1977; gray color – Hydroelectric Power Station building-2; II – driveway beginning; III – driveway end.

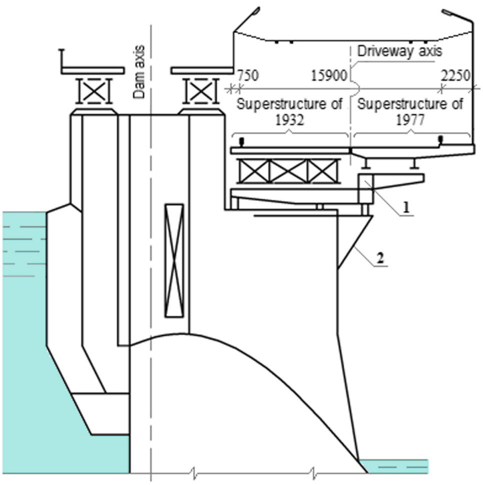

Fig. 5. Cross section of the dam after reconstruction of the carriageway in 1977: 1 – frame girder; 2 – side brace.

And the steel spans of 1977 are executed as split ones consisting of two main girders, each of them 1,000 mm high, crossbeams – with a pitch of 2,000 mm, longitudinal stiffeners – with a pitch of 400–575 mm with a steel plate located on top of them. Let's note that the crossbeams and longitudinal stiffeners, commonly with the decking, form a steel orthotropic deck plate of the carriageway, and the main girders have transverse stiffeners in the places where the orthotropic deck plate crossbeams rest on them. To reduce the number of transverse expansion joints, the top plates of the deck were welded during assembly with a pitch of about 100 m, thus forming a number of separate enlarged blocks of superstructures with a continuous plane of the carriageway. Taking into account the above, and also the fact that the length of the driveway on the dam superstructures makes 666.00 m, it becomes clear that in total, during the construction, six such span blocks were created. It should also be noted another significant design property of the driveway caused by the fact that the span of 1932 and the span of 1977 have a significant difference in rigidity. Since this difference could contribute to the negative action from one superstructure to another, in order to avoid such an effect, a continuous longitudinal expansion joint was arranged between these two constructs (Fig. 6), which additionally serves as a central reservation area for two traffic directions. As for the structural solution of the joints, both the longitudinal expansion joint and the transverse joints between the enlarged blocks are of the spring type.

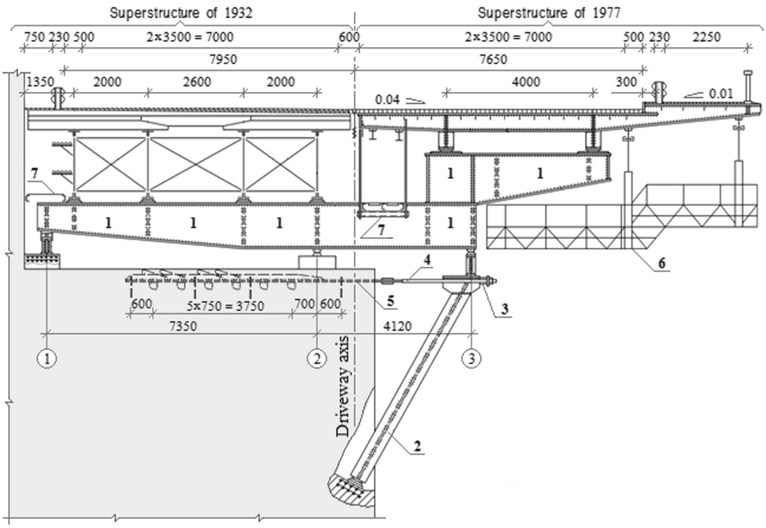

Fig. 6. Cross section of frame girder structures after reconstruction of the driveway in 1977: 1 – frame girder; 2 – side brace; 3 – lifting bar; 4 – tie rod; 5 – tie bar; 6 – movable inspection platform; 7 – inspection gangway; 8 – monorail; ①, ②, ③ – frame girder bearing axis.

In the 90s of the last century, it became necessary to repair steel frame girders, which was caused by three important factors. The first of them was a significant (several times) increase of vertical mobile loads from vehicles (especially large-capacity vehicles). The second one was associated with corrosion damage of the frame girders themselves, which led to partial loss of their cross-section. And the third, probably the most important one, consisted in progressive dynamics of grows of through cracks in concrete piers of the dam. Therefore, after development of several variants of repair work, it was decided to reinforce the frame girders by placing rigid side braces under their consoles (Fig. 6).

3 Design Model of Superstructure

Taking into account the configuration, structure and topology of the superstructure, a finite element model was taken as a design model, built by modeling six enlarged blocks of the driveway superstructures at the Dnieper HPS dam. In the model, all the constituent parts of the orthotropic deck plate and main girders are approximated by a set of thin shell triangular and quadrangular finite elements. In places where the main girders are supported on steel cantilever beams (Fig. 6), bracing elements are applied on corresponding displacements of the joints. The material of the structural components of the orthotropic deck plate and the main girders is steel grade 09G2S-12 with elastic modulus $E = 2.1 \cdot 10^5$ MPa, Poisson's ratio $\nu = 0.3$ and design resistance $R = 340$–400 MPa, depending on structural element thickness.

In terms of specifying and further taking into account in the finite element model the actual values of thicknesses of the finite elements of the steel plate, crossbeams and longitudinal stiffeners of the orthotropic deck plate, as well as the lower flange, web and transverse stiffeners of the main girders, we note the following. During the last special surveys of the driveway at the Dnieper HPS dam, a number of defects in bridge structures were identified (their exhaustive list and description is given in article [7], which not only negatively affect the durability of the latter, and in some cases even lead to a decrease in the total bearing capacity of the driveway. Therefore, to assess the influence of these factors on the stress–strain state of bridge structures, the existing defects were represented in the finite element model of the driveway superstructures, which was created by selecting all the initial data from the detailed drawings when carrying out priority strength and dynamic calculations of the driveway at the Dnieper HPS dam with design values geometric parameters. However, due to the significant number of identified defects, before their introduction into the finite element model of the driveway superstructures, their systematization and a certain typical averaging were performed. As a result, it was assumed that the defects can be reproduced in the model by adjusting the thickness of the elements of driveway spans, depending on the depth of their corrosion damage while maintaining other geometric and rigidity parameters. In Fig. 7, the constructed finite element model of the enlarged superstructure block of the driveway at the Dnieper HPS dam is shown, which has 9,938 joints and 9,920 finite elements. And in Fig. 8 the general finite element model of the driveway superstructure at the Dnieper HPS dam is shown, which has 59,047 joints and 59,588 finite elements.

4 Load on the Superstructure

It should be noted that the permanent loads and imposed loads on the superstructure are accepted in full compliance with the requirements of the norms [3, 6].

The permanent load on the driveway superstructure is the dead weight of its structural elements and it was taken into account by two types of uniformly distributed design load – along the line (designated by symbol P with corresponding subscript) and over the area (designated by symbol q with corresponding subscript) – depending on the type of structural element. Taking into account the above, nine components of the permanent load were taken as a whole, namely: carriageway plates ($q_2 = 2.4$ kPa), main girders ($P_1 = 2.6$ kN/m), barrier railings ($P_2 = 0.5$ kN/m) and pedestrian railings ($P_3 = 0.9$ kN/m), monorails with a movable inspection platform ($P_4 = 1.0$ kN/m), lighting poles ($P_5 = 0.9$ kN/m), inspection gangway ($P_6 = 1.0$ kN/m), carriageway deck surfacing ($q_2 = 2.4$ kPa) and sidewalk pavement ($q_3 = 1.0$ kPa). The load case for application of permanent load components to the cross-section of the superstructure is shown in Fig. 9.

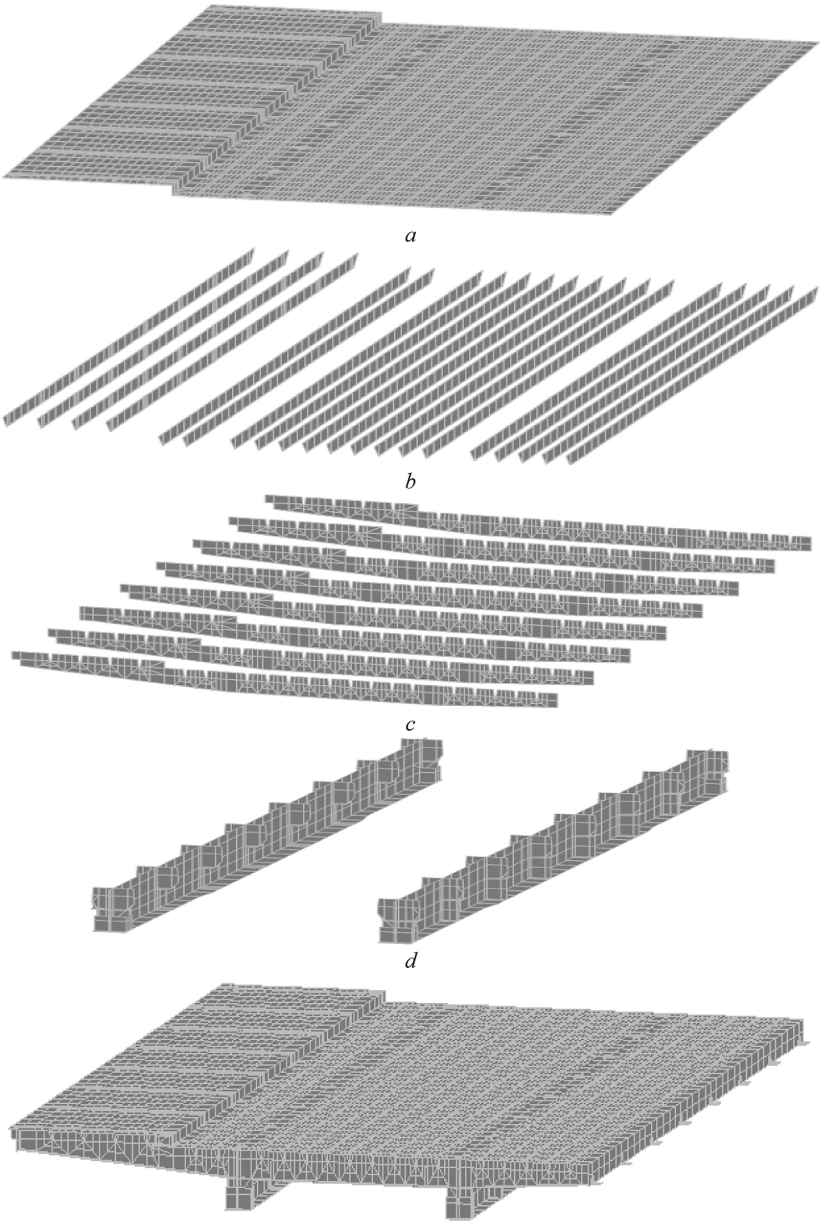

Fig. 7. Finite-element model of enlarged block of the driveway superstructure at the Dnieper HPS dam: *a* – model of plate decking; *b* – model of longitudinal stiffeners; *c* – model of crossbeams; *d* – model of main girders; *e* – general model of enlarged block.

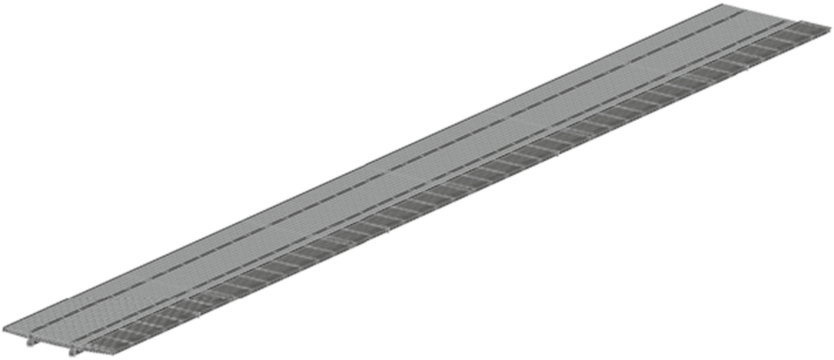

Fig. 8. General finite-element model of the driveway superstructure at the Dnieper HPS dam.

As for the imposed load on the driveway superstructure, it consists of three components, namely: wheel four-axle load from one motor vehicle weighing 0.8 MN (HK-80), wheeled vehicle load of a convoy of cars weighing 0.3 MN each (H-30) and uniformly distributed design load from human crowd on the sidewalk ($q_4 = 11.7$ kN/m). The load drawing concerning application of imposed load components to the cross-section of the superstructure is given in Fig. 10, which also shows the position of the resultant (indicated by *R* letter with corresponding subscript) from motor vehicles.

From the above information, it follows that the calculation took into account two groups of loads: permanent and imposed. Consequently, the total number of design loads was twelve, and combinations of the loads taken according to requirements of regulatory documents [3, 5, 6] was fourteen, depending on the application place of imposed wheeled vehicle loads on the span length in accordance with actual operating conditions of the driveway, and permanent loads were included in all combinations. Comparison of the calculation results obtained for all fourteen combinations of loads made it possible to determine the most unfavorable combinations for each characteristic design point of the driveway superstructure – on the supports and in span quarters.

At the same time, the most interesting thing concerning these results was that all considered fourteen load combinations (once again we emphasize – all of them!) turned out to be the most unfavorable. However, thirteen of them are of a local nature caused by acting permanent and temporary loads (HK-80 wheeled vehicle load and human crowd load on the sidewalk), since they were formed by sequentially rearranging the HK-80 wheeled vehicle load along the driveway, which led to occurrence of vertical displacements and normal stresses exclusively within limited areas around this wheeled vehicle load – from one four-axle wheeled vehicle weighing 0.8 MN. And only one combination, containing both permanent load and imposed load (from H-30 wheeled vehicle and human crowd on the sidewalk), turned out to be the most unfavorable combination as a whole for the entire driveway, an attribute of which was occurrence of maximum values for vertical displacements and normal stresses along the entire plane of its superstructure.

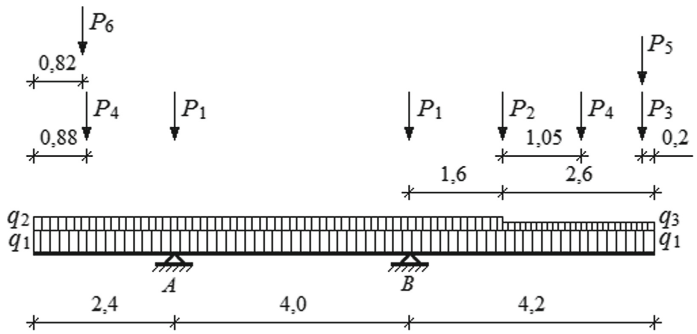

Fig. 9. Load case for application of permanent load components to the cross-section of the driveway superstructure: *A*, *B* – main girders.

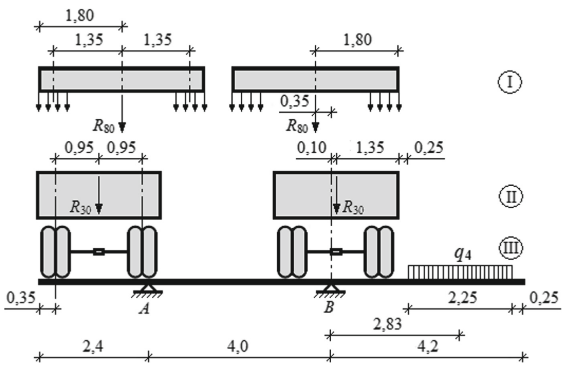

Fig. 10. Load drawing for application of imposed load components to the cross-section of the driveway superstructure: *A*, *B* – main girder; I – wheel four-axle load from motor vehicle HK-80; II – wheeled vehicle load H-30; III – load from human crowd on sidewalk.

5 Results of Numerical Calculations Made for Superstructure Bearing Strength

Taking into account the above, we'll consider behavior of the driveway superstructure at the Dnieper HPS dam under the most unfavorable load combination, namely: permanent load and imposed load as a part of the H-30 wheeled vehicle load from the convoy of cars weighing 0.3 MN each and the uniformly distributed design load from human crowd on the sidewalk. The results of numerical calculations in form of isofields for vertical displacements and normal stresses in the decking of an orthotropic plate and tangent stresses in the webs of main girders are shown in Figs. 11, 12 and 13.

As follows from the analysis of the results obtained, the decking of the orthotropic plate on the entire length of the driveway superstructure has two types of fundamentally different zones of vertical displacement (Fig. 11). The first zone is located above the places where the main girders are supported on the cantilever girders, it covers almost the entire width of the carriageway of the orthotropic plate with maximum positive displacement values of 3.51 mm. The second zone is located in the spans between the cantilever girders, where the decking deflects downward with a maximum value of 36.22 mm. As for distribution of normal stresses in the decking of the orthotropic plate (Fig. 12), we note that above the places where the main girders are supported on the cantilever girders, the decking resists tensile stresses in the range of $\sigma_x = 43$–48 MPa, and in span centers between the cantilever girders, the decking as a whole is more or less uniformly compressed within the range of $\sigma_x = 45$–80 MPa. As for behavior of the webs of main girders, a multidirectional stress state can be traced in them (Fig. 13): there are compression stresses that are insignificant in terms of values (up to $\tau_{xy} = 82.1$ MPa) in some fragments of these webs, in others of them there are tensile stresses (up to $\tau_{xy} =$ 82.1 MPa), which is caused by peculiarities inherent for the support of main girders on cantilever girders.

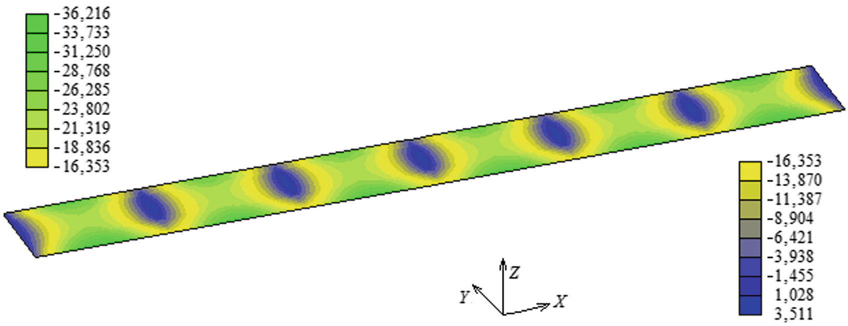

Fig. 11. Isofields of vertical displacements w in decking of orthotropic plate for the most unfavorable combination of loads, mm.

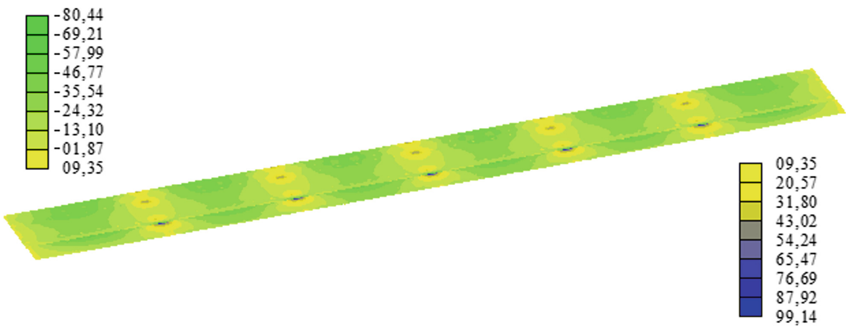

Fig. 12. Isofields of normal stresses σ_x in decking of orthotropic plate for the most unfavorable combination of loads, MPa.

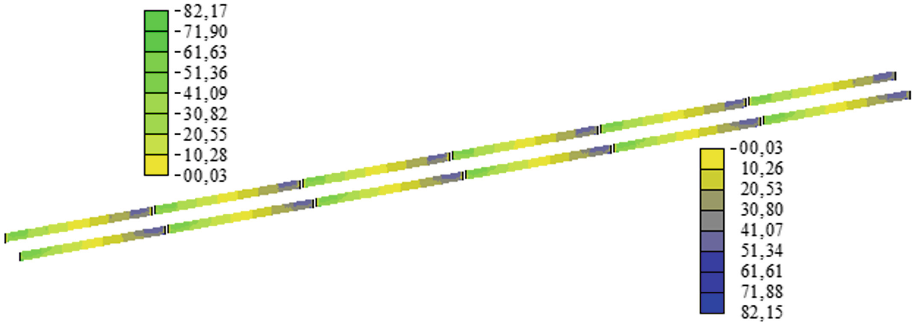

Fig. 13. Isofields of tangent stresses τ_{xy} in webs of main girders for the most unfavorable combination of loads, MPa.

Comprehending now the presented results, it is possible to understand that the orthotropic plate under the most unfavorable combination of loads is in a somewhat understressed state. The above can be explained by the fact that the design and construction of the driveway superstructure was primarily aimed at accelerating the solution of urgent city transport problems in Zaporozhye, which was then carried out in a very short time, and the main attention was paid exclusively to executing several accompanying organizational and technological measures, and economic indicators were practically not taken into account.

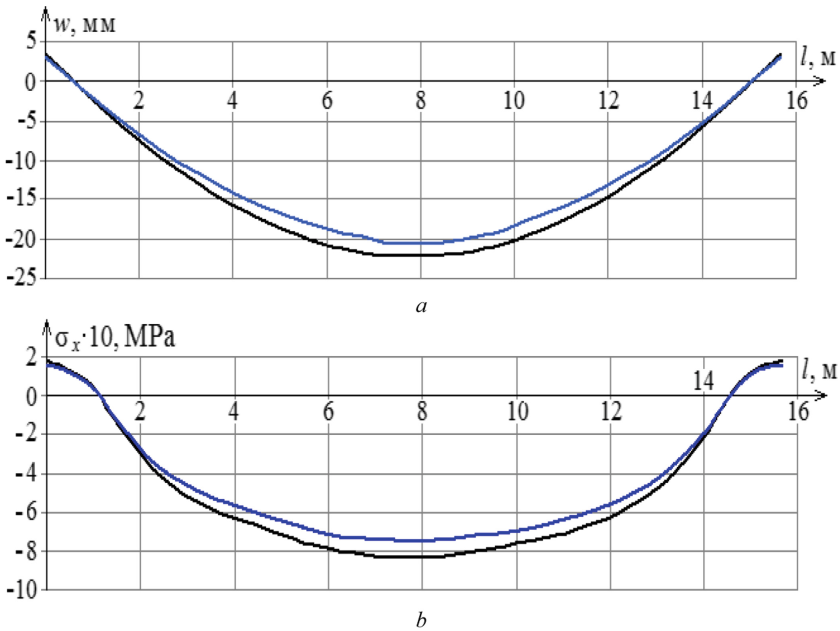

Fig. 14. Graphs of changes in stress–strain state parameters along local longitudinal axis of the second enlarged block of driveway superstructure: *a* – vertical displacements; *b* – normal stresses.

In addition to the above information, for one of typical enlarged blocks of the driveway superstructure at the Dnieper HPS dam (in this case, the second one), Figs. 14 and 15 show graphs of changes in stress–strain state parameters along the local longitudinal axis connecting the joints of the orthotropic plate decking with the largest positive displacement values and the local transverse axis in the middle of this block. Let us draw attention to the fact that the solid black curve on these graphs corresponds to the actual operation of the superstructure, taking into account defects and damage (including corrosion) of bridge structures revealed during the last special surveys of the driveway at the Dnieper HPS dam [7], and the solid blue curve presents its behavior in design position.

If we carefully analyze the graphs presented in Figs. 14 and 15, we can reach the expected qualitative conclusion that the defects and damage of the driveway superstructure arisen during the operation period affect the parameters of the stress–strain state of bridge structures. As for the quantitative assessment of this effect, we point out that discrepancy between displacements and stresses of design state and actual state is in the range from 8 to 11% for almost all structural elements due to actually uniform long-term corrosion of metal with slight weakening of cross-sections (simultaneously from Fig. 15*b*, it also implies that in the decking above the main girders of the orthotropic plate, there are insignificant undulating fluctuations in the values of normal stresses caused by conditions of interaction between the decking and the girder).

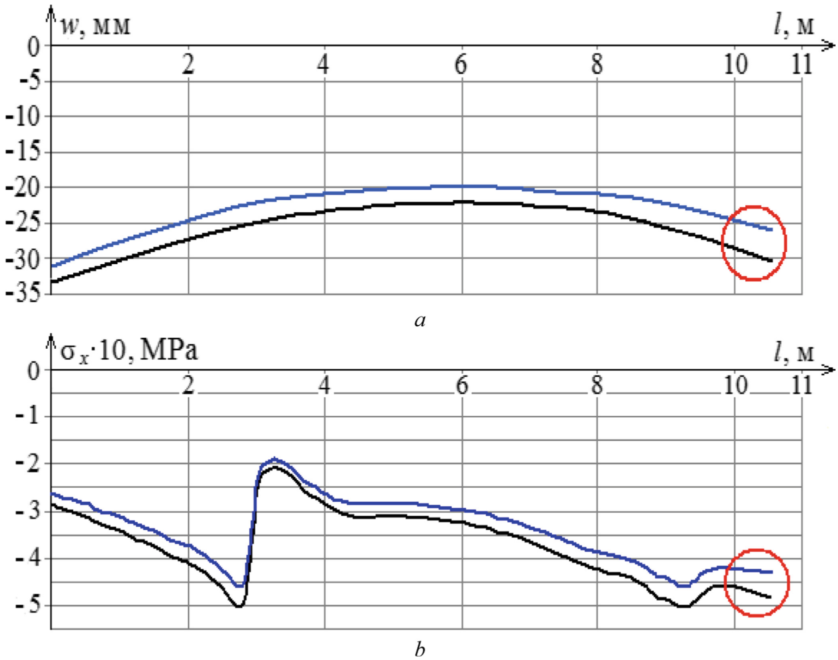

Fig. 15. Graphs of changes in stress–strain state parameters along local longitudinal axis of the second enlarged block of driveway superstructure: *a* – vertical displacements; *b* – normal stresses; **O** – zone at longitudinal expansion joint.

The only exception to this is a zone of about 0.6–0.7 m width, adjacent to the longitudinal expansion joint between the superstructure of 1932 and superstructure of 1977, which extends along the driveway entire length (Fig. 3). And the peculiarity of this zone is that, in addition to corrosion damage, there are also many through-burns in metal of the orthotropic deck plate elements, which happened in 2010 during disassembling and installation of longitudinal expansion joint, which is expounded in detail in the previously mentioned paper [7]. As a result in this zone, the difference between displacements and stresses of design state and actual state is increased by about 2–2.5 times and reaches 22–25%, which is clearly shown in Fig. 15.

6 Conclusions

The paper discusses the methodological aspects of an integrated approach to the calculation and analysis of operation of bridge superstructures with orthotropic deck plates of the carriageway. The paper presents a technique for studying the uneven distribution of normal stresses in the orthotropic plate decking. As a matter for research, the superstructure of existing driveway at the Dnieper HPS dam was considered, the design model for which was created taking into account the actual technical state of structural elements based on the results of the performed engineering surveys. The results obtained using the numerical calculation method are presented and analyzed in form of vertical displacements and normal stresses in the orthotropic plate decking, as well as tangent stresses in webs of the main girders. The reduction factors have been determined by applying engineering-and-analytical (normative) method and numerical calculation method. Comparison of the obtained results is carried out and recommendations are given regarding application of calculation methods.

The results and, as a consequence, recommendations formulated on their basis are as follows.

1. Calculations of bridge structures must be carried out taking into account their actual technical condition determined on the basis of the results of engineering surveys. The analysis of the data obtained shows that defects, damage and destruction in structural elements that have been in operation for many decades significantly affect under-load operation of these elements.
2. In order to obtain the numerous results most adequate to the actual technical condition of investigated bridge superstructure, one should proceed as follows. Namely: accounting for defects, damage and destruction in the finite element model of bridge structures due to their significant number (as a rule!) should be carried out not by presenting these "imperfections" directly, but indirectly – by adjusting thicknesses of finite elements depending on presence of corresponding "imperfection", while maintaining other geometrical and rigidity parameters. Moreover, the magnitude of this correction is to be determined by averaging the existing "imperfections" for the whole surface of the structural element.
3. When conducting research of bridge structures that have defects, damage and destruction acquired during operation, and in particular those that, in addition to this, are characterized by non-standard design solutions, it is necessary to use numerical

calculation methods for two reasons. The first one is that only numerical calculation methods make it possible to achieve the maximum possible structure detailing when creating a numerical model and, as a consequence, to obtain better, more reliable and accurate calculation results. The second reason is associated with the current regulatory documentation, where methods are absent that allow accounting in the standard engineering-and-analytical calculation method neither for engineering survey results for structural elements of bridge structures, nor for the applied non-standard structural solutions, and therefore the results determined on its basis contain a significant error.

4. Taking into account that the current regulatory documents do not contain provisions regarding the calculation and design of bridge structures accounting for data of engineering surveys and non-standard design solutions, it is recommended to supplement these regulatory documents with appropriate content sections.

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