

# Features of bridge superstructures modeling during the stress-strain state monitoring

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**Abstract.** The complication of element design performance of artificial structures leads to the need for more detailed superstructure modeling, since simplified models don't always allow taking into account deformation features of elements. In addition, during the building period, the actual stress-strain state of structures can be significantly influenced by factors that are difficult or impossible to account for at the design stage. Correct modeling of deformation of elements is one of the essential conditions for the prevention of emergencies during construction, maintenance or timely elimination of the consequences of such incidents with minimal losses. When performing this kind of work, attention should be focused on the most significant design parameters, which comprehensively reflect the nature of the structure operation. This will make it possible to effectively carry out control and promptly make appropriate adjustments to the building process, if it is necessary. The article describes an example of the simplified model modernization to more fully match its actual deformation and the experience of eliminating the consequences of emergency that arose during erection of unique bridge. Thus, the importance of adapting the computational model to the actual nature of the engineering design is confirmed.

## 1 Introduction

According to the research results [1-3], errors at the construction stage account for about 15% of the main causes of emergency destruction of bridges. Even when performing many times debugged earlier technological operations, abnormal situations are possible, which is clearly confirmed by the information in [4-6], and with the complication of the structural design of the elements, the risk of their occurrence increases even more. Timely refinement and adaptation of calculation models during the control allow to reduce the risks of emergency situations during construction and significantly increase the reliability of the structure during operation.

The difference in the values of the actual and calculated stresses indicates a discrepancy between the calculated model and the actual operation of the superstructure. This is undesirable, but permissible in the case of a clear understanding of the available design reserves, which is possible only if there is confidence in the correctness of the compiled design scheme and taking into account the main factors affecting the stress-strain state of the elements [7-9].

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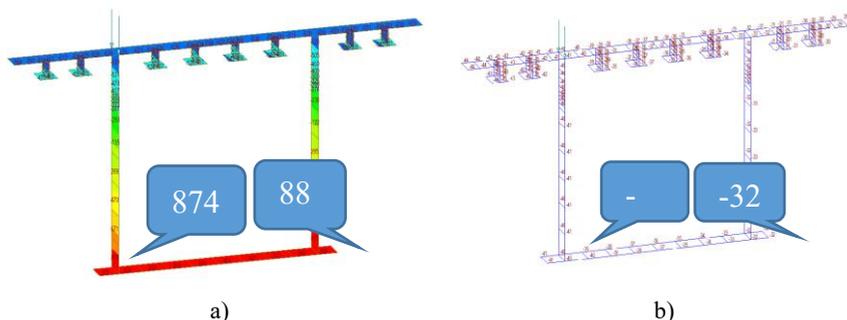
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## 2 Methods

### 2.1 Accounting of the torsional stiffness of the structure in simplified modeling

Drafting a detailed design superstructure model is not always advisable. Raising of the calculation models detailing, accompanied by an increase in labor costs, as a rule, does not give an equivalent reduction in the error of calculation results, as evidenced by the comparison of various calculation schemes with the results of field observations.

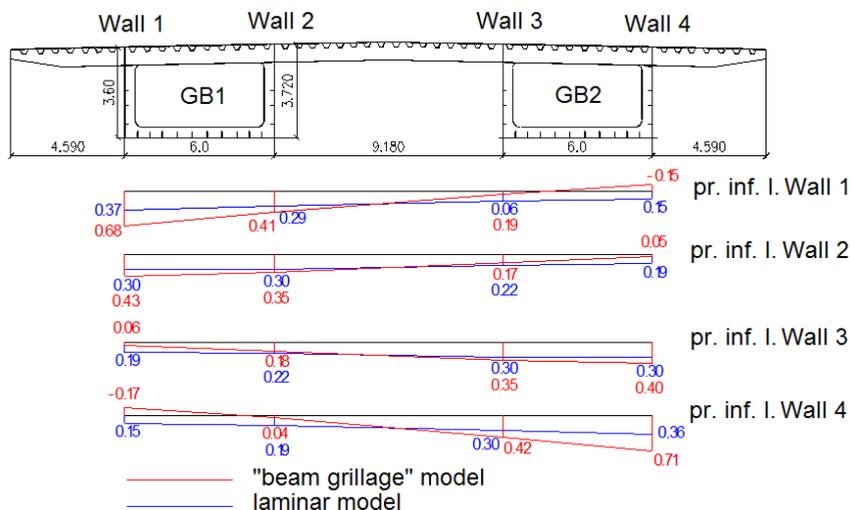
The torsional stiffness of the structure is one of the factors that significantly affect the value of the transverse stress distribution in the structural elements (the value of the transverse installation coefficient TIC). Under the action of temporary or permanent loads with some eccentricity relative to the center of bending of the superstructure in the elements of a closed cross-section (which is important in the case of overlapping large spans with box beams), additional stresses arise due to deformations of pure torsion and constrained torsion [10]. From the action of the bimoment, the magnitude of the transverse distribution of stresses in the elements of a closed structure for a number of points is inversely proportional to the magnitude of the vertical displacement. So, for example, when the load is located above the left wall of a box girder, as shown in Fig. 1, due to the effect of the bimoment, the magnitude of the normal stresses at the level of the lower belt is greater under the right wall, while the value of the vertical displacement is less.



**Fig. 1.** Influence of the bimoment on the stress-strain state of the element a) normal stresses acting along the axis of the superstructure, kgf/cm<sup>2</sup>; b) vertical displacements from a similar load, mm.

For the most correct accounting of the influence of these parameters, closed load-bearing structural elements should be modeled by laminar finite elements. In this case, it is necessary to ensure the condition of invariability of the section contour, which is possible only if all stiffeners are taken into account in the model. For fairly resource-intensive tasks, such as the accounting of all stages of the longitudinal sliding of an extended bridge, the creation of a laminar model and subsequent calculations will require significant time expenditures, therefore, it looks more preferable to model the superstructure as a "beam grillage". Each wall of the box with adjacent sections of the upper and lower orthotropic plates is represented by its own branch of finite elements, and in the transverse direction these branches are united by the upper and lower transverse beams with the adjacent sections of the flooring sheet. But it is necessary to take into account the fact that the total stiffness for free torsion of longitudinal rod finite elements will be an order of magnitude lower than when modeling a closed section with one branch of beam elements, which in turn will lead to an incorrect distribution of stresses between two walls. As an example, Fig. 2 shows the pressure influence lines for each box wall in a superstructure, consisting of two box-section beams, at the point of maximum deflection of the beams from the applied load. For the purpose of comparison, the ordinates of the pressure influence lines were determined in two different

ways: from the calculation of the laminar design model of the structure and from the beam model with the torsional stiffness of the elements calculated by the automated tools of Midas Civil.



**Fig. 2.** Pressure influence lines on the walls of boxes.

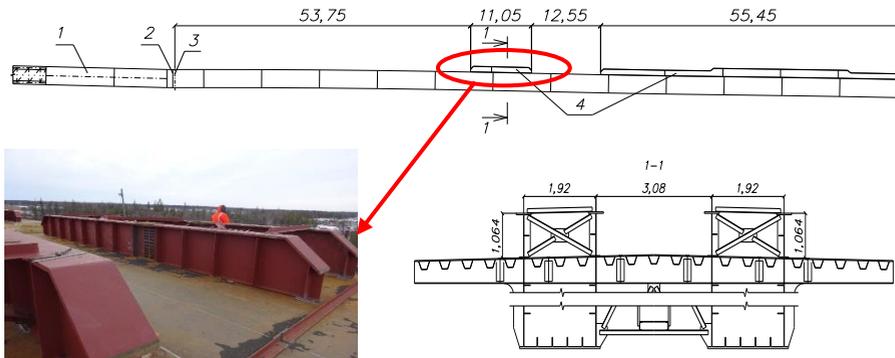
Accounting for the constrained torsion of thin-walled rods is not implemented in all finite element complexes, and, often, the torsional stiffness of a structure must be calculated separately. The selection of the required torsional stiffness of the longitudinal beam elements that make up a unified box can be performed using a spatial laminar model or analytically. When choosing the first option, detailed modeling of the calculated superstructure is necessary with the obligatory consideration of the transverse stiffeners of the walls and the bottom belt and supporting diaphragms when using laminar or volumetric finite elements. And further selection of the actual torsional stiffness of longitudinal beam finite elements corresponding to the box wall with adjacent sections of the upper and lower orthotropic plates can be performed, for example, by comparing the deflections of the plate and beam models from the same load value applied to the box wall. During analytical calculation, the torsional stiffness of an element with a section in the form of a box wall with adjacent sections of the upper and lower orthotropic plates should be equal to half the torsional stiffness of the closed section. Performing such calculations is much less laborious than compiling a laminar model when obtaining comparable results with an accuracy sufficient for engineering calculations.

## 2.2 The need to clarify and adapt calculation models during control

Uncorrected accounting or ignoring seemingly insignificant factors during the design process can cause an abnormal situation even when performing routine installation operations, which is why specialized organizations monitor the stress state of structures during the construction of atypical structures [11-14]. For example, the conveyor-rear assembly with subsequent longitudinal sliding, which allows put the river bed without erecting temporary supports or scaffolds, is one of the most common technological processes for the construction of metal superstructures. With an increase in the length of the spans, special auxiliary structures and devices - lightweight launching girders and subdiagonal trusses - or even their combinations are widely used. Other structures are used less often, therefore, less experience in their use and, accordingly, calculation, does not yet allow foreseeing in advance all the features of their work, the manifestation of which is possible

during the construction process. In this regard, malfunctions of the stiffener beam may occur due to the use of previously unused devices or technologies.

This happened, for example, when using external reinforcement elements during sliding the metal superstructure of a road bridge, made according to the scheme (84+105+2x126+105) m. In the transverse direction, the superstructure consisted of two box-shaped main beams, united on top of ortotopic desk. External reinforcement consisted of I-beams of variable height from 1.05 to 1.5 m, located in two sections along the length of the superstructure above the walls of the boxes and connected in pairs by a system of longitudinal and transverse ties. The schematic and illustrations of external amplification structures are shown in Fig. 3.



**Fig. 3.** Scheme of external reinforcement of the superstructure: 1 – launching girder; 2 – end of the superstructure; 3 – axis of bearing on a permanent support; 4 – external reinforcement.

The elements of external reinforcement were supposed to be included in the joint work with the main beams by welding the lower flanges of the I-beams to special tables welded to the flooring sheet (Fig. 4). The length of the tables was 0.20...0.25 m. The step of their location along the length of the superstructure was 0.95...2.05 m.

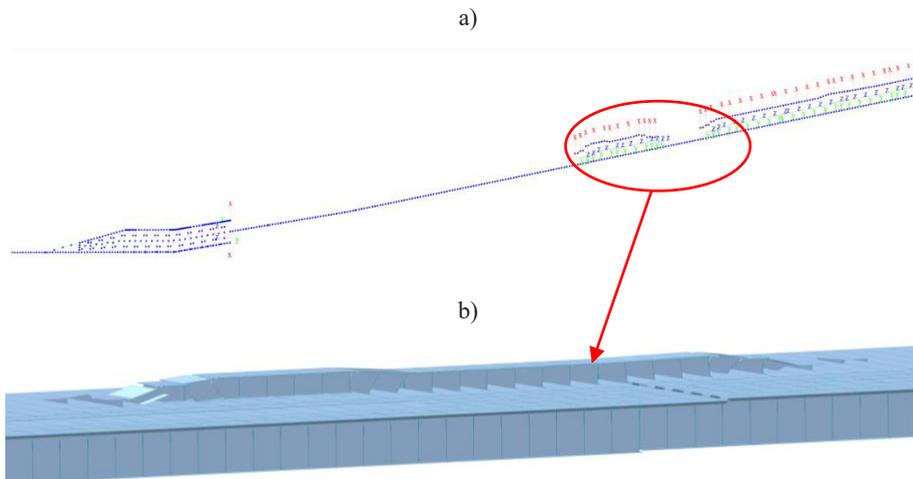
At the first stage of sliding into a span of 105 m, at the moment of the transition of the beginning of the first section of reinforcement through the support axis of the intermediate support, welds broke along the perimeter of the tables, and the reinforcement was partially turned off from the joint work with the beams of the superstructure. Cracks in the seams were formed at the point where the I-beams were attached to the first three tables (see Fig. 4), which resulted in a sharp jump in stresses in the main beams in this section. After the reinforcement I-beams were restored to the superstructure, the sliding was continued and the first stage was completed without significant incidents. However, due to the fact that the reinforcement was re-enabled at the moment of a non-zero stress level in the dangerous section, the design prerequisites laid down in the development of the installation technology no longer corresponded to the actual state of the structure. Moreover, four reinforcement beams were switched off from the joint work with the superstructure not at the same time and in different sections (all the tables separate in the first two sections, and only one in the third). All this led to the uncertainty of the stress-strain state of the superstructure above the initial section of reinforcement before the next stage of sliding.

At the next stage, a sliding into a span of 126 m was envisaged, respectively, larger values of efforts were expected, which increased the likelihood of a repetition of an emergency situation with separation of the reinforcement from the superstructure. The rupture of the seams during the first stage of sliding led to a redistribution of efforts in neighboring attachment nodes and the exclusion of reinforcement from work in certain sections. The situation was aggravated by the fact that the beams of the superstructure before the start of the second stage were clearly loaded more than it was anticipated in the project.



**Fig. 4.** Defects in the seams of attachment of external reinforcement to the superstructure.

To prevent the development of such malfunctions, computational studies were performed before the next stage of sliding. For this, a flat finite element model was prepared in the MidasCivil design complex, taking into account the staging of the construction of metal structures and the application of loads, self-extracting of the launching girder, as well as the shutdown of external reinforcement elements at the appropriate stages of sliding and their subsequent restoration. Main beams and reinforcement I-beams are defined by beam finite elements, their centers of gravity are connected by rigid links. The same model was used to determine the forces acting in the elements of the superstructure during the sliding process. Fragments of the design model of the superstructure are shown in Fig. 5.



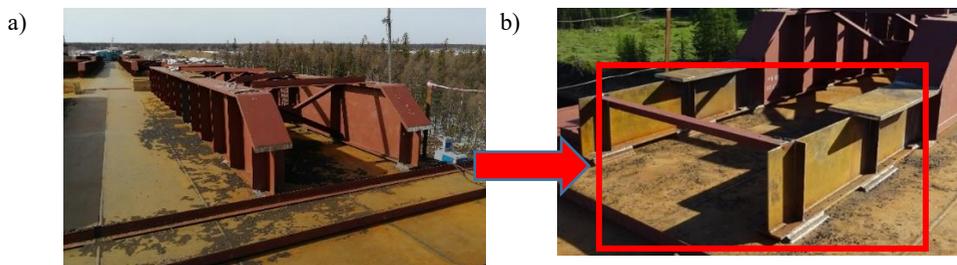
**Fig. 5.** Fragments of the design model of the superstructure: a) a fragment of the design model "in the axes"; b) a fragment of the design model "in the body".

In the course of the studies carried out, an assumption was made about the storey nature of the structure operation and the representation of tables as "support nodes" for external reinforcement I-beams. This differed from the initial design assumptions, according to which the main beams and temporary reinforcing I-beams should work together in the elastic stage without taking into account the point support. According to the results of refined calculations, the actual length of welded joints in the initial section of reinforcement was found to be insufficient for the perception of horizontal support reactions occurring in these nodes. In order to test this hypothesis and assess the degree of inclusion of external reinforcement elements in the joint work at the next stage of sliding, one of the controlled sections was assigned in the alignment with reinforcement I-beams, and the other – in the alignment without reinforcement.

The control of the level of normal stresses in the cross-sections of the superstructure and reinforcement elements was carried out by measuring local deformations by two independent systems – mechanical and strain gauge. The combined use of different methods for monitoring the stress-strain state of a structure made it possible to obtain reliable data. The mechanical control system included the setting of strainmeters and additional periodic measurements by comparators, the strain gauge system included measuring sensors of the "Tensor MS" system. The "Tensor MS" equipment is widely used when performing all kinds of measurements [15], including for fixing atypical parameters, for example, deformations of asphalt concrete pavement of the roadbed of road bridges [16].

The results of monitoring the stress-strain state of the superstructure elements have confirmed the validity of the hypothesis put forward. In the section without reinforcement, the behavior of the main beams and the distribution of stresses were in good agreement with the expected ones, while in the reinforced section a somewhat different picture was observed. The actual values of the stresses in the I-beams of the reinforcement in the span between the tables were almost half the calculated ones, and along the lower fiber they turned out to be higher than along the upper one, which did not correspond to the initial thesis about joint work with the main beams in the elastic stage. This type of stress distribution was due to the application of tensile forces to the lower flange of the I-beam, and not to its center of gravity, which led to the appearance of a bending moment compressing the upper fibers of the reinforcement beams.

On the basis of the results obtained, a design of additional reinforcement of the superstructure has been developed, which ensures its smooth inclusion into joint work with the superstructure when "hitting" the support (Fig. 6, b). In particular, the required length of the welded seams was calculated, the geometric parameters of the cross-section of the newly constructed elements were assigned, the stages of the arrangement of additional sections of reinforcement and the removal of residual stresses in the existing welded seams were determined. This design has confirmed its efficiency at the subsequent stages of sliding, the amplification was no longer switched off from work.



**Fig. 6.** Design of external reinforcement elements of the superstructure: a) the originally designed external reinforcement structure; b) the design of additional reinforcement developed based on the results of calculations for a smoother inclusion of external reinforcement elements in joint work with the main beams.

### 3 Results and discussion

The article considers the process of adapting the calculation model to the superstructure real nature using certain examples. Based on obtained in practice results, engineering solutions developed, which are able to eliminate the consequences of deviations of the actual work of structures from the expected, unaccounted by calculations. The results presented in the article show that accounting various factors and further model refinement are very important to ensure the safety of construction work and the regulatory reliability of the operation of the

structure. This accounting does not always require a lot of labor, which is demonstrated in this work. A problem often can be solved by local changes in design parameters, for example, reinforcement, or small adjustments to the work process.

## 4 Conclusions

Thus, adaptation of the design model to the actual nature of construction makes it possible to more reliably assess the technical condition of the structure at any time. The refined data make it possible to clarify the conditions for passing the moving load on the bridge, preventing the introduction of unreasonable prohibitions or limiting the mode of movement of atypical vehicles. All this leads to increasing in the safety and durability of the structure, reduces the risk of emergency situations during operation and allows them to be eliminated in a timely manner and with less losses.

## References

1. I.Yu. Maystrenko, et.al, Internet magazine "Transport structures" **4(4)** (2017) <https://doi.org/10.15862/13TS417>
2. I.G. Ovchinnikov, et.al, Internet magazine "Transport structures" **4(4)** (2017) <https://doi.org/10.15862/14TS417>
3. S.A. Dergunov, A.B. Satyukov, A. Yu. Spirina, S.V. Serikov, Bulletin of KSUCTA **2(64)**, 289-294 (2019) <https://doi.org/10.35803/1694-5298.2019.2.289-294>
4. V.I. Travush, V.I. Kolchunov, E.V. Leontiev, Industrial and civil construction **2**, 46-54 (2019) <https://doi.org/10.33622/0869-7019.2019.02.46-54>
5. S.-J. Zhao, et.al, Journal of Railway Engineering Society **34**, 59-64 (2017)
6. L. Betuzzi, Collapse of the Second Narrows Bridge during Construction, pp. 151-163 (2017) <https://doi.org/10.1061/9780784481035.014>
7. V. Yu. Krasnoshchekov, Bulletin of the Siberian State Automobile and Highway University **15(1)** (2018) <https://doi.org/10.26518/2071-7296-2018-1-88>
8. A. Adams, et.al, (2019). Manual for Refined Analysis in Bridge Design and Evaluation
9. Y. Gosteev, et.al, Energies **15(5)**, 1864 (2022) <https://doi.org/10.3390/en15051864>
10. Y. Koleková, M. Kováč, I. Balaz, Procedia Engineering **190**, 603-610 (2017) <https://doi.org/10.1016/j.proeng.2017.05.386>
11. I.G. Ovchinnikov, I.I. Ovchinnikov, O.I. Nigmatova, E.S. Mikhaldykin, Russian journal of transport engineering **1(2)** (2014) <https://doi.org/10.15862/01TS214>
12. V.I. Nayanov, Yu.V. Nayanov, The third Saratov Salon of Inventions, Innovations and Investments. Saratov: Saratov University Press. Part 1, pp. 105-106
13. V.Ya. Shvidkij, T.G. Zvereva, University news «Geodesy and aerial photography» **62(3)**, 265-270 (2018) <https://doi.org/10.30533/0536-101X-2018-62-3-265-270>
14. Campbell Middleton, et.al, Bridge Monitoring: A Practical Guide (2016)
15. I. Zasukhin, et.al, Lecture Notes in Networks and Systems 403 (2022) [https://doi.org/10.1007/978-3-030-96383-5\\_9](https://doi.org/10.1007/978-3-030-96383-5_9)
16. A.N. Yashnov, S. Polyakov, Russian Journal of Building Construction and Architecture **3(39)**, 93-106 (2018)