INSPECTION AND PREPARATION FOR TESTING OF THE ROAD OVERPASS OF THE ALMATY-KAPSHAGAI HIGHWAY AFTER THE VEHICULAR IMPACTS

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Resume

The 33.0 m long reinforced concrete bridge beams of the overpass superstructure after vehicular impacts were taken as a research object.
The overpass consists of 14 beams, six of which were repaired by restoring the widened lower part with EMACO FAST TIXO, manufactured by BASF.
The remaining 8 beams were completely dismantled and replaced with new ones. The new beams have been fully tested for the perception of vehicular loads.
The fully reconstructed span structure showed compliance with the design loads of A14, NK-120 and NK-180 based on test results.

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1 Introduction

The present article brings the survey results of the bridge beams damages of a prestressed concrete girders double-span structure of a length of 33 m, caused by vehicle impacts in the Almaty-Kapshagai highway overpass section.

The basic goal of the constructed bridge structure survey was to establish its compliance with the project plans and the regulatory requirements on the operation quality.

The road overpass is located on a straight section in the plane and on a longitudinal axis in the $i = 5$ ‰ profile. As per the project, the angle of intersection of the longitudinal axis of the road overpass with the existing highway is 66° (Figure 1). The bridge scheme was adopted as 33+33 m. The total length of the bridge according to the project is 66.91 m [1].

The overpass superstructures are made of 33 m long pre-stressed VTK-33U beams (VTK - VTK - Russian abbreviation for High-Tech Construction).

On the top of superstructures beams, a laid-on slab made of monolithic reinforced concrete with grouting joints along the slab of the roadway of the VTK-33u beams was installed.

2 Materials and methods

A detailed examination of the damaged VTK-33 beams at the superstructure No. 1 was performed after the accident. As can be seen from Figure 2, most of the VTK-33 beams were significantly damaged.

Figure 2 shows that all fourteen VTK-33u beams were damaged as a result of vehicle impacts. Along with the destruction of the widened lower part of the beams, the destruction of the bottom flanges of the beams took place with the formation of cracks in them along the longitudinal axis of the beams.

Table 1 lists data on the presence of longitudinal cracks in beams ribs and their length.

The destruction of the widened lower part and the flange of the beam No. 1 are present with a rupture of the wire of the ropes K-7 (Figures 3 and 4).

A significant damage in beams 2, 3, 4 and 11 are present (Figures 5-8). In them, the bottom flange is

Figure 1 *The general view of the overpass from the Kapshagai city side: 1 and 2 - superstructure numbers*

Figure 2 *The general view of the damaged superstructure No. 1 from the Kapshagai city side*

destroyed, ruptures of the reinforcement of a smooth and periodic profile took place. Longitudinal cracks of various lengths were revealed in the flanges of those beams.

Authors presented the results of a damage survey and an assessment of the prestressed concrete girders viaduct repair works, damaged as a result of a vehicle impact [2]. It has been analytically proven that the structure can be strengthened by assembling prestressed reinforced concrete beams at both ends.

Beams 5, 6, 7 and 8 have a lower degree of damage (Figures 9-12). In them, the destruction of the widened lower part of the beams is mainly present.

Table 1 *The numbers of beams, the presence of cracks and the length of their stretch in bottom flanges*

Numbers when counting from the Kapshagai city towards Almaty city	1	2	3	4	5	6	7	8	9	10	11	12	13	14
The presence of longitudinal cracks in the ribs of the beams at the base of the upper part of bottom flange and the length of their stretch, m	+ 8.6	+ 6.2	+ 9.2	+ 3.3	- -	- -	- -	- -	- -	- -	+ 3.9	+ 6.2	+ 5.4	+ 3.2

Note:
«+» - longitudinal cracks are present in the flange of the beams;
«-» - no longitudinal cracks are present in the flange of the beams.

Figure 3 *A general view of the destruction in beam No. 1 from the Kapshagai side (Destruction of the bottom flange of beam and its widened lower part. A longitudinal crack was revealed in the upper part of the beam flange of a length of about 8.6 m)*

Figure 4 *Fragment of destruction in beam No. 1 and damage with wire rupture of K-7 ropes from the Kapshagai side*

Figure 5 *A general view of the destruction of the bottom flange of the beam No. 2 (The reinforcement of the smooth and periodic profile is damaged. The bundle of prestressed reinforcement is exposed)*

Figure 6 *Destruction of the widened lower part of beam No. 3 from the Kapshagai side (Smooth and periodic profile reinforcement was torn. A longitudinal crack was revealed in the upper part of the beam rib of a length of about 9.2 m)*

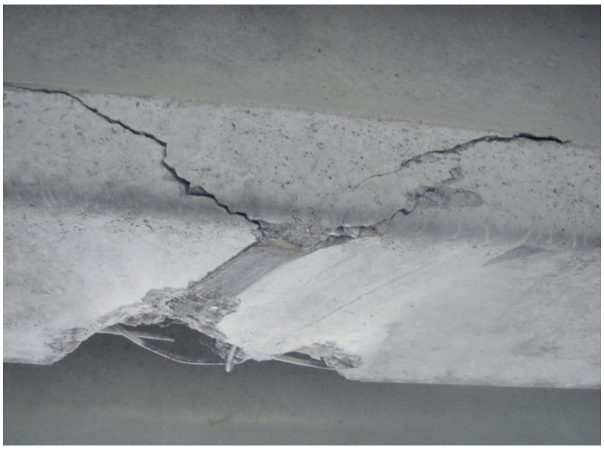

Figure 7 *Destruction of the widened lower part of the beam No. 4 from the side of Almaty (The reinforcement of the smooth and periodic profile is torn. A longitudinal crack was revealed in the upper part of the beam rib of a length of about 3.3 m)*

Figure 10 *Damage of the protective concrete cover in the widened lower part of beam No. 6 from the Kapshagai side (The longitudinal smooth reinforcement is damaged)*

Figure 8 *Destruction of the widened lower part of the beam No. 11 from the Kapshagai side (The reinforcement of the smooth and periodic profile is damaged. A longitudinal crack was revealed in the upper part of the beam rib of a length of about 3.9 m)*

Figure 11 *Damage of the protective concrete cover in the widened lower part of beam No. 7 from the Kapshagai side*

Figure 9 *Fragment of the destroyed widened lower part of beam No. 5 from the Kapshagai side (The reinforcement is torn; the bundle of prestressing reinforcement is exposed)*

Figure 12 *Destruction of the widened lower part of the beam No. 8 from the Kapshagai side*

3 Technical assessment of the superstructure beams No. 1

The obtained survey results of the VTK-33U beams of the superstructure No. 1 showed that destruction of the widened lower part of the beams (bottom flange), rupture of smooth and periodic reinforcement, rupture of wires and exposure of K-7 ropes of the pre-stressing reinforcement took place in them. Additionally, damage to ribs in a number of beams with the formation of longitudinal cracks at the base of the upper haunch were present.

Article [3] presents the results of inspections of reinforced concrete bridges in the north of Slovakia after 40-50 years of operation. Based on the results of residual life calculation, recommendations for further bridge operation are given. An assessment model for determining the priority strategy for bridge maintenance is described in [4]. The publications [5-6] present the results of the analysis of damage to bridges with box girders and risk assessment for the bridge structures using a fuzzy structure in a car explosion.

The following notes should be considered based on the analysis of damage in the VTK-33U beams:

- in beam 1, the destruction of the rib, the lower widened part and the rupture of the wires of the K-7 ropes are present;
- in the beams 5, 6, 7, 8, 9, 10 destructions of the widened lower part, damage to the reinforcement of a smooth and periodic profile, exposure of bundles of strained reinforcement have occured;
- in the beams 2, 3, 4, 11, 12, 13, 14 there were destructions of the widened lower part, damage to the reinforcement of a smooth and periodic profile, exposure of bundles of stressed reinforcement and longitudinal cracks of various lengths in the edge of the beams at the base of the upper part of bottom flange.

Bridge damage detection based on the visual data was performed by revealing the undamaged areas, that make up more than 80-90% of the total surface area. Next, the contour masks were created to refine the classification in the surface texture. The mentioned method reduced the search space for the inspector by 90.1% [7].

The results of reinforcement models of reinforced concrete T-beams are presented in [8]. It is shown that reinforcement with one MBrace® S&P CFK 150/2000 bend lamella and three S&P C-Sheet 640 100 mm wide improves the bending strength of the models.

Damaged beams, excluding the beam No. 1, could be repaired. At the same time, in the beams No. 1, 2, 3, 4, 11, 12, 13 and 14, along with the damage to the lower widened part, there were longitudinal cracks of various lengths in the bottom flange of the beams, at the base of the upper sides. The presence of those cracks significantly reduced the bearing capacity of those beams. Given the presence of such damage, it was recommended to dismantle these beams, which was done.

In beams No. 5, 6, 7, 8, 9 and 10, only the lower broadened part of the beams was damaged, caused by the destruction of the protective layer of concrete. A detailed examination revealed that the bundles of stressed reinforcement in the lower widened part of the beams were not damaged, which did not cause the reduction of the bearing capacity of these structures, the presence of such damage gave reason to recommend the repair of those beams.

For repairment of beams No. 5, 6, 7, 8, 9 and 10, BASF's EMACO FAST TIXO was recommended. This material is a non-shrinking, fast-hardening dry mix of a thixotropic type, containing polymer fibers, intended for structural repairs of concrete and reinforced concrete.

In article [9] are cited and described the methods and repair materials applied towards the improvement of prestressed concrete beams performance.

Application of organomineral modifier for the transport structures is given in [10]. On an example of concrete samples of brands M50 and M100 40×40×160 mm with W/C = 0.5 it is shown that the bending strength of modified concretes is higher on the average by 89-104%.

The use of fiber-based repair compounds can slow down the corrosion in loaded reinforced concrete beams [11], improve the operation of reinforced concrete structural elements under extreme conditions [12].

4 Bridge beam testing

4.1 Beam testing preparation

As stated in section 3, beams No. 1, 2, 3, 4, 11, 12, 13 and 14 were recommended to be dismantled. These beams were dismantled and replaced by the new prestressed concrete bridge beams VTK-33u, 33 m long, manufactured by Magnetik LLP (Zarechny village, Almaty region, Kazakhstan). Those beams are designed for superstructures of road bridges designed for the impact of temporary mobile loads A14, NK-120 and NK-180.

As per the project, the VTK-33u beam concrete class is B35 according to [13]. Figure 13 demonstrates a longitudinal section and cross sections of the VTK-33u beam. Seven-wire strands K-7 with a diameter of 15 mm according to GOST 13840-68 "Steel reinforcing ropes 1x7. Specifications" [14], combined by four strands into a cables (7 wires) and two strands, combined into one cables (Table 2, Figure 13). The area of the prestressing tendons, adopted in the project when reinforcing the beam, is A_p = 4170 mm^2. Three rods with a diameter of 14 mm of class A 400 according to GOST 34028-2016 are accepted as working longitudinal non-stressed reinforcement [15]. The area of the non-stressed reinforcement adopted in the project when reinforcing the beam is As = 462 mm^2.

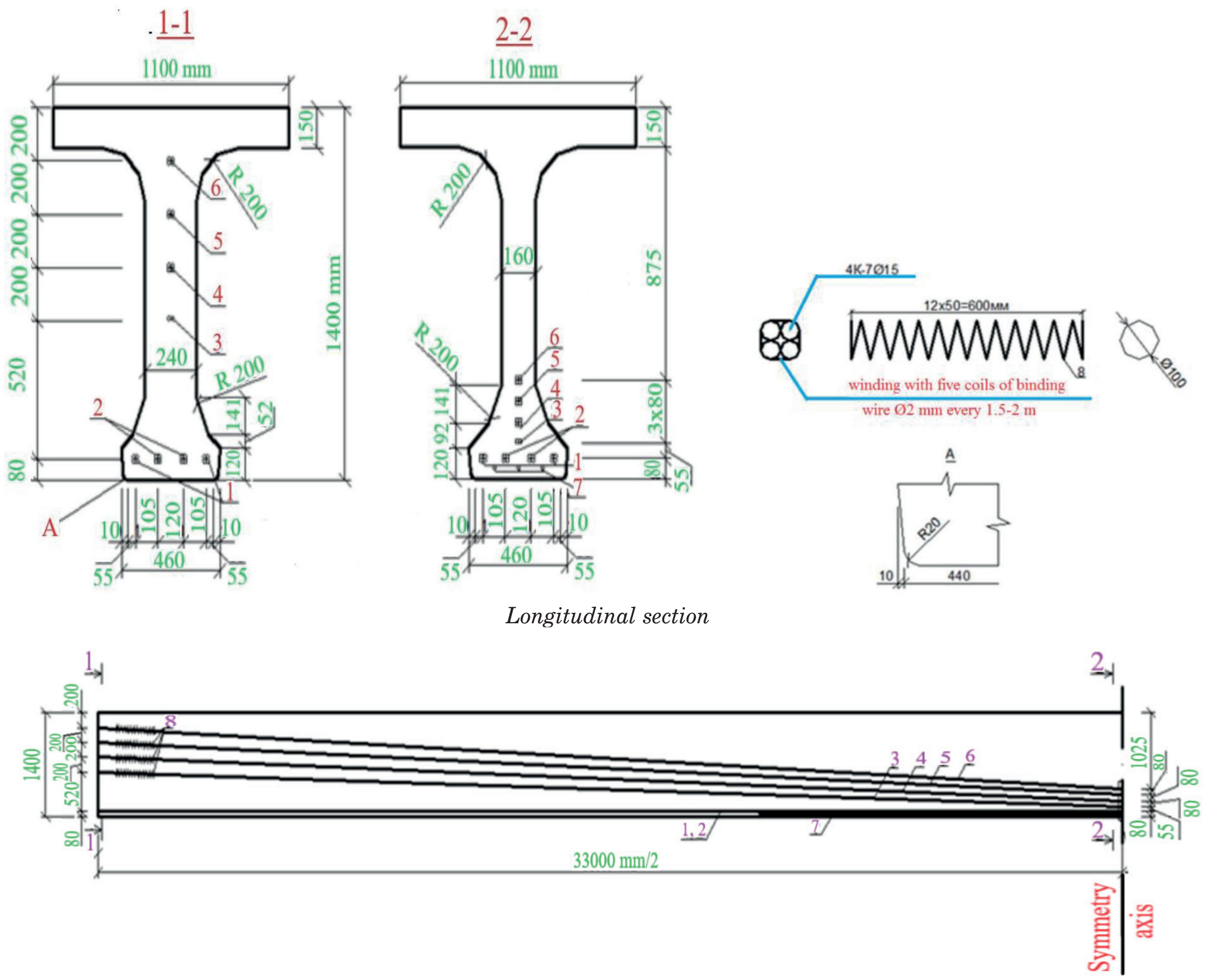

Figure 13 *Longitudinal section and cross sections of the VTK-33u beam*

Table 2 *Specification of the beam reinforcement elements*

No.	Title	Elements	Quantity	Weight, kg
1		L=34200	2	301
2		L=34200	2	301
3	Cables of 4 strands K-7 Ø 15mm as per GOST 13840-68*	L=34200	1	150.5
4		L=34200	1	150.5
5		L=34200	1	150.5
6		L=34200	1	150.5
7	Longitudinal bars	Ø 14 A 400 L=11700	3	43
8	Helix	Ø 6 240 L=4500	16	16

As per Construction Directives and Rules 2.05.03-84* "Bridges and Pipes"[16], the beam testing control loads were:

- Stiffness - $2P_c$ = 280 kN;
- Crack resistance - $2P_c$ = 360 kN;
- Strength - $2P_c$ = 630 kN.

Prior to the start of the tests, an instrumental survey of the bottom of the beam was done using a geodetic tool in order to determine the beam deflection. Instrumental survey was carried out using a SOKKIA C3030 level and a geodetic rod.

The experimental bending of the beam in the middle of the span, loading devices mass included was 42mm and the loading devices mass excluded was- 45.4mm.

The difference between the calculated and experimental bending of the beam made Δ = 45.8 - 45.4 = 0.4mm, which indicates a satisfactory convergence.

Figure 14 demonstrates the design scheme of the experimental product and the outline of the beam bottom in the presence of loading devices on it. Review of the beam deflection curve indicates a satisfactory shape of the formwork bottom.

On the day of testing, the actual strength of the

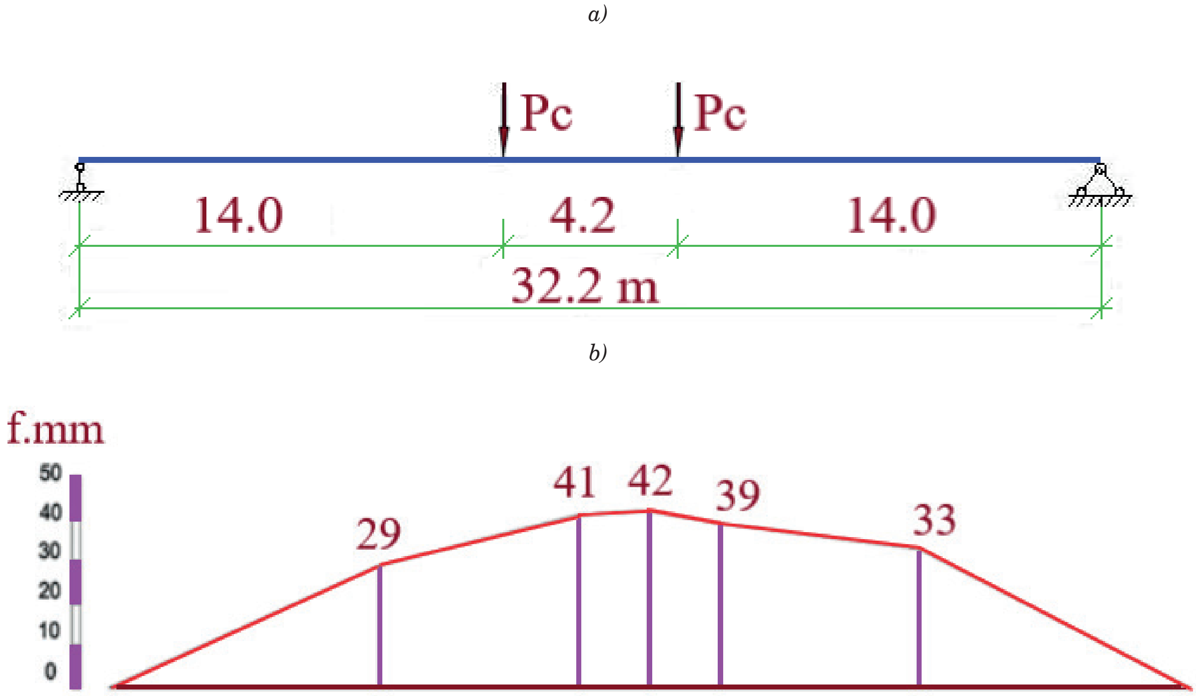

Figure 14 *The design scheme of a 32.2 m long composite beam (a) and the outline of its lower face, loading devices weight included (b)*

Table 3 *Concrete beams experimental grades*

Item No.	Structure name	Number of experimental single values, n	Average value of concrete strength $\hat{R}$, MPa	Standard σ, MPa	Variation coefficient, v	Experimental concrete grade, B
1	2	3	4	5	6	7
1	VTK-33u	34	46.47	5.397	0.116	37.33

beam concrete was determined. The strength of concrete was evaluated by the shock pulse method, as per GOST 22690-88 "Concrete" standard. Determination of strength by mechanical methods of non-destructive testing" [17] using an electronic concrete strength meter IPS-MG4.03, developed by SKB Stroypribor LLC (Chelyabinsk, Russian Federation).

Table 3 presents the processing results of the concrete strength experimental values. The experimental grade of the beam concrete at the day of testing made B37.33. The concrete strength corresponds to the nearest B35 grade.

4.2 Beam testing

Control (certification) tests of the reinforced concrete structures are carried out according to the schemes provided for in the design documentation, both before the start of mass production of structures and periodically in the process of their manufacture.

A certified power stand of Magnetik LLP was used for testing. To create and control the magnitude of the vertical load during the testing of a beam, a power plant was used, a hydraulic jack DP140G300, a pressure gauge, high pressure hoses and a pumping station included.

The estimated length of the prototype, adopted in the tests was 32.2 m, i.e. the axes of the supporting parts were located at a distance equal to 0.4 m from the ends of the beam. In the middle part of the span, at a distance made 2.1 m from its middle, the test load on the prototype was applied in the form of the two concentrated forces P. The test scheme for the VTK-33u beam corresponded to the scheme adopted in the project.

During the tests, the deflections of the beam were determined in the middle of the span, under two loads P and at a distance of 8.05 m from the axis of the beam supports. Beam deflections were recorded using deflectometers of Aistov system and Kucherenko Central Research Institute of Building Structures (Moscow, Russia). The subsidence of the supports was controlled using dial indicators with a division value of 0.01 mm.

In the course of testing, the possible slippage

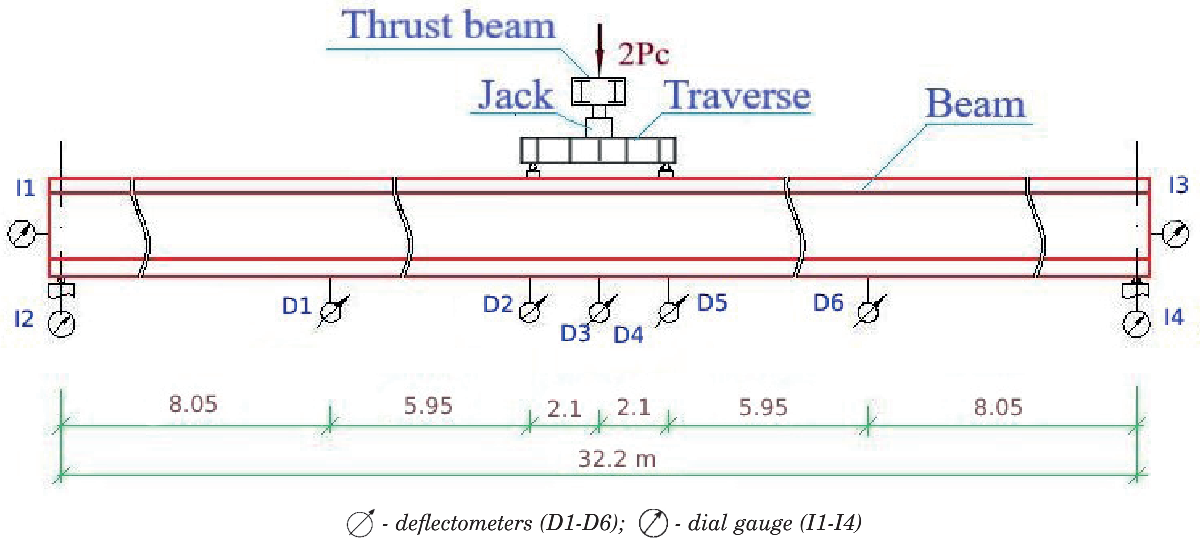

Figure 15 *Experimental product scheme with the loading equipment and mechanical devices*

Figure 16 *General view of the power stand and the VTK-33u tested beam*

Figure 17 *The support zone of the VTK-33u beam with dial indicators*

of prestressing reinforcement beams relative to the concrete of the beam was controlled using dial indicators with a division value of 0.01 mm, installed at the ends of the beam. Figure 15 shows the design scheme for testing the VTK-33u beam and the scheme of an experimental product with loading devices.

To control the moment of a crack formation, the side surfaces of the slab in the middle of the span were additionally covered with a thin layer of lime solution. The crack extension width was determined using a Brinell microscope.

A phased load was applied to the experimental beam. After each phase of loading, readings were taken on deflectometers and indicators.

Figure 16 shows a general view of the beam being tested and the certified power bench during the testing and Figure 17 shows the support area of the beam with dial gauges.

4.3 Test results

According to Table 39* [16] the category of crack resistance requirements, imposed on the VTK-33u bridge beam, is - 2b, in which the formation of cracks is allowed under the action of the design loads.

The control loads for testing the pilot beam were determined in accordance with the requirements of GOST 8829-94 [18] and [19].

According to the project, the experimental deflection

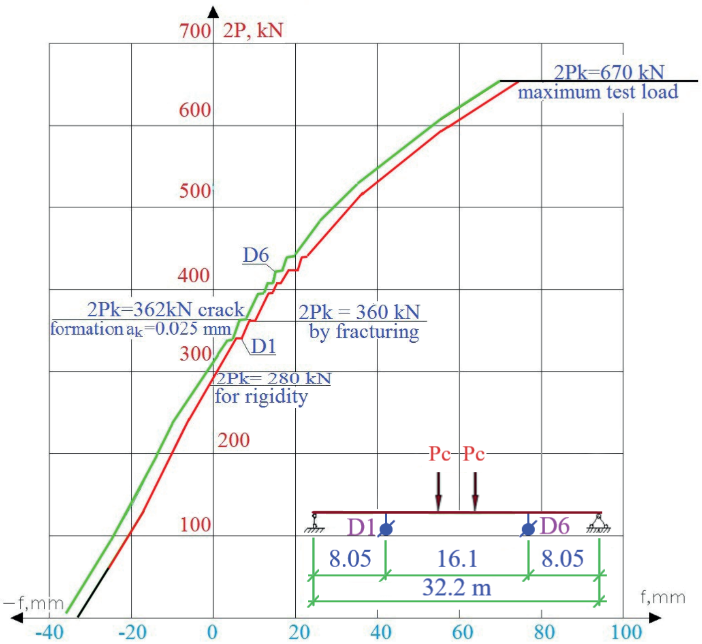

Figure 18 *Deflection curves at a distance of 8.05 m from the middle of the span: D1 and D6 - deflectometers*

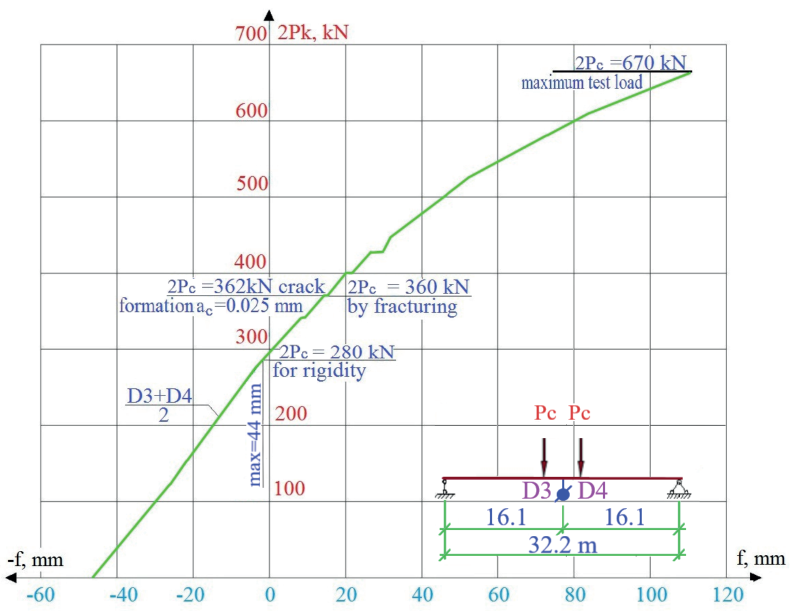

Figure 19 *Mid-Span Deflection Graphs: D3 and D4 - deflectometers*

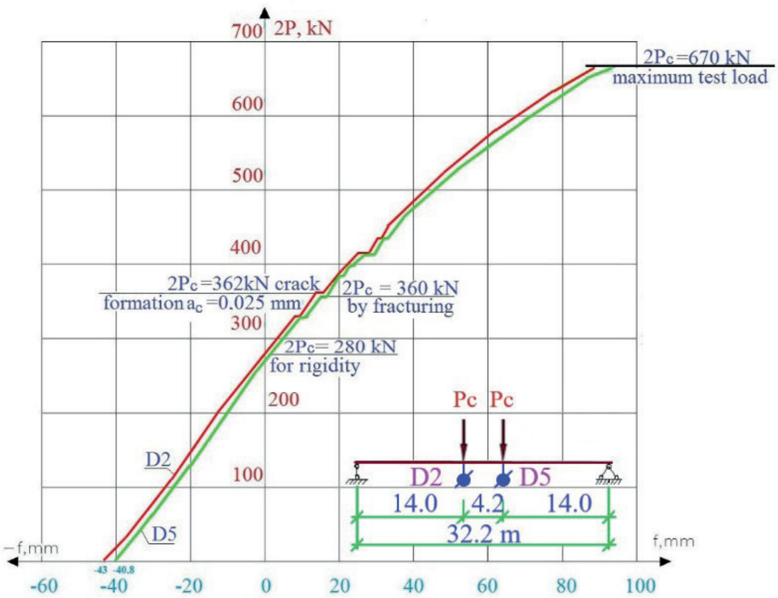

***Figure 20** Deflection curves at a distance of 2.1 m from the middle of the span: D2 and D5 - deflectometers*

value, under the action of the control load on stiffness, should not exceed the value equal to f_c = 59.2 mm and the crack extension width, under the action of the control load on crack resistance, should not exceed the value equal to a_{cr} = 0.09 mm.

Figures 18, 19 and 20 show graphs of deflections during testing of the VTK-33 bridge beam.

At the first stage of testing, the stiffness of the beam was evaluated. With a control load in terms of rigidity equal to $2P_c$ = 280 kN, the control deflection of the beam should not exceed a value equal to f_c = 59.2 mm. Upon reaching this load, the experimental deflection in the middle of the span of the beam had a value equal to f_{exp} = 44.0 mm, which made 74.3 % of the control deflection.

At the second stage of testing, the crack resistance of the beam was evaluated and the crack extension width was controlled. One of the main indicators, characterizing the reliability of the bridge beams, is their crack resistance, since this primarily affects their durability and, ultimately, strength. According to the design, with a control load of $2P_c$ = 360 kN, the experimental crack extension width should not exceed the control value equal to a_{cr} = 0.09 mm. When the control load 2*Pc* = 360 kN was reached during the testing, there were no cracks in the concrete of the beam.

At the third stage of testing, the strength of the beam was evaluated. When checking the strength of the beam, the ultimate load, at which the bearing capacity of the experimental structure is estimated, is the control load equal to the value 2 P_c = 630 kN. At a load of 2 P_c = 362 kN, a crack was formed with an extension width a_{cr} = 0.025 mm. With further loading and reaching a load equal to 2 P_{exp} = 432.9 kN, the experimental width of the crack extension did not exceed the value a_{cr} = 0.09 mm. An assessment of the stress-strain state of the VTK-33u beam at a load value of 2 P_c = 670 kN, indicated that the limit state had not been reached in it and the experimental design had a reserve of the bearing capacity.

During the tests, the possible displacement of bundles of prestressing reinforcement, relative to the concrete of the beam, was monitored. At all the stages of loading the beam, there was no displacement (pulling) of the bundles of prestressing reinforcement relative to the concrete of the beam, which indicated the reliability of the working reinforcement coupling with concrete.

Experimental tests of the reduced model of reinforced concrete beam from 24 m to 18 m are given in [19]. The research was carried out in cooperation with Prefa Sucany, Inc., the University of Zilina and Projstar PK, Ltd. As a result of the tests the sufficient load-bearing capacity of the beam was confirmed for the design load.

5 Calculation of the span in the finite element program "Lira"

The accepted design scheme of the bridge deck is an idealized mathematical model, which is used in the finite element method (FEM), in which a building structure is represented as a set of rod, plane or volume finite elements having common points - nodes. The more finite elements the computation scheme has, the more accurate the results will be.

In the considered case, the calculated span of the VTK-33u girder is 32.2 m. The span was divided lengthwise into 20 equal parts - 1.61 m each.

In this example, the reinforced concrete span is considered without considering the reinforcement. This can be explained by the fact that the difference between the given geometrical characteristics of the reinforced concrete section and the geometrical characteristics of the concrete section in determining the forces has little effect on the accuracy of the forces. However, this greatly simplifies the input of initial data and speeds up the calculation time.

After assigning the type of stiffness to the plates and rods, the scheme of the span becomes physically meaningful (Figure 21).

The slab does not go on top of the beams because in the calculation scheme, i.e. in the mathematical model in Lira, all the elements are combined by their centers of gravity (Figure 21). The flanges of the beams are not actually joined together in the computational scheme. This is a conditional simplification of the computational scheme - the joint work of the beams in the transverse direction is ensured by the plate passing through the centers of gravity of the beams. This will have absolutely no effect on the accuracy of the calculation (determination of forces in the beams). The span slab redistributes the load between the beams: it will be affected by the temporary load and the load from the bridge deck.

The next step is to set the boundary conditions, i.e. the span support conditions. To calculate the forces in the superstructure elements, it is not necessary to specify the supports precisely enough (i.e. take into account elastic properties of rubber-metal bearing parts; this can be considered when calculating the bridge abutments). In this case, it is assumed that at one edge there are linear-moving support parts and at the other edge - linear-non-moving ones.

The results of the deflections calculation in the three stages of the span test are shown in Figures 22-24.

The criterion for the positive performance of the overpass spans is the design factor determined by the formula:

$$K = \frac{f_e}{f_{cal}}, \tag{1}$$

where:

f_e is the experimental deflection in the beam from the test load;

f_{cal} - design deflection in the beam from the test load.

Table 4 shows the results of acceptance tests of the overpass.

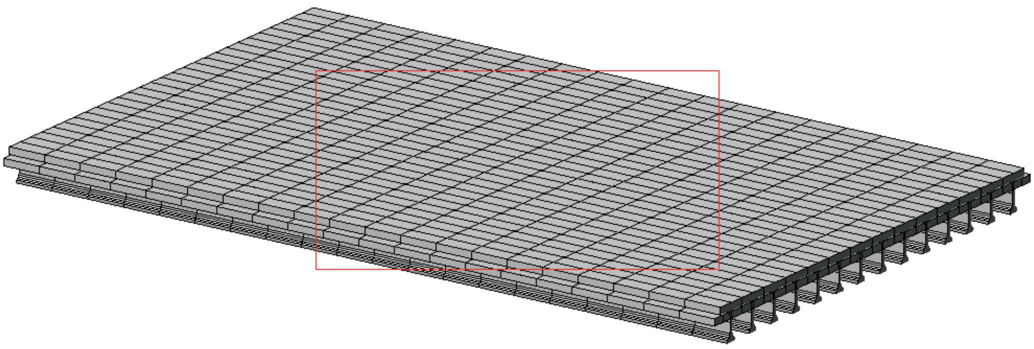

Figure 21 *Spatial model of the bridge span*

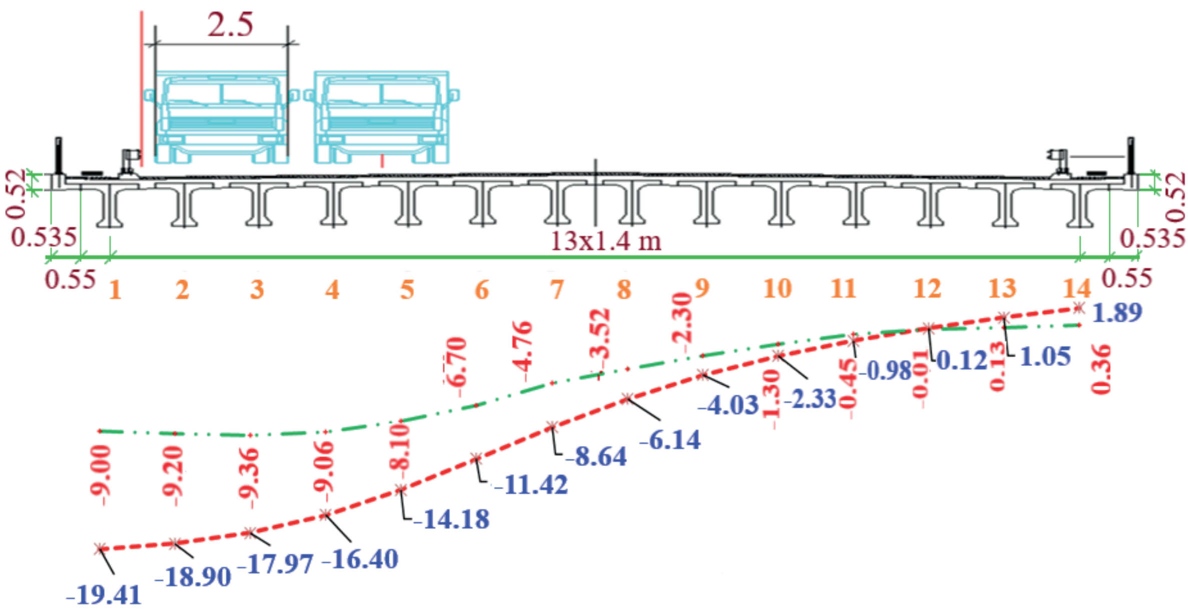

Figure 22 *Deflections in the beams of the span in the first stage [1]:*
-··- maximum deflection according to the test results, - - - design deflection

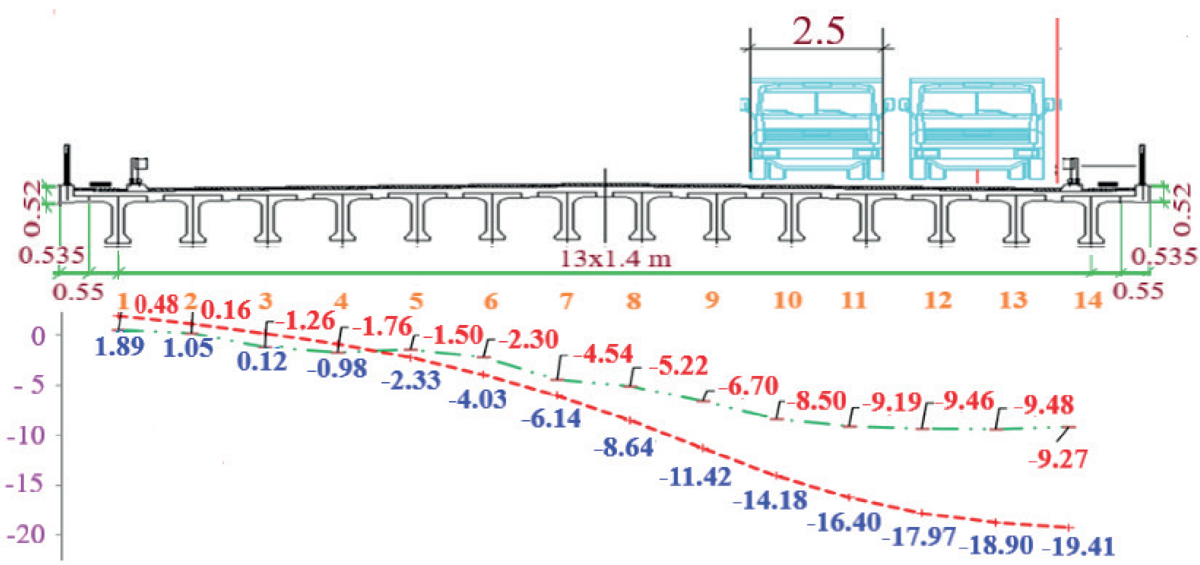

Figure 23 *Deflections in the beams of the span in the second stage [1]:*
-··- maximum deflection according to the test results, - - - design deflection

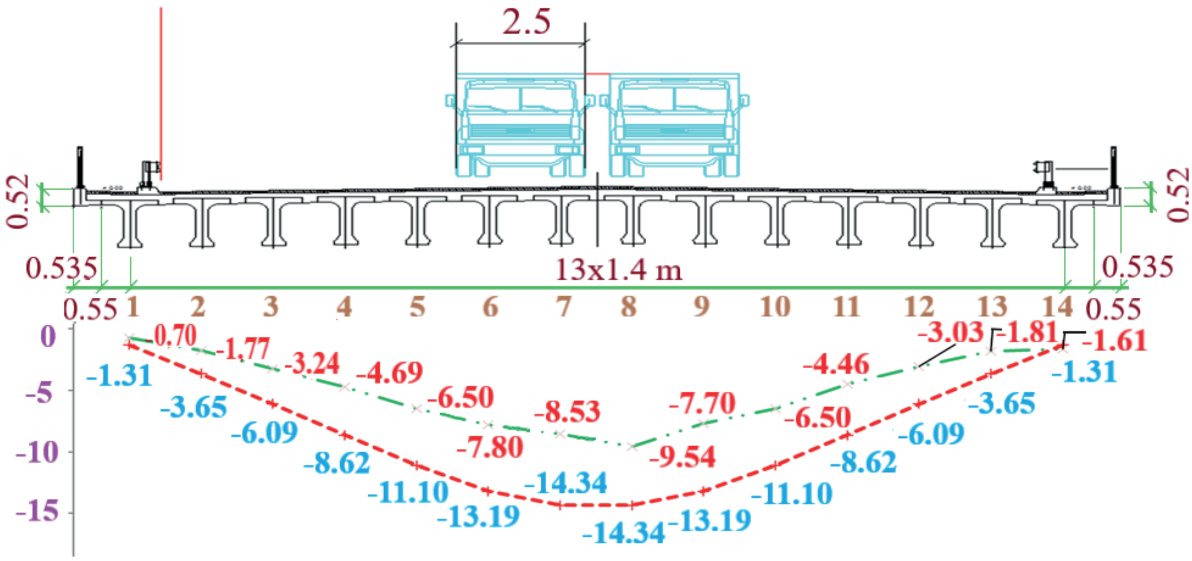

Figure 24 *Deflections in the beams of the span in the third stage:*
-··- maximum deflection according to the test results, - - - design deflection

Table 4 *Values of experimental and design deflections and the design coefficients*

Overpass	Actual maximum deflection, f_e, mm	Calculated maximum value of deflection, f_{cal}, mm	Design factor, K
First stage of tests	9.36 (Figure 23)	17.97 (Figure 23)	0.52
Second stage of testing	9.48 (Figure 24)	18.90 (Figure 24)	0.50
Third stage of tests	9.54 (Figure 25)	14.34 (Figure 25)	0.67

According to the results of static tests, the value of the structural coefficient K, according to [18] for the elements of the spans, in which the calculations do not take into account the joint work of the main beams with the elements of the roadway and pavement, as a rule, is from 0.5 to 0.7.

The obtained values of structural coefficients indicate that the tested span structure works according to the design models accepted in the project.

Theoretical aspects and numerical implementation of the stress sensitivity analysis of a beam, centered on the cross-section parameters and implemented in the Matlab package are presented in [20].

The article [21] presents the results of modeling of the concrete and rod elements, reinforcement by the three-dimensional finite-element models taking into account the crack opening. The calculation was performed by the finite element method in the SCAD program. The method made it possible to obtain values of the crack opening, taking into account the work of longitudinal reinforcement located in the tensile and compressed zones of the structure and transverse reinforcement. These results are more difficult or practically impossible to obtain in the calculation according to the existing standards.

6 Conclusions

1. A close inspection of the VTK-33u beams of the superstructure No. 1 of the Almaty-Kapshagay highway, damaged by vehicle impacts, was carried out.
2. In beam 1, a rib is destroyed, the lower part is widened and breaks in wires and K-7 strands are found. Beam 1 was not subjected to repair and was dismantled.
3. The superstructure beams were repaired. Given the responsibility of performing this type of repair work, especially the injection of beam into cracks, it was entrusted to an expert organization with a prior license.
4. Beams 5, 6, 7, 8, 9 and 10 were repaired by restoring the widened lower part with EMACO FAST TIXO, manufactured by BASF.
5. In beams 2, 3, 4, 11, 12, 13 and 14, along with the destruction of the lower part, longitudinal cracks of various lengths were also found, followed by significant decrease of their bearing capacity. Those beams were completely dismantled and replaced by the new ones.
6. An experimental bridge beam VTK-33u was manufactured using the bench technology at the production base of Magnetik LLP (Zarechny village, Almaty region, Kazakhstan).
7. The control load upon examination of the beam stiffness was 2 R_k = 280 kN. With a given load, the experimental deflection should not exceed the control deflection equal to f_k = 59.2 mm. With a control load equal to 2P_c = 280 kN, the experimental deflection in the beam had a value equal to f_{exp} = 44 mm, which amounted to 74.3 % of the control deflection equal to f_c = 59.2 mm.
   The stiffness grade of the VTK-33u bridge beam complies with the requirements of GOST 8829-94 standard, the project and norms of Construction Directives and Rules 2.05.03-84* “Bridges and pipes”.
8. The control load upon examination of the VTK-33u bridge beam crack resistance was 2 P_c = 360 kN. Under this load, the experimental crack extension width should not exceed the control value equal to a_{cr} = 0.09 mm. When the control load of 2 P_c = 360 kN was reached during testing, no signs of cracks are found in the concrete of the beam.
   The crack resistance of the VTK-33u bridge beam complies with the requirements of GOST 8829-94 standard, the project and norms of Construction Directives and Rules 2.05.03-84* “Bridges and pipes”.
9. The control load upon examination of the VTK-33u bridge beam strength was 2 P_c = 630 kN. An assessment of the stress-strain state of the VTK-33u beam at a load value of 2 P_c = 670 kN indicated that the limit state had not been reached in it and the experimental design had a reserve of the bearing capacity.
   The strength grade of the VTK-33u bridge beam complies with the requirements of GOST 8829-94 standard, the project and norms of Construction Directives and Rules 2.05.03-84 * “Bridges and pipes”.

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