Failure analysis and repair assessment of a steel box girder bridge

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Abstract. The paper focuses on the failure of a motorway steel box girder bridge. After almost entire bridge had been completed (excluding deck equipment), there was a few month break in its construction. During this break, in two support sections of the steel box girder, an unusual failure occurred comprising the significant deformation (buckling) and local cracking of the steel webs. In the paper, the analysis of failure causes and the assessment of how these failures impacted the stress distribution in box girders have been presented. After construction of the bridge had been resumed, the failures were repaired. The steel superstructure after repair was also subjected to detailed analysis regarding change of stress distribution in repaired girder elements. All analysis were carried out using advanced numerical FEM models, allowing to consider sequences of particular loads (including those not typical for bridges) and repair works. Conclusions drawn from these analysis concern both the proper shaping of the steel bridge structure as well as the methodology of assessing impact of this type of failure on stress distribution in the structure and effectiveness of the applied repair method.

1. Introduction

Studying past failures can be useful in mitigating the incidence and potential of future adverse events [1]. The causes of failure in bridges have been often divided into seven categories [2]. The first three are limited knowledge, design errors and other aspects of human error. The remaining four are natural hazards, accidents, overloading and deterioration. Design errors include cases where there was evidence of incorrect design assumptions, wrong estimation of loads, mistakes in calculations etc. Limited knowledge captures the cases where there was insufficient understanding of a failure mode at the relevant time, such as aerodynamic instability or a structural/material problem such as brittle fracture, fatigue or buckling. The remaining aspects of human errors are distinguished from design errors and cover those failures caused by negligence, ignorance, as well as poor workmanship, wrong assembly sequence, etc. Natural hazards encompass cases where failures have taken place due to extreme loading such as flooding, storms or very high winds. Accidents pertain to vehicle impacts, fire and explosions. Corrosion is the principal cause of deterioration in the case of steel bridge failures leading to collapse. However, in many cases, failure is induced by a combination of causes. In these cases, an attempt has been made to identify as far as possible the primary cause, and to classify the case accordingly. It has been also the case presented in the paper.

The paper focuses on the failure of a motorway steel box girder bridge. The analysis of failure causes and the assessment of how these failures impacted the stress distribution in box girders have been presented. The steel superstructure after repair was also subjected to detailed analysis regarding
change of stress distribution in repaired girder. All analysis were carried out using advanced numerical FEM models. Conclusions drawn from these analysis concern both the proper shaping of the bridge superstructure as well as the methodology of assessing impact of this type of failure on stress distribution in the structure and effectiveness of the implemented repair method.

2. Bridge description
The motorway bridge has total length of 212.0 m and consists of two independent parallel three-span structures with theoretical spans of 65.0 m + 80.0 m + 65.0 m. Both, north and south structures are practically the same, except for the total width of the deck, which is 18.95 m and 17.55 m in two parallel structures respectively. The superstructure of both bridges has a steel box girder and accompanying concrete deck slab acting compositely with a girder. The supports of the bridge are massive, reinforced concrete wall abutments and pillars, located parallel to the river with the angle of 80° to the bridge longitudinal axis.

The depth of the box girder is variable and ranges from 2.20 m in the mid-span to 3.50 m over the intermediate supports (Fig.1). The concrete deck slab has variable thickness ranging from 0.25 m to 0.56 m and is transversely prestressed with cables spaced at 1.0 m. The bottom plate and both webs of the steel box girder are stiffened with closed ribs and the transverse inner bracing is mounted every 4.0 m to prevent girder’s cross-section deplanation. Additionally, over the supports the double steel box cross-beams are located and the reinforced concrete bottom slab is cast along the negative moment’s section, connected with the steel bottom plate. The steel girder is made of S355J2+N mild steel. The deck slab is made of C35/45 concrete reinforced with B500A steel rebars and is prestressed transversely with Y1860 steel cables. The same concrete and reinforcement were used for casting the bottom slab.

After almost entire bridge had been completed (excluding deck equipment), there was a few month break in its construction due to the contractor’s bankruptcy and withdrawal. After that break had been terminated the detailed inspection of the existing structures was carried out by the succeeding contractor. In two support regions of the north steel box girder an unusual failures were detected comprising the significant deformation (buckling) and local cracking of steel webs just over the both pillars. Steel web cracking was placed around the rectangular section of cross-beams located on the other side of the web (Fig.2).

The analysis carried out by the authors aimed to re-check the limit states of the superstructure according to the codes used in the basic design to assess and confirm the bridge safety and to find failure causes, before the works were resumed. Additionally, the independent checking of the repair method proposed by the contractor was also provided.

![Figure 1. Cross-section of the bridge superstructure (north side)](image-url)
3. Re-checking of the superstructure
The first goal of the analysis was to check the safety of the superstructure according to the codes used in the basic design. The global FE models of the superstructure were developed in the **Sofistik** system (Fig. 3). The four-node shell elements were applied to model the box girder and its internal support cross-beams as well as the RC top and bottom slabs. The concrete reinforcement and transverse prestressing cables were also modelled in the subsequent concrete elements. The beam finite elements were used to discretize the longitudinal stiffening ribs of the webs and bottom flange as well as the box internal bracing. All finite elements had an assigned thickness or cross-section corresponding to the actual geometry of steel plates / shapes and the material parameters were assumed according to material testing results, obtained in the quality assurance procedures during construction.

The FE models were loaded with permanent loads (self-weight, equipment, prestressing, rheological effects etc.) and variable loads (live load, climatic effects) according to the Polish bridge standard used in the basic design. The actual phases of the bridge construction and their sequence implemented on-site were taken into account in FEM calculation, as follows: assembly of the steel members on temporary supports, removal of temporary supports after connecting members, casting the bottom slab, casting the deck slab and the final transverse prestressing of the deck slab. Concrete shrinkage and creep in the bottom and top slab were included in the particular construction phases. In the final phase of the evaluation the equipment weight and live loads as well as the environmental impacts were taken into account.

The global FE models were used for a linear analysis: the maximum values of internal forces were determined and then the basic ultimate limit states were checked according to the Polish bridge standards. Several load combinations were applied taking into account f.e. temperature effects. The equivalent stresses were determined based on the H-M-H theory and compared to the ultimate strength of the relevant material. Particularly, the entire steel superstructure was examined in order to check its...
appropriate strength and stability. As a result of the analysis based on FEM application, it was found that the superstructure was properly designed both during construction and in service condition. Thus any design error and limited knowledge case were excluded as the main causes of the detected failures. The remaining aspects of human errors as caused by negligence, ignorance, as well as poor workmanship, wrong assembly sequence, etc. had to be considered in searching the actual failure cause.

4. Failure analysis of the girder web

After excluding the design error and limited knowledge as the failure cause the supplemental inspection of the superstructure was undertaken to find another potential cause of web buckling and cracking. The unexpected water flooding inside the steel box in the support zones was discovered when the winter had finished (Fig. 4). The authors hypothesized that the failure of steel webs might be caused by water trapped in cross-sections of the support crossbeam and frozen at low temperatures in winter. Therefore the next stage of numerical analysis was aimed at checking this hypothesis.

Basing on the on-site inspection and design drawings of the superstructure, a local FE model of the damaged part of the structure over the support was developed. The model consisted of one chamber constituting a part of the support cross-beam (Fig. 5). The total number of 6611 four-node shell elements with dimensions of 0.05 × 0.05 m were used to discretize the steel structure under consideration. The stress-strain curve described in Eurocode 3 was used to describe the non-linear behaviour of structural steel. The concrete bottom slab was discretized with 8450 eight-node solid elements with dimensions of 0.05 × 0.05 × 0.05 m. A linear isotropic material model according to Eurocode 2 was adopted for C35/45 concrete. The bottom slab reinforcement were also modelled in the subsequent concrete elements.
The decisive issue in the failure analysis was to model the appropriate loads, suspected to be a main cause of failure. Due to water appearance inside the box girder an atmospheric ice loads due to freezing rain or snow shall be considered in the assessment of ice-sensitive parts of structure, as the closed steel chambers of support cross-beam. In the areas where records or experience indicate that freezing water produces large loads, the site-specific studies shall be used [3]. The load was modelled as the surface pressure applied to steel plates creating the chamber. The pressure was gradually increased up to the lack of the convergence of FE model. By means of the geometrically linear and materially non-linear FE analysis the maximum pressure value was determined at which the yielding of steel occurred and the iteration was not convergent (Fig. 6).

**Figure 6.** Numerically determined web deformation and stress distribution against actual web deformation and failure.

Based on the FE analysis results the displacement plot of the web centre point depending on the applied pressure of freezing water was established (Fig. 7). The yielding of web steel occurred at the approx. 6 MPa pressure and at approx. 12.5 MPa the plot approached the asymptote, which meant cracking of the web. The corresponding strain reached 48.5 ‰, which was a value many times higher than yielding point of 1.6 ‰ for S355 steel.

**Figure 7.** Displacement of the web centre point depending on the applied pressure of freezing water.

The numerical analysis clearly revealed that the hypothesis the freezing water had caused buckling and cracking of steel web was good justified. The detailed on-site inspection and actual deformation measurement confirmed also that hypothesis - observed damages resulted from freezing water filling the support cross-beam chamber. The analysis showed that the pressure applied on the internal surfaces of the chamber associated with the increase in the volume of freezing water could cause significant deformation of the web.
5. Repair method assessment
The web repair method prepared by the contractor comprised (Fig. 8):
• cutting damaged/deformed web plates and replaced them with the new ones, welded in the same place,
• welding of vertical stiffening ribs on the girder’s web,
• welding of vertical stiffening ribs on the support cross-beam’s webs,
• drilling drainage holes with diameter of 30 mm in the bottom plate of the cross-beam’s chamber.

![Figure 8. Repair method of the web: stiffening ribs (red, blue), replaced plates (green) and drainage holes (yellow).](image)

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![Figure 9. The local model of the support zone after repair: mesh (left) and the phase of damaged web plates removal (right)](image)

Figure 9. The local model of the support zone after repair: mesh (left) and the phase of damaged web plates removal (right)

To confirm that the proposed method of repair is correct, a numerical analysis was carried out using two models: the global one, used for safety analysis of the entire structure (Fig. 3) and the local one, which discretized the support zone of the superstructure (Fig. 5). The local model comprised a bigger part of the box girder than the local model used for failure analysis: the half of the box girder (one web) and the girder section of ± 3.15 m from the support axis. The subsequent concrete deck slab and the RC bottom slab were modelled as well (Fig. 9). In contrary to the first local model the possibility of transverse cracking of concrete deck slab was taken into account and the cracked concrete with the reduced stiffness in the longitudinal direction and tension stiffening effect of
reinforcement were assumed in the calculation. The second local model was loaded with internal forces derived from the safety analysis carried out by means of the global model (Fig. 3). The girder support on bearings was modelled using elastic finite elements with infinitely high stiffness densely situated on the surface corresponding to the bearing plates. The entire local model consisted of 15 388 shell elements including 13 077 nodes.

The FE analysis was divided into two phases of repair:
- phase 1 - strengthening of the girder webs and the cross-beam with vertical ribs while cutting-off the damaged part of webs,
- phase 2 - repair of the webs with welded plates, i.e. fixing the holes.

Assuming two phases of repair works, loads acting on the girder were divided into two groups. In phase 1 the self-weight, early concrete shrinkage and transverse prestressing of the deck slab were considered. In phase 2 the equipment self-weight, final concrete shrinkage and live loads were additionally taken into account.

The linear FE analysis was carried out and the maximum values of equivalent stresses in superstructure were determined according to the H-M-H theory. On Fig. 10 the total equivalent stresses for both phases of repair works are presented. After repair and strengthening the maximum equivalent stresses in the girder web at the support zone reached approximately 180 MPa. It should be noted that the concentrated stresses in the corners of the model result from concentrated external forces and should not be treated as overstressing the web. The equivalent stresses in vertical stiffening ribs were also checked. In any case, the equivalent stresses did not exceed the yield strength of the S 355 steel. Therefore, it was confirmed that in any phase of the repair works as well as in service the ultimate limit state (safety) of the repaired zone would not be exceeded.

![Figure 10. Summary of the equivalent stresses in the girder web in both phases (maximum stress in the support zone approx. 180 MPa).](image)

6. Summary
The subject of this paper is an untypical failure of a motorway composite box girder bridge. After the unplanned break in bridge construction the contractor discovered the significant deformation and local cracking of the steel web in the support zones. The detailed FEM analysis aimed at determining the safety status of the superstructure and the damage causes as well as assessing the repair method was carried out. Analysis of the damage causes was preceded by a detailed analysis of structural performance and the hypothesis that the failure may result from freezing water collected in the
chambers, theoretically empty, located at the lowest point of the box section. Such failures of steel structures with box section members occur often as a result of negligence, ignorance, as well as poor workmanship, wrong assembly sequence, etc. [2], [3].

As a result of the conducted analysis, it was confirmed that the assumed hypothesis was correct and it was found that the direct cause of deformation and cracking of steel web was the freezing water filling the closed cross-beams at low temperature. The following contributed to the failure:

- design errors in shaping the steel box girder including lack of drainage of the support zone and lack of drainage of the chamber of the box-shaped cross-beams located at the lowest point along the length of the superstructure;
- negligence during construction, enabling the inflow and long-term accumulation of large amounts of water in the supporting zone of the box section;
- lack of the appropriate superstructure protection during the break of construction related to the change of the contractor.

These design errors and former contractor’s negligence resulted in the need to design and implement a complex repair project, which in turn resulted in increased bridge construction costs and extended time for completion.

References

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