

# Corrosion as a cause of the failure of the pipeline steel supporting structure

Wiesław Paczkowski<sup>1</sup>, and Szymon Skibicki<sup>1,\*</sup>

<sup>1</sup>West Pomeranian University of Technology, Faculty of Civil Engineering and Architecture, al. Piastów 50, Szczecin, Poland

**Abstract.** The article describes the failure of the steel supporting structure of the pipeline caused by corrosion. The structure in question is a three-span steel bridge with truss girders. The structure was erected in the 1960s and is located above the railway track. Due to the specific shape of the hinges, which join the middle span with the rest of the structure one of them was damaged by corrosion, which caused it to break off. That led to situation where the middle span was retained by only three joints. To achieve proper safety of the structure it was additionally supported and strengthened. Due to the necessity of quick repair (there was a real threat of a collapse), some of the works performed had a provisional character. The article presents a description, detailed causes for failure and all the works, which were carried out in order to protect and repair the structure. The analysis of this case showed that, despite adequate protection of structural elements against corrosion, improper design resulting in retaining water led to serious failure.

## 1 Introduction

Corrosion of structural steel is an electrochemical process. However not only proper protection but also proper design of a structure can have significant influence on its corrosion resistance.

The paper presents an intricate case of structural failure caused by the corrosion of structural steel members in a bridge carrying a water pipeline 2Ø600. The erection process required a use of individual system of hinges. The corrosion of hidden parts of the hinges led to significant material loss and eventually substantial decrease of the bearing capacity. Luckily, the minor distortion of the structure caused by hinge socket rupture was spotted by the passer-by who alarmed the authorities leading to immediate actions.

A typical corrosion process occurs when two metals are immersed in an electrolyte solution leading to the transfer of electrical current. Pure water is a poor conductor of electricity, but its conductivity increases with the content of salts or acids. The contamination can easily occur in urban or industrial environments. In the case of structural steel, the difference of potentials may be caused by slight changes in steel composition or variations in temperature or external contamination (e.g. by acidic sulphur dioxide).

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\* Corresponding author: [szymon.skibicki@zut.edu.pl](mailto:szymon.skibicki@zut.edu.pl)

The corrosion of steel requires both water and oxygen to be present. The process leads to hydration of ferrous oxide which produces the  $Fe_2O_3 \cdot H_2O$  known as rust. The rate at which corrosion occurs will depend upon the availability of oxygen and water. The examined bridge was under permanent influence of both water and oxygen [1].

One should also consider the difference in electrochemical potential of steel used for the structure itself and steel used for pivots. No reliable data is known concerning the grade of steel used for pivots in the bridge under consideration. Stainless steel (which was not the case in analysed situation) can act as a cathode and highly accelerate process of corrosion of structural steel.

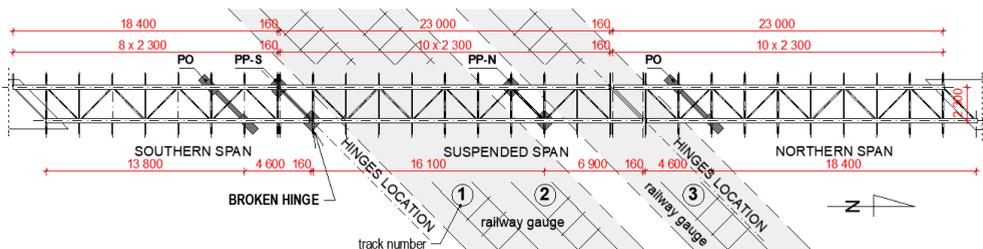
For the great majority of buildings the interior steelwork is subjected to water for only a brief period resulting in an insignificant corrosion. This is not the case of structures exposed to atmospheric conditions. In this case the design of structure, regardless of protective coatings, can have a significant influence on the corrosion resistance. Since the longer the period of wetness, the greater the corrosion it is important that structures should be strongly designed to discard water rather than retain it. Any situation that is likely to lead to the entrapment of moisture and dirt should be avoided. Drainage holes should be provided to ensure that all of the water would escape from the members. Additionally leaving a free flow of air can secure proper drying [1].

Design for maintenance is also important. If access is difficult or space is cramped – as in the analysed case – it is unlikely that maintenance will be properly carried out. This is considered to be the main reason of structural failure of the bridge.

## 2 Construction details

### 2.1 Location and construction design

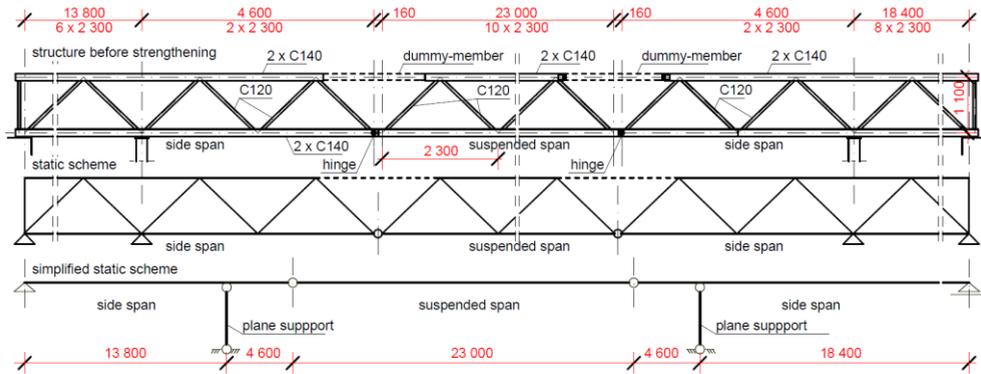
The steel supporting structure of the water supply system 2Ø600 is located above the dual-track of electrified national railway line and the electrified railway line running beyond national borders. The construction diagram is shown in Fig. 1.



**Fig. 1.** Plan of the supporting structure (PO – structural supports, PP-S – southern temporary supports, PP-N – northern temporary supports).

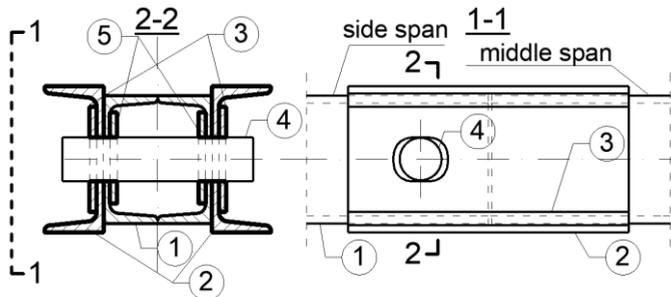
### 2.2 Construction details

The original steel supporting structure is made as a three spans Gerber beam with truss girders and two plane vertical supports. Abutments and foundations of plane supports are made as reinforced concrete elements and are not the subject of that analysis. The structure was erected in the 1960s before the electrification of the railway line.

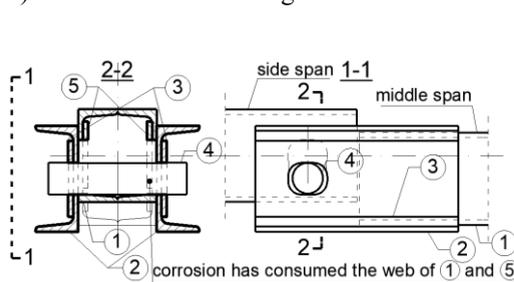


**Fig. 2.** Side view of structure and statical scheme.

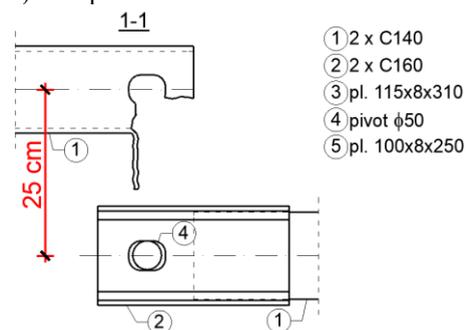
a) hinge design



b) destruction of the strengthened web



c) collapse



**Fig. 3.** Hinge design (a) and failure mechanism (b, c).

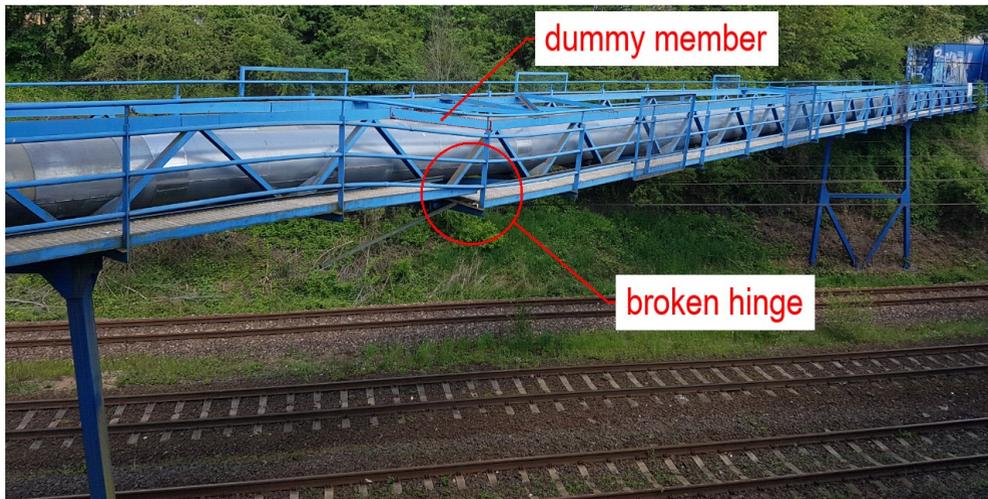
A diagram of the structure is shown in Fig. 1 and Fig. 2. All three spans are right-angled. The main girders are formed by postless trusses of type W with parallel chords [2]. The axial spacing of the main girders is 2300 mm, which means that the transverse axis of the bridge and the axes of the hinges of the suspended span run at an angle of 45° to the axis of the steel bridge. The steel bridge has three spans (Fig. 1 and Fig. 2), which are connected by hinges as seen in Fig. 3 a).

The upper and lower chords of the main girders are made of rectangular hollow sections 120x140 obtained by welding two C140 channel sections on the edges of their flanges. The central part of the compressed upper chords of the suspended (central) span is reinforced. In a distance of 5x2.30 m a 130x8 side plates are fixed to the chords by fillet welds. The axial spacing between the upper and lower chord is 1100 mm, which results in diagonal members angle of 43.7°. A horizontal wind bracing is provided in the bottom chords in the form of a

W-type post truss. The water mains pipes are placed symmetrically with respect to the steel bridge axis at a spacing of the 1100 mm. Pipes are supported on each cross beam with oak bearings. The pipe axis is located 450 mm above the upper level of the crossbars. In both main girder chords there are assembly joints strengthened with 130x8 straps.

### 3 Failure details

The incident occurred on Sunday in the early morning. A passerby heard a suspicious sound and noticed visible damages to the steel bridge (Fig. 4). Immediately the city officer on duty was notified. On the same day, a visual inspection was carried out, which allowed preliminary to determine static scheme of the structure, the extent of damages and the threat level. Direct action was taken to secure the superstructure of the bridge.



**Fig. 4.** Structure at the time of failure.

A closer inspection of structure allowed to determine that four bars of the upper chord were dummies added for aesthetics and security, but they did not transmit internal forces in the structure (Fig. 5). This means that the middle suspended span works as a simply supported system. As a result the whole structure works as a double hinged gerberian beam with a skewed system of supports and joints. As a statically determinable system, the structure did not have a load-bearing capacity reserve. The spatiality of the system, including its skewing and redundant number of hinges, protected the structure from total failure. An additional factor protecting the structure was the stiffness of the pipelines.

The failure of the structure occurred as a rupture of the southeastern hinge (W-E) of the middle span (Fig. 4). The layout of the compass directions used in this description is in accordance with Fig. 1. The material belonging to the bottom chord of the eastern girder have been torn out from the southern span. The pivot responsible for hinge operation remained in the C160 channels that were fixed to the lower chord of the middle suspended span. The size of the structural displacement in the broken hinge reached 25 cm. The mechanism of hinge damage is shown in Fig. 3 b) - c).

a) torn webs of the lower chord



b) displacement of the pivot



**Fig. 5.** Rupture of the hinge.

Despite the undeniable dynamic effect associated with the sudden displacement of the middle span, the load was transferred to three remaining hinges that managed to retain the bearing capacity. It is worth to notice the effect (show in Fig. 7a) of the slippage of the pivots in the remaining hinges to the level of the lower flanges of the hollow 2xC140 section of the lower chord. The dummy members acted as a special type of dilatation that isolated the deformations of both main girders upper chords. As a result, the main elements of the load-bearing structure were not damaged. On the other hand, the pipes  $d = 40$  mm used for the external railings were fixed in the posts through, thanks to which they retained the continuity of deformation without being a subject to permanent damage (Fig. 6).



**Fig. 6.** The dummy-member (bar on the left) and the deformation of the railings.

The horizontal displacement of the upper chord of the eastern main girder which appeared in the dummy strut did not cause permanent deformations in the superstructure.

Permanent deformations occurred only in the immediate vicinity of the broken hinge. The side beams of the deck were deformed and the plastic gratings of the passage were torn off. The rods supporting the platforms near the W-E and N-W hinges were also torn off. In all three other hinges retaining their load-bearing capacity, the pivots slipped down. As a result the pivots leaned towards the lower flanges instead of designed middle position in webs of the bottom chord. This caused a visible discontinuities in the upper chord and distinct setoff of the side beams of the passage.

## **4 Causes of damages**

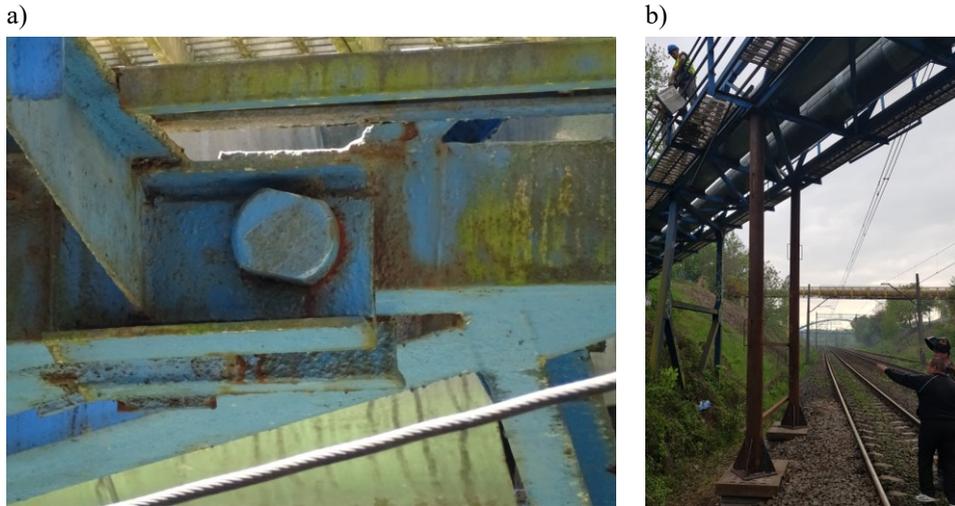
### **4.1 Long-term causes**

The analysed structure was erected in the mid-sixties meaning its service life already exceeded 50 years. Such period is assumed to be the typical exploitation time of a structure. Construction standards covering both loads and load-bearing capacity of the structure are calibrated for such a period. This does not mean, of course, that 50 years of service life is the limit. After exceeding the service period it is necessary to provide additional maintenance, which is revealed as a noticeable statistical effect. Considered structure was exposed to two types of loads: service loads and vibrations generated by the intensive train traffic on both railway routes. Quantitative assessment of this dynamic effect was not included in this study. Service load from water-filled pipelines is one of the most stable and predictable ways of loading the structure. Another group of actions are climatic effects (wind, snow), which were taken into account in this study displaying negligible influence.

In addition to the load influence, the safety of the structure is also determined by the material factors. In case of considered structure, corrosion was spotted on parts of the hinges supporting the middle span of the bridge. The corrosive spots were however hidden from the sight during a standard maintenance inspection. The designer correctly assumed an increased effect of the loads occurring in the hinges, adding strengthening strips  $t = 8$  mm. The strips were added both on the inner surfaces of the C140 webs that form the bottom chord and on the outer surfaces of the C160 webs which are the extensions of the bottom chord of the middle span. However, there was no section closing the face of the complex cross-section of 2C140 forming the lower chords of the main girders. As a result, corrosion occurred, the effects of which could not be suppressed by carried out standard anti-corrosion measures. The lack of the correct shelter of the internal part of the hinges should be classified as a design error.

### **4.2 Direct cause of the failure**

Any sudden event leading to a structural failure must be caused by some particular initiating factor. The integrity of the load-bearing structure depends on many factors. Rarely, slowly progressing, prolonged processes lead to the physical equilibrium being exceeded. A potential cause of the failure in this particular case could have been the heavy rain that occurred at night before the failure. Each crossbar that was placed every 2.30 m carries two wooden bearings. Their upper surface is shaped to match the curvature of the pipe resting on it. The remaining walls form a regular cuboid with dimensions of 40x20x12 cm. As a result of heavy rainfall, the water could soak up unevenly, resulting in differences in each bearing volume. Possible overloading of the outer bearing in the middle span could have led to the failure of the S-E hinge, which was already weakened due to long-term processes described in pt. 4.1 The downward movement of the middle span led to a rapid redistribution of the load from the eastern water mains pipe, stopping the movement of the span. The eastern pipe, as well as a part of the western pipe took over the loads, relieving the central span in the zone of the southern hinges. This mechanism, which was not considered during the design stage, saved the structure from a catastrophic failure. The redistribution that utilized the beyond standard load-bearing capacity of the pipes increased the load on the southern span, without causing any structural damage.



**Fig. 7.** a) slippage of the pivot in the hinge; b) provisional support during construction stage.

## 5 Strengthening of the structure

After an emergency check of the condition of the damaged structure required measures were taken. Lowered structure was dangerously close to the electric connect lines. The electricity was switched off and train traffic in this area was stopped. At the same time, the section of water mains running through the bridge was cut off with simultaneous drainage of the remaining water. The structure was relieved of the main service loads and the dynamic factors were reduced. The disconnection of the water mains and the traffic block were highly strategic operation for the whole city. Although there was no water shortage or train traffic was not completely cut off, the object's serviceability ought to be restored as quickly as possible. The initial maintenance check of the structure was carried out under a threat of a possible catastrophic failure of the middle span. Removal of water from both pipelines relieved the middle span of 130 kN load, which allowed the check the situation from the level of the structure.

Initial, most crucial task was to support the middle span at the location of the broken hinge. The temporary support, considering available materials, was erected in the form of two columns consisting of  $\text{Ø}244.5 \times 8$  steel pipes with height of 5.62 m (PP-S support in Fig. 1 and Fig. 7 b)). The column heads were placed at the level of the lower chords of the middle span. The top of the eastern column was thus at the level of the chord, after it had been torn off. The base of the columns were made of 500x500 mm base plates with a thickness of  $t = 20$  mm and stiffened with four ribs ( $h = 450$  mm,  $a = 190$  mm,  $t = 10$  mm). The column bases were placed on five 14 cm thick reinforced concrete slabs. The slabs are enclosed in C140 frames. The column heads were permanently welded to the structure of the supported span. The column bases were not fixed to the concrete slabs. Instead, the slabs were covered with railway gravel from three sides. Temporary supports were installed directly to the east, outside the railway clearance gauge of track no. 1 (Fig. 1). Installed columns provided protection from span collapse, however the span was not in a safe distance from catenary wire.

A repeated inspection carried out after the erection of the southern temporary support (PP-S) showed that the steelwork is in a generally good condition with the exception of the northern hinges of the suspended span. The limited access to the both hinges made it impossible to carry out an accurate quantitative evaluation of their carrying capacity. The

qualitative evaluation of the condition led to the conclusion that the effects of the long-term adverse processes indicated in point 4.1 may have accumulated in both hinges. It was concluded the leveraging of the middle span was not possible, as could lead to the rupture of the northern hinges. At the same time, it was determined that the observed technical condition of the northern hinges, in view of the continuous traffic on tracks no. 2 and 3, could lead to a catastrophic failure of the hinges with tragic consequences. In this case, it was decided that the northern part of the central span should be immediately secured with a second temporary support. Due to the large enough distance between track no. 3 and no. 2, it was decided to set up a temporary support between these tracks without breaking the railway gauge. The support was set at a distance of  $3 \times 2.30 = 6.90$  m from the northern hinges, counting along the main girder chords. This changed the static scheme of the middle span from a free-supported of  $L = 23.0$  m in length to a free-supported span of  $L = 7 \times 2.30 = 16.10$  m with a cantilever section of 6.90 m in length. This change was only related to the effect of the temporary supports. The design of that support was similar to the southern support. The support was erected at night when train traffic was stopped.

It was also decided that the damaged southern hinges should be replaced by a system of hangers. The installation of a temporary north support allowed for the commencement of assembly work, which was carried out at night when trains were not in operation. The installation of temporary supports could not be considered as final protective measure for the structure. For time being when the pipes were drained from water, the solution was sufficient. Taking into consideration long-term service under load of filled pipelines as well as close vicinity of passing trains it was necessary to change it. So, it was decided that the hinges should be excluded from operation by replacing them with a system of hangers. In fact, they were left in the same condition as they were on the day of the failure, and their role was taken over by properly constructed hangers.

As a basic criterion for the selection of hangers, it was assumed that they should be cheap, easy to make and have the same load capacity as the hinges. The structure was designed in 1964, in accordance with PN-62/B-03200 [3] standard, which does not specify any strength conditions for the load-bearing capacity of bolts. In this situation the PN-90/B-03200 [4] standard was used. On the other hand, it was accepted that the Eurocodes differ too significantly from the design methods in force at that time.

The hanger consists of two M20 class 8.8 bolts. The design resistance of such a two-bars system is 264 kN, which highly exceed maximum design reaction on one hinge 132.3 kN. The solution is shown in Fig. 8.

Each of the hangers had the same construction:

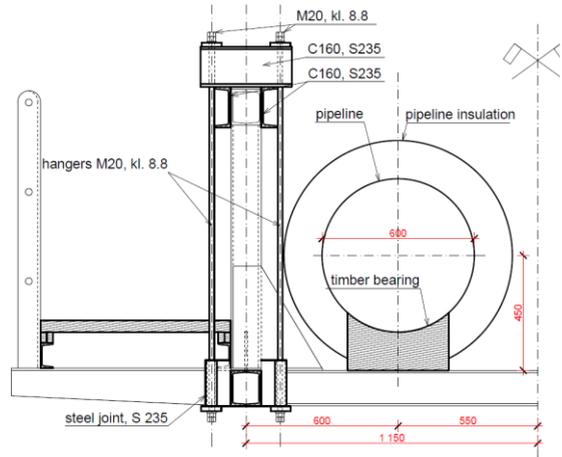
- the dummy member above the replaced hinge was removed from the upper chord;
- the upper chord of the side span was extended with 2C160 channels connected web to web by welding;
- an additional diagonal member was installed of C120 channel with  $t = 6$  mm strengthening plate on the edges of flanges;
- new diagonal member and an external diagonal member from the side span were connected with an 10 mm thick gusset plate matching the angles;
- additional setup of 2C160 channels with flanges outwards (24 mm in light) was added perpendicularly to upper level channels;
- two horizontal plates 80x80 mm with a hole in diameter of 24 mm providing support for M20 bolt nuts were welded to the system of transverse channels;
- two sleeves welded on both sides to the lower chord of the middle span where used to suspend it on two M20 class 8.8 bolts;
- the bolts were threaded on all length what allowed to regulate the suspension by the use of counter nuts.

Leaving the original hinges allowed for mutual stabilization and geometric invariability of individual spans.

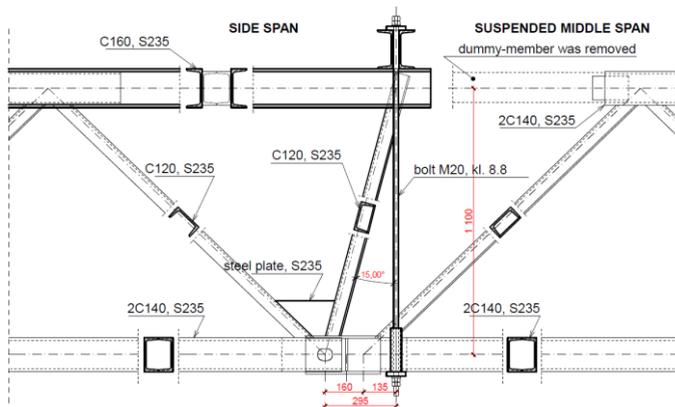
a) the overall view



b) cross-section



c) side view



**Fig. 8.** Hanger replacing the hinge.

The leveraging was carried out using a 1000 kN hydraulic lift. The traffic stopping was necessary for this operation. The new position of the middle span, corresponded to the previous height of the S-E hinge and was stabilized using hangers. The span was supported with the east column of the south provisional support. Moving the middle span to a safe distance from the overhead contact wires allowed to enable the train traffic. Achieved condition was agreed to be sufficient to return the water mains to operation. The southern hinges of the middle span were fixed to the southern span by hangers and simultaneously were supported by the columns (Fig. 9). The northern hinges of the middle span were suspended on hangers while both main girders of the middle span were supported by columns located near the northern hinges.

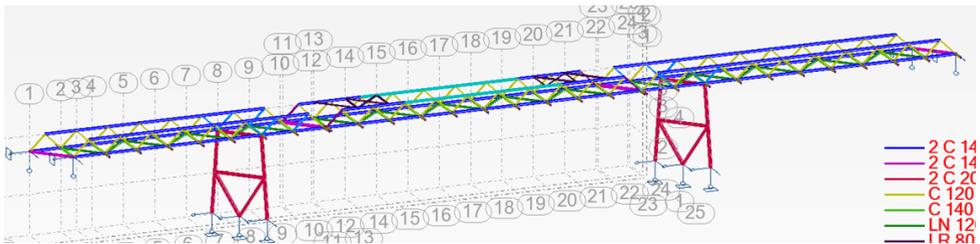


**Fig. 9.** Structure after strengthening by provisional supports (view from the south-west).

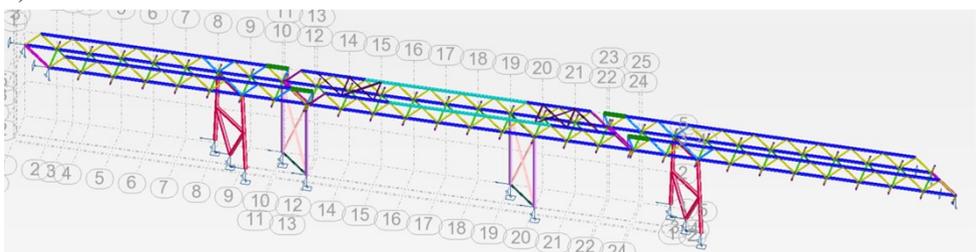
## 6 Structural analysis

The computer model of the analyzed steel supporting structure of the pipeline has been analysed in Autodesk's Robot Structural Analysis Professional software. The structure was mapped using a three-dimensional bar model (Fig. 10).

a) MODEL 1



b) MODEL 3



**Fig. 10.** Numerical model of the structure.

Five models have been created:

- 1) original structure after water mains replacement under full service load in 2015- marked MODEL 1 (Fig. 10 a));
- 2) original structure under full service load at the moment of failure after breaking of the S-E hinge - marked as MODEL 1a;
- 3) structure under full service load after introduction of hangers, assuming that all hinges have been removed and the middle span is attached to hangers only - marked as MODEL 2;
- 4) structure under full service load, assuming that the middle span is supported only by the temporary supports - marked as MODEL 3a;

5) structure under full service load in the final system of hangers and temporary supports - marked as MODEL 3 (Fig. 10 b).

Each of the above mentioned models was tested for several load combinations taking into account the water level of operational water mains and climatic actions. The load was calculated in accordance with Polish standards [5–7], the combinations were made on the basis of formula (1) from standard [7]. The buckling length of truss bars was determined in accordance with PN-EN 1993-2 (Table D3 point 2) [8] and literature [9,10]. The obtained results confirmed that both the initial design and temporary reinforcements - both columns and hangers are correct. The design and structure documentation does not specify the steel grade from which the structure was made. In order to determine the steel grade five samples were taken to carry out the tests in accordance with PN-EN ISO 6892-1:2010. It was determined that the design strength of the tested steel was 230 MPa (mean: 277.86 MPa, standard deviation: 7.8 MPa), which allowed the steel to be qualified as St3S corresponding to S235. The mean value of Young's modulus of elasticity was  $E = 206$  GPa, which is consistent with the standard value of  $E = 205$  GPa.

## 7 Conclusions

Proposed solution of strengthening the steel structure meets the limit state requirements under the load of both operational pipelines. This has been achieved by double security measures:

- suspending the middle span to the side spans at the location of all hinges (Fig. 8), while leaving three original hinges with an uncertain degree of reduction of load-bearing capacity,
- use of temporary supports supporting the middle span in the southern hinges of both main girders and installing additional support at a distance of 6.90 m from the northern hinges of both main girders (PP-N and PP-S in Fig. 1 and Fig. 7 b)).

The double measures resulted from the progress of the rescue action. First of all, the middle span had to be supported at the S-E hinge (failure point), which was achieved by introducing a flat support made of two  $\text{Ø}244.5 \times 8$  pipe columns located under the southern hinges of the middle span. The main purpose of the actions was, in addition to protecting the structure against the rupture of the middle span, to elevate the span to a safe height above the overhead contact line. Due to the weakened load-bearing capacity of the remaining hinges caused by long-term factors, this operation could not have been undertaken without securing the span with a second temporary support similar to the first one. Only then it was possible to install the four sets of hangers that played the role of hinges connecting the side and suspension spans. Both measures of reinforcing the span allowed to progress with the leveraging process of the S-E hinge to its original position. Afterwards the operational functions of pipelines and railroad tracks were restored.

The premiss of the solution was the bridge administrator's declaration that within two years a new steel supporting structure will be constructed. The applied solution, although ensuring safe operation of the water mains, cannot be treated as a final solution. The reason for this is the way the temporary supports are based on reinforced concrete slabs. Possible irregularities in the operation of temporary supports (e.g. caused by washing out the soil from under the supports as a result of extreme torrential rains or by swelling of the frozen soil) will lead to a change in working conditions of the entire structure and redistribution of internal forces occurring in it. However, this will not affect the load-bearing capacity of the structure. The solution with the use of provisional supports without appropriate foundations was made because it was the only sensible solution that could be executed quickly and without disturbing the railway lines in operation. The decision to do so was made in the face of the threat of a catastrophe, which could have had very serious consequences.

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