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ATYPICAL EXAMPLE OF THE REDUCTION OF THE SAFETY AND DURABILITY OF A TYPICAL MOTORWAY OVERBRIDGE

NIETYPOWY PRZYPADEK ZMNIEJSZENIA BEZPIECZEŃSTWA I TRWAŁOŚCI TYPOWEGO WIADUKTU NAD AUTOSTRADĄ

A b s t r a c t

The present paper is related to the motorway overbridge, the technical state of which has been showing deficiencies related mainly to the presence of cracks. Verification of the design and construction documents was carried out prior to advanced analysis which covered the construction process with phases, loads acting in various periods of time as well as the progressing ground deformations. Conclusions drawn on the basis of the analysis results are not typical because the emergency situation is resulting from many simultaneously acting factors. The level of the reduction of the safety margin of the bridge has been estimated.

Keywords: *prestressed concrete, emergency state, analysis, finite elements method, prestressing technology, settlement, safety, durability*

S t r e s z c z e n i e

Opracowanie dotyczy wiaduktu nad autostradą, którego stan techniczny wykazywał nieprawidłowości związane głównie z obecnością zarysowań. Przeprowadzono weryfikację dokumentów z czasu budowy obiektu oraz zaawansowane analizy obliczeniowe obejmujące proces budowy, panujące obciążenia, a także postępujące odkształcenia podłoża. Wnioski wysunięte na podstawie analiz mają nietypowy charakter, ponieważ awaryjny stan ustroju jest wynikiem splotu wielu jednocześnie występujących czynników. Zmniejszenie zapasu bezpieczeństwa oraz trwałości obiektu zostało oszacowane.

Słowa kluczowe: *beton sprężony, awaria, analiza, metoda elementów skończonych, technologia sprężenia, osiadanie, bezpieczeństwo, trwałość*

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1. Introduction

The present paper is related to a bridge structure located at a G2 class main road axis over a motorway in the south of Poland (Fig. 1). The bridge built over the motorway is composed of two parallel, load-bearing structures. The cross-section of each of them is composed of three girders and there are six spans along the length of the bridge. The angles of intersection with supports and abutments axes are variable. The bridge is built in an area of geologically non-stable ground – this fact was already known at the time of construction. In effect of the ground settlements, redistribution of internal forces in the load-bearing structure was possible and this effect could provoke the structure overloading.

Fig. 1. View of the structure

On the basis of the owner's reporting of the presence of visible damage to the surface of the bridge members, complex investigation of the load-bearing structure was carried out. An advanced level of damage was observed, in particular represented by cracks on the surface of the structure.

The question of the reduction in bridge safety levels as an effect of progressive damage is raised frequently. The usual description of the problem is related to the corrosion process (e.g. [1]). The case which is the subject of the present research includes a substantial influence of nonconformities generated at the time of construction as well as related to external impact – for these reasons, it is more complex in comparison to the usual examples.

The aim of this work is to present: observed irregularities in the technical condition of the structure; the performed tests and analyses; conclusions relating to the possible origins of the existing damage; the influence of the damage on structural safety and durability. Additionally, a description of the structure is presented, the observed nonconformities and damage, and the results of the evaluation with regard to safety.

2. Description of the bridge and its history

A continuous six-span viaduct made of prestressed concrete with a total length of 180.88 m; its transversal cross-section consists of three main girders of a trapezoid cross-section joined with a deck slab of 0.25 m. The width of the structure reaches 13.70 m. The depth of the main girder is constant at 1.50 m, while the width varies from 1.20 m to 1.80 m from the bottom edge to the top. The axial spacings of the main girders are 4.30 m.

Girders are stiffened by the transverse beams located in the midspans as well as over the piers. Slab cantilevers are designed at the side girders and their length reaches 1.6 0 m. Length of the spans equals from east to west: 27.0 m, 29.22 m, 29.22 m, 33.25 m, 33.25 m and 27.0 m. Two angles of intersection between the main structure and the abutments line are different and their values are 49.84° and 60.83°.

The load-bearing structure is supported by column piers and abutments. All supports are based on reinforced concrete piles with a diameter of 1500 mm.

The bridge includes standard features: SMA runway pavement of the regular thickness; modular expansion joints; concrete pavement slab covered with an epoxy surface insulation; stone corbels; barrier and guard-rail. With consideration to the structural work, attention was paid to the fixed bearing position at the bridge abutment.

The bridge was designed in the end of nineteen-nineties, and its construction was executed in the years 2001–2003. Only a portion of the archived construction documents are available, the preserved documents are mainly related to the prestressing operation. These circumstances were considered as a reliable reference which, with additional assumptions described further, could be used for an analytical approach to the structure load-bearing capacity.

The design documents subjected to investigation allow the assumption that during the construction period, one of the principal difficulties was related to the state of the ground. At various points in the documents, expressions related to the operation of the ground compaction (consolidation) are present. These circumstances influenced the decision to change the solution used for the foundation of the piers, initially designed as supported directly on the concrete footing. The construction schedule of subsequent parts of the structure was also subjected to change. Initially, it was assumed that the bridge would be built in five phases starting from the eastern abutment, finally both, – the final construction drawings as well as the construction documents describe the progress of works starting from the western side. Additionally, initial analysis of the construction design and documents revealed an important difference between the values of the permanent prestressing forces in the design at the building permit phase and the design in the executive phase – in the other documents, the values of the forces are around 30% lower than in the first ones. The modifications introduced also influenced the details of the execution of prestress – it was decided that the grouting of cable ducts located in the bridge sections built in phases 4 and 5 would be carried out in one step.

3. Testing of the structure

The structure was subjected to tests aimed at evaluating the extent of damage. These tests were performed due to irregularities in the technical state of the bridge, particularly a significant settlement of the runway in the immediate proximity of the bridge as well as the advanced cracking morphology of the beams.

The measurements of the runway presented a significant unevenness in the surface shape concentrated in particular in the road section approaching the bridge from the west. As a result of the settlement, a 6 cm ramp for vehicles arriving on the bridge from west was present. In the analysis, the dynamic factor for the loads was assumed to be at its utmost level.

In order to assess the state of the bridge in the area of the observed cracks, several tests were carried out. In the main program of testing, drilled cores were collected, passing through the observed cracks in the side beam of the side span from the west. The first of the cores was cut through the upper crack of 1.5 mm; the second, through the lower crack of 0.8 mm. The drill through the first of the crack delivered the following information:

– 15 cm from the surface, concrete cover was splitted by a vertical plane of crack,
– full inspection of the prestressing tendon duct in the area adjacent to the coupling anchorage in the joint of the segments was possible by means of opening the conical shield of the cable coupling anchorages.

Within the framework of the investigation, the quantity and positioning of the reinforcement were also verified, followed by checks of its state and the cover thickness.

Apart from destructive testing in the form of drilled cores, several other types of tests were conducted, both destructive as non-destructive. Tests were carried out by two independent laboratories. Cylinders, the diameters of which were 100 mm, were cut from the investigated bridge span and were used to assess the compressive strength and the secant modulus of elasticity. The strength results obtained for the tested cylinders were 60.7 MPa, 70.9 MPa, 70.3 MPa and 52.6 MPa. These values confirm that the concrete strength in the bridge was substantially higher than the strength related to the B40 concrete class assumed in the design (class description following the bridge code of the time [2] and laying between C30/37 and C35/45). Similar results were obtained with the pull-out test (results reported are from 40.5 MPa to 84.7 MPa) and with sclerometric tests (concrete class estimated was C40/50 or C45/55). Concrete splitting resistance, measured with the pull-off method, reached slightly above 2.0 MPa – this refers to the tensile resistance for class C20/25. On the other hand, measurements of the modulus of elasticity demonstrated that not all characteristic parameters of concrete are increased in a similar way. The value of the modulus of elasticity complies with the requirements of the typical values for class B45 (C35/45) only. Similarly, volume weight of the concrete collected from the structure proved to be 7% lower than the weight of typical concrete with basalt gravel. The measured properties of concrete present an important scatter.

In addition to the strength tests, chemical analysis was also carried out – the results confirmed the alkaline property of concrete with the pH value exceeding 11.

4. Description of the structural damage

Clearly visible cracks served as the fundamental reason to undertake the tests and evaluation of the technical state of the structure. In the western side span of the bridge, the south girder was the most damaged. Numerous horizontal and skew cracks were concentrated mainly in the area located more than 5.0 m from the support towards the abutment. The area with a high concentration of cracks is located directly at the construction joint, which is 5.0 m from the support. Fig. 2 presents the crack distribution over the surface of the beam together with the position of the cable ducts and the joint between the construction segments built successively.

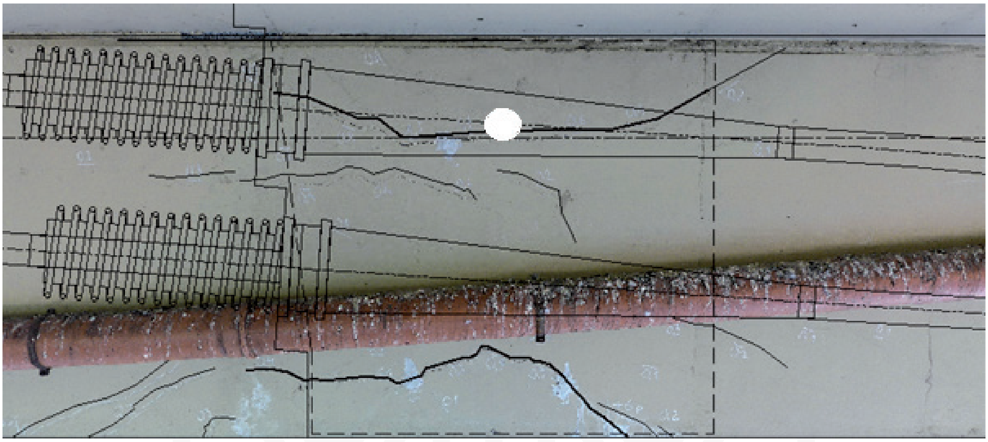

Fig. 2. Layout of cracks on the beam surface and the position of the joint between the structure segments and prestressing cable ducts

Due to the large width of cracks observed on the member's surface, the decision was taken to drill a core specimen along one of the cracks. The aim of this drill specimen was to investigate the depth and direction of the crack. The drilling was positioned at the level of the axis of the upper prestressing tendon, where the diameter of the cable duct progressively changes from 275 mms at the coupler to 100 mms at the entrance to the regular cable duct. In Fig. 2, the position of the drill is shown (indicated with a white dot). Because the crack width (Fig. 3) observed in the direction perpendicular to the surface was not decreasing, the drilling continued until the steel cover of the cable duct was reached. Inspection of the drill surface revealed the presence of another crack perpendicular to the drill axis and located 80 mm below the concrete surface. This crack is visible in the upper part of the drilled wall, slightly before the steel shield of the cable duct (Fig. 4). The observation of the crack inside of the drill specimen is in line with another observation of the concrete cover spalling – this was confirmed by means of an acoustic check of the concrete surface using a metal hammer.

Fig. 3. Crack position in the drill wall of the beam under investigation

Fig. 4. View into the drilled hole in the girder subjected to investigation. The steel cover of the prestressing cable duct and its interior are visible

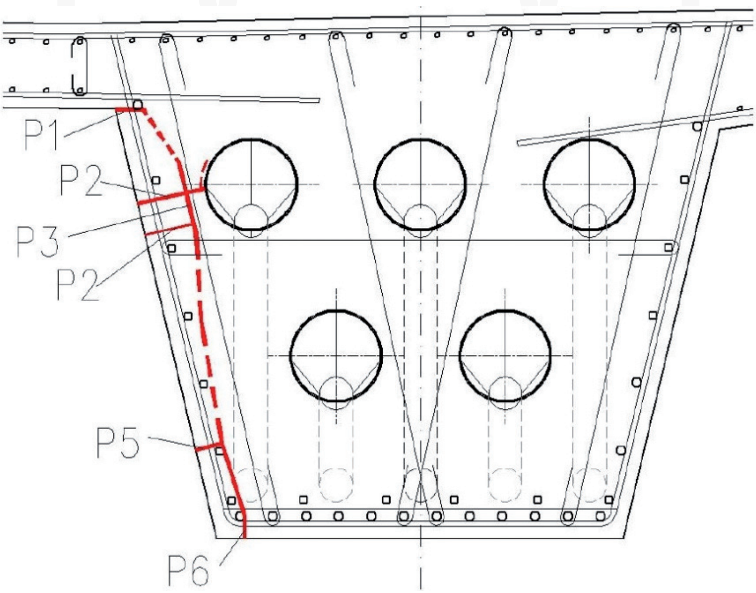

Fig. 5. Spalding of concrete along the depth of the girder – the continuous line represents the observed cracks, the dashed line represents their hypothetical trajectory. P1 to P6 – observed locations of cracks

The acoustic check indicated that the spalled layer occurs on the whole vertical edge of the beam from the slab at its top to the bottom edge and the length of the damaged area along the beam exceeds 2.5 m. The position of the observed splitting and its hypothetical trajectory are shown in Fig. 5.

The existence of the mesh cracking of concrete was observed in several zones of the bottom surface of the girders independently from the damage observed in the side span. The width of the cracks in these areas was measured at 0.1 mm and their directions were both parallel and perpendicular to the member axis. The layout of the cracks is similar to the distribution of the reinforcement in the girders. Verification by means of hammering has shown that the depth of the cracks reached 5–10 mm. The area of the widest cracks of this type from all surfaces of the girders was observed in the fourth span. In all areas of the mesh fissures their width in both directions is similar. It is suprising because prestressing of the girders introduced in the longitudinal direction has no influence on the cracks width although it should be expected that the prestress should force the perpendicular cracks to close.

Cracks of smaller widths (less than 0.05 mm) are observed on the girders' surface in many areas. Their presence may result from concrete shrinkage as well as thermal deformations of beams at the time of the concrete hardening. A partial explanation of their presence may be related to the diameter of the main reinforcing bars (28 mm) as well as the low cover thickness reaching only 35 mm.

Observations regarding materials inside of the structure in the area of drilled cores also brought many interesting observations. After penetrating the cable duct shield, it was found that instead of the duct being completely filled with cement grout as expected, there was a substantial lack of grout (Fig. 4). Punctual corrosion of the stressed, exposed prestressing strands was also found inside of the duct. The mortar filling the bottom part of the prestressing cable duct was non-cohesive – it was in the form of a powder with a grain size of 1–4 mm and with a white-beige colour. The lack of grout in the proper form, local corrosion of the prestressing strands as well as the corrosion points on the surface of the cable duct shield are evidence of the risk of tendon corrosion. The interior of the cable duct is shown in Fig. 6. Naked strands, local corrosion of the prestressing steel as well as of the internal surface of the duct shield was observed. Moreover, concrete splitting from the external surface of the cable duct shield was also observed (Fig. 4).

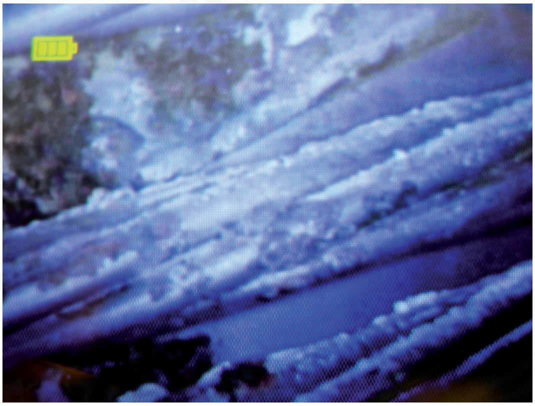

Fig. 6. View of the interior of the prestressing cable duct

Although initially it was considered that the position and corrosion levels of the prestressing cables be verified by means of projection [3], based on the relatively good condition of the prestressing cables and the existing cement grout in the ducts in other locations and cables, the concrete investigation was limited to drilling.

Subsidence is a major problem for the whole structure. The geodetic measurements which are available for the entire eleven-years history of the bridge prove large settlements values and their important relative differences. The west abutment is the most exposed to the effects of subsidence and as a consequence, this support settled by 55.5 mm. Table 1 shows the medium settlement values for the supports of the bridge at the end of the monitoring period. The largest observed relative settlement is more than five times greater than the usual design assumption of 10 mm. For this reason, the influence on the structural safety must be investigated.

Table 1

Total settlement values of the bridge piers over 11 years of service

<table>
<tr><td rowspan="2"></td><td colspan="7">Support number</td></tr>
<tr><td>A</td><td>B</td><td>C</td><td>D</td><td>E</td><td>F</td><td>G</td></tr>
<tr><td>Mean settlement [mm]</td><td>55.5</td><td>12.8</td><td>15.6</td><td>19.7</td><td>6.2</td><td>5.0</td><td>3.3</td></tr>
<tr><td rowspan="6">Relative settlements [mm]</td><td colspan="2">42.7</td><td></td><td></td><td></td><td></td><td></td></tr>
<tr><td></td><td colspan="2">2.8</td><td></td><td></td><td></td><td></td></tr>
<tr><td></td><td></td><td colspan="2">4.1</td><td></td><td></td><td></td></tr>
<tr><td></td><td></td><td></td><td colspan="2">13.5</td><td></td><td></td></tr>
<tr><td></td><td></td><td></td><td></td><td colspan="2">1.2</td><td></td></tr>
<tr><td></td><td></td><td></td><td></td><td></td><td colspan="2">1.7</td></tr>
</table>

Although the support deformations are relatively large, their overall distribution only has a slight influence over the internal forces in the bridge girders. This is due to the relatively low bending rigidity of the main girders. Nevertheless, this influence is included in the presented results.

The damage and irregularities of the structure subjected to analysis are of various origins; however, they simultaneously influence its durability and safety. In order to assess the aggregate influence of the observed deficiencies, complex analysis of the structure was carried out as described below.

5. Varying safety margin for the bridge

5.1. Limit states verifications of the load-bearing structure according to the design phases

Prestressing force reduction

In the framework of safety verification of the structure, detailed analysis of the available documents relating to the bridge was carried out. All of the assumptions made as well as the static model formulation were declared to be correct. One of the principal observations based on the data included in the documents is related to the values of the prestressing forces. The notional tensile resistance of the 19T15 prestressing cables used in the design is:

$$F_d = f_{pk} A_p = 1860 \text{ MPa} \cdot 19 \cdot 1.50 \text{ cm}^2 = 5301 \text{ kN}$$

The generally governing admissible level of maximal stressing force reaches 80% of this value:

$$P_{0max} = 0.80 \cdot 5301 \text{ kN} = 4240.8 \text{ kN}$$

In the design at the phase of the building permit the permanent prestressing forces (after all losses) were estimated and the final stressing level of the tendons was assumed at the level of 59% to 68% of the admissible stress level (see Tab. 2). Such low tension was resulting from the decision of a relatively low initial stress level in prestressing cables as well as the friction losses in cables stressed only at one end. Nevertheless, the construction drawings created at the next step of the project process – at the final phase of design – provide the values of the permanent prestressing forces reaching 41% to 49% only of the admissible stressing forces (also see Tab. 2). In accordance with the values given in the construction drawings, prestressing procedures were formulated and approved; furthermore, tensioning of cables was carried out. Considering the lack of both substantiation for such prestressing force reduction as well as of controlling evaluation of its influence on the load-bearing conditions, a check-up calculation of the bridge girders was undertaken.

Table 2

Permanent prestressing force in the spans in the initial and construction design

Span No.	Final force (initial design) [kN]	Final force (construction design) [kN]
1	2513	1745
2	2427	1781
3	2873	1781
4	2493	1787
5	2813	1788
6	2480	2032

A lower value of prestressing forces in the bridge structure, which could potentially result from a misunderstanding of the design provisions from the first phase, has an influence on the static effect of prestress. It was estimated that proportional to the force decrease, the bending moments as well as compression forces provoked by prestress are 19% lower than initially assumed. As a result, the safety margin for normal stresses in cross-sections is reduced and in many sections of the girders, cracking of the concrete at the bottom surface of the spans may appear. Moreover, lower prestressing forces influence the load-bearing capacity for bending. The principal conclusions regarding this problem are presented further in this paper.

Load-bearing capacity for bending

The varying safety margin for the member regarding its load-bearing capacity for bending was investigated in accordance with the Eurocode 2 approach [4]. Table 3 gives the maximal bending moment resulting from the loads. The influence of the force decrease is presented for span 6, which is where the damage was observed.

Table 3

Maximal bending moment for the final situation

Span/Support No.	Maximal bending moment [MNm]
1	8.19
B	–11.62
2	7.91
C	–13.21
3	9.36
D	–12.61
4	8.13
E	–11.10
5	9.18
F	–11.76
6	8.08

In this evaluation, the influence of the supports settlement was also accounted for. Its general effect is positive due to the fact that the static bending moment is reduced as an effect of this deformation. The load-bearing capacity for bending estimated following the simplified approach – from the equilibrium conditions – equals M_{Rd} = 10.63 MNm. The relative decrease of the capacity estimated with the general approach (following the strain diagram in cross-section) was showing 10.3% reduction of the initial value. In effect, the safety margin for the member cross-sections drops from 31.6% to 18% and this has a substantial influence on the overall safety of the bridge.

Load-bearing capacity for shear

In the same way as for the bending moment the safety margin for shear was reduced. The area of damage concentration is located in the area of the contact between the two parts of the bridge which were the last to be constructed. The initial capacity for the shear force was estimated for the non-cracked phase according to the expression given in EC2:

$$V_{Rd,c} = [(0.18/\gamma_c)\ k(100\rho_l f_{ck})^{0.333} + 0.15\sigma_{cp}]b_w d$$

With the assumption of the parameters taken from the bridge design, the value of:

$$V_{Rd,c} = 1265 \text{ kN}$$

is reached. This value is compared to the maximal shear force obtained from the static calculation, reduced in effect of prestressing force action:

$$V_{Ed} = 1430 \text{ kN} - 2480 \text{ kN} \cdot \sin 11.1° = 952 \text{ kN}$$

The supports' relative settlements are also included in this result. The above values are typical for a regular PC structure, where shear is not bringing any risk of collapse and even cracking in shear has a safety margin of 33%.

A lower value of the prestressing force results in both a reduction of the shear load-bearing capacity and an increase in the static force from loads:

$$V_{Rd,c} = 901 \text{ kN}$$

and

$$V_{Ed} = 1430 \text{ kN} - 2032 \text{ kN} \cdot \sin 11.1° = 1038 \text{ kN}$$

Shear force is 15.2% higher than the ultimate limit state load-bearing capacity for shear in the first phase. As a consequence of the lower prestressing force, shear may provoke cracks, although considering safety factors for loads and concrete strength concrete may resist the applied stress level. The remaining load-bearing capacity of the cross-section in phase II based mainly on the shear reinforcement is sufficiently high and reaches (with vertical stirrups composed of 6 bars of 16mm in the spacing of 12 cm) the level of:

$$V_{Rd,s} = 1430 \text{ kN}$$

Based on the evaluation results, it may be concluded that the shear force in cross-section is increased as an effect of lower prestressing force and on the other side, load-bearing capacity for shear is shifted from the first phase to the second phase (with cracks). This will have an significant influence on the durability of the member. In this area, an approach based on the durability model presented in [5] is included in the further plans of this analysis.

5.2. Analysis of the influence of damage on the load-bearing capacity of the structure

Regarding the location of the area subjected to damage as well as the type of the irregularities, it was recognised that they may negatively influence the load-bearing capacity of the beam under shear forces in particular. The lack of the cement grout in the cable ducts causes a radical decrease of the shear resistance evaluated in accordance with the present calculation methods. Moreover, the concrete cover splitted on the whole depth of the girder cannot be taken into consideration in the evaluation of its shear resistance.

The analysis in this scope was undertaken in agreement with the regulations of the Eurocode 2 [4] and was carried out for the cross-section situated directly at the interconnection of the bridge segments where the cable duct diameter is increased as a consequence of the size of the coupling anchorages. The compressive strength of concrete was assumed to be at its design value (ULS) as f_{cd} = 28.6 MPa and the mean stress provoked by prestress was calculated as σ_{cp} = 5.75 MPa. As a consequence of the detected lack of cement grout in the cable ducts, the width of the cross-section was reduced in accordance with the principles given in [4]. With the diameter of the coupling anchorages shield being 275 mm and the multiplication factor for the empty ducts at 1.2, the substitute width of the beam was reduced from 120 cm to only 21 cm. The reinforcement ratio of the longitudinal bars was estimated to be at a level of 0.37%. With the mentioned assumptions, the resulting shear force resistance, related to the concrete cracking, reaches a value of $V_{Rd,c}$ = 337 kN, while the shear force obtained from the static calculation is V_{Ed} = 952kN. By comparison with the values estimated for the non-damaged cross-section, it can be concluded that the importance of cracks on the girder safety is no more questionable.

Moreover, the value of the principal tensile stress in the skew direction was estimated – this is related to the risk of the skew cracks. Local stress values were considered on the assumption that the whole cross-section weakened by the presence of voids is acting in the force transfer. The principal stress reaches 70% of the concrete tensile strength. This gives evidence of an increased risk of exceeding the concrete strength and evidence of the appearance or development of cracks in the case of the cumulative action of various simultaneous loads on the structure.

6. Conclusions

The structure subjected to the investigation is substantially weakened as an effect of the combination of several independent factors. A decrease of the prestressing force causes a substantial reduction of the safety margin. The observed instances of damage have further influence on the load-bearing capacity. The settlement of supports does not negatively affect the overall balance of the internal forces. The investigation of the structural consequences with regard to safety is separately carried out for various limit states. Estimation of the structural durability decrease which is an evident effect of the damages is an important subject worth of further analysis.

Strengthening of the structure is necessary as is the elimination of the potential dangers. Due to the complexity of the case, an advanced model of the structure must be built in order to perform a detailed analysis of the multiple influences. Such a model will allow the estimation of the real margin of safety in a much more precise manner than that which is provided by the grid model. The construction of such model is also in the further plans related to this case.

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