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DESIGN AND CONSTRUCTION OF A TWO-HINGED STEEL ARCH BRIDGE ALONGSIDE HISTORIC BRICKWORK VAULTS

Ph. Van Bogaert[1]

ABSTRACT

The two-hinged steel arch bridge crossing of 27.3m spanning the Industrial Boulevard in Brussels was built to carry a fourth track of the railway line from Brussels to Ostend, thus accommodating the increase in train traffic. The existing bridges are brickwork vaulted arches with heritage value and have also been refurbished during this project. Therefore a slender steel closed section, two-hinged arch was designed to contrast with these solid structures. The design had to account for the perpendicular pressure effect on flanges of box sections, due to the arch curvature. The arch box has no stiffeners, except for the vertical struts, carrying the concrete bridge deck. In addition, the hinges are continuous with the box section and needed internal stiffening. Particular care has been given to compliance with the steel arch of the hinge base sockets, encased in the concrete abutments. A procedure was worked out to compensate any misalignments and angular rotations, within the limits of possible additional steel stress. The steel arch was welded on site from 3 prefabricated elements and hoisted as a single piece to its final position. Recently a load test with heavy lorries, including dynamic loading, was carried out. The measurement results clearly demonstrate that fatigue resistance is the main issue and that the lateral pressure on flanges really occurs and may condition the strength of structures with high curvature.

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ThThe two-hinged steel arch bridge crossing of 27.3m spanning the Industrial Boulevard in Brussels was built to carry a fourth track of the railway line from Brussels to Ostend, thus accommodating the increase in train traffic. A slender steel closed section, two-hinged arch was designed to contrast with these solid structures. The design had to account for the perpendicular pressure effect on flanges of box sections, due to the arch curvature. The arch box has no stiffeners, except for the vertical struts, carrying the concrete bridge deck. In addition, the hinges are continuous with the box section and needed internal stiffening. Particular care has been given to compliance with the steel arch of the hinge base sockets, encased in the concrete abutments. A procedure was worked out to compensate any misalignments and angular rotations, within the limits of possible additional steel stress. The steel arch was welded on site from 3 prefabricated elements and hoisted as a single piece to its final position. Recently a load test with heavy lorries, including dynamic loading, was carried out. The measurement results clearly demonstrate that fatigue resistance is the main issue and that the lateral pressure on flanges really occurs and may condition the strength of structures with high curvature.

Introduction

The Industrial lane is an important access road for all traffic moving from the city centre towards the southern part of the circular motorway. Both at morning and in the evening traffic is dense and jams are occurring each day. The railway line from Brussels to Ostend at the coast crosses this important road with 2 brickwork vaulted arch bridges. The highest and largest one carries the main double track line, whereas the lowest arch carries a single track. The owner wishes the railway line to have 4 tracks in future. Hence, a bridge deck for single track had to be built. The latter is oriented towards the city and will have important visual impact for people leaving the city, since the initial vaulted arches already crystallize the idea of a gateway between the city and its suburbs.

During the design phase, already a long time ago [1], various ideas have been considered, both in concrete and steel as well as composite structures. It was decided that the alternative of a steel arch with slender box section would be the most compatible to the historic brickwork arches. It would allow maintaining an open view on the heritage structure and simultaneously demonstrating the large contrast existing between historic and modern bridge building. Since a long period separated the initial design from the actual building, the former was to be changed, also in view of some restrictions the steel contractor had some

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difficulty to cope with or was unsure about. In the following, comments and particular features are given concerning the design of short steel arches and particular verifications as well as the erection procedure are being commented. These data may prove to be of use for other similar construction.

Steel arch design

The steel arch itself has a span of 27.468 m and a rise of 8.038 m, the arch springs showing a skew angle of 65°. Because of its constant curvature, the basic shape is a circular segment. In addition, compared to the second degree parabola, the circle provides more vertical clearance at the walkways near to the abutments. Second degree parabolas are an interesting shape for uniform loading, which is seldom the critical case for bridges, or for larger bridges with many hangers, connecting the arch to a lower chord. Since live loads and most of the dead load are being transferred by vertical struts (Fig. 1), the arch loading mainly consists of knife loads. The latter are acting on the arch through the vertical struts.

Figure 1. General view after erection of structure

One would expect that the box section near the intersection with the highest strut would be the most critical one. Firstly, this section is the closest to one quarter of the span distance from the arch crown or springs, this being the location of maximum bending moments. Secondly, the longer struts must introduce larger bending at the intersection with the arch. However, the largest stresses are found in the obtuse angle of the arch springs at the corner of the upper flange plate.

Table 1. Comparing vonmises stresses (in MPa) in various sections ULS.

Arch section	Top plate	Top angle	Lower plate	Lower angle
1st field	70.1	81.2	189.4	130
Large strut	20.8	31.4	89.2	82.2
2nd field	40.6	50.9	48.1	24.8
Short strut	114.1	278	56.6	51.1
Crown	108.1	127.7	56.6	51.1

If the railway load takes a classification factor of 1.2, the maximum vonmises stress equals 315 MPa, which is just acceptable for compressed 45 mm thick plates. Table 1 shows the overview of vonmises stresses in other sections of the arch.

In the initial design, the plates were relatively thin (25 mm), thus causing serious transverse bending of the top and bottom flange. This is due to the effect displayed in Fig. 2, which is characteristic for short radius arches. Two closely spaced arch sections are being considered, the normal compression force being assumed constant. The intersection of both forces is drawn and it is noticed that both axes are not identical and have an angular rotation β. This results in the radial force P, which equals P = N β if the angle β is sufficiently low.

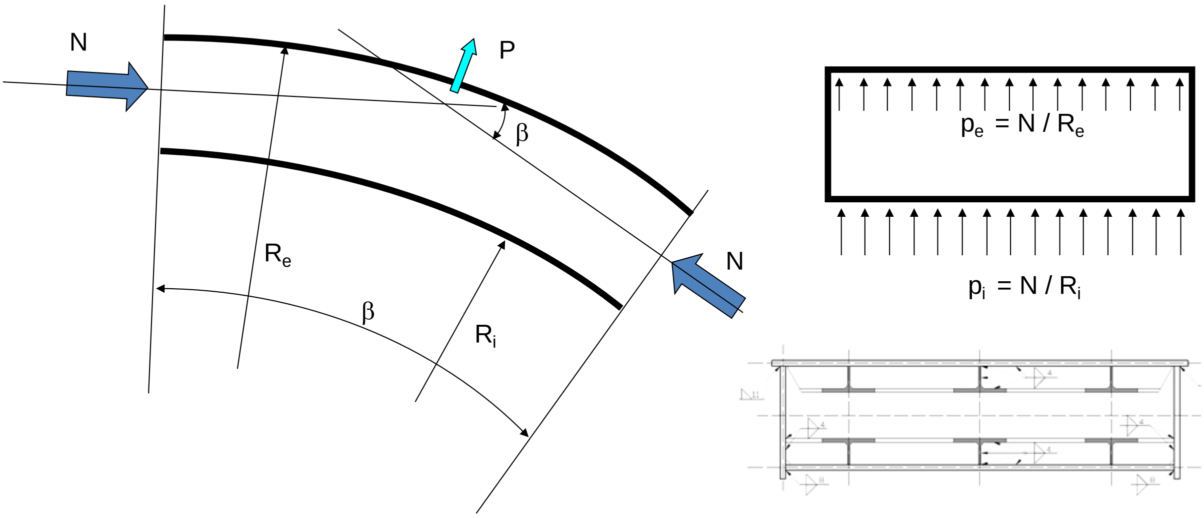

Figure 2. Force equilibrium for curved sections

The resulting force P can only exist if both flanges are subjected to radial pressure p, as is shown to the right of Fig. 2. If b equals the width of the box section, P also equals P = p β b R or p = N / b R. Hence the lateral pressure is easily found. Obviously the radial pressure, shown to the right of Fig. 2 introduces large deformation and may cause plate buckling. This is shown by the mode shape of Fig. 3, in which local buckling patterns are appearing.

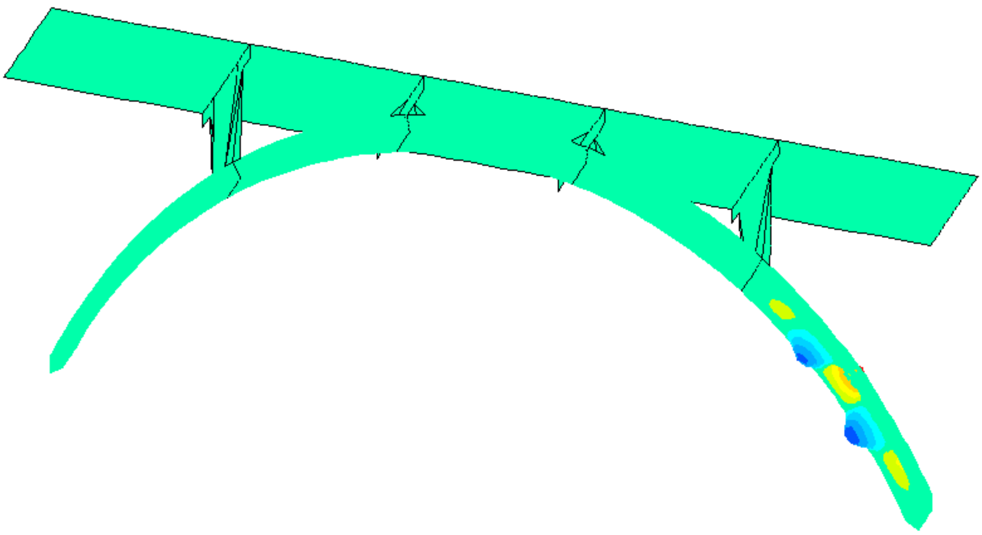

Figure 3. Local arch buckling patterns due to lateral pressure

The aforementioned effect of lateral pressure thus required stiffening of the top and lower box plates, due to transversal bending. This would have been internal stiffening as explained in [1] and shown to the right of Fig. 2. However, in the final design, more heavy plates were used, thus avoiding the internal stiffening. The increase in dead weight most certainly being compensated by the simplified production.

The internal stiffeners of the initial design could not be welded entirely to the box webs, since the box needed to be closed by the top flange. To the bottom right of Fig. 2 the non-continuous arrangement is shown. However, non-continuous stiffening of compressed and bended panels may have almost equal resistance as continuous stiffening, according to research [2].

However, no box section can keep its shape unless distortion is being prevented, this requiring diaphragm or other types of stiffening. Internal stiffening of small closed sections is rather difficult. The basic idea has been to use the struts as diaphragms, passing right through the arch box section. This can be seen in Fig. 4, taken during assembly of the structure at the workshop. Obviously, the adjacent parts of the arch box are ready to be welded to the flat strut web and stiffening gusset plates are to be added.

Figure 4. Assembling of arch box section to diaphragm stiffening strut

The gusset plates ensure the stiffness of the connection between both elements. Their size, measured along the box section rib has been determined to reduce the stress variations at the connection, thus obtaining sufficient fatigue resistance.

Arch springs

The arch springs consist of a simple circular rounding, closing the arch box section, which is fitting into a hollow steel solid supporting socket. No complicated hinge had to be fabricated, which is often the reason why clamped arches are preferred to hinged structures. Obviously, both parts, the arch spring and the hollow socket must be fitting together perfectly.

Since both the vertical reaction and the arch thrust force must be introduced through the hinge and because of the skewness of the bridge, a torque is resulting from the superstructure reactions. The characteristics of the hinge have been determined by the Hertz-contact formulas. This is in principle a rather old and conservative method, since it does not apply at ULS and results in many discussions concerning the length of the contact area. In addition, the method is not applied to bolts and other contact problems. Since the type of hinge, being used (Fig. 5) also constitutes a curved member, similar to the situation of Fig. 2, the curved plate is subjected to radial pressure at its lower contact area. This pressure tends to push inward the edge rib of the curved ending. Hence, rib stiffeners have been provided, thus allowing the distribution of the contact pressure across the webs of the box section. The lateral webs of the hinge allow resisting horizontal forces and show a slight gap with their counterparts of the lower sockets. The various parts can be seen in the perspective view of Fig. 5

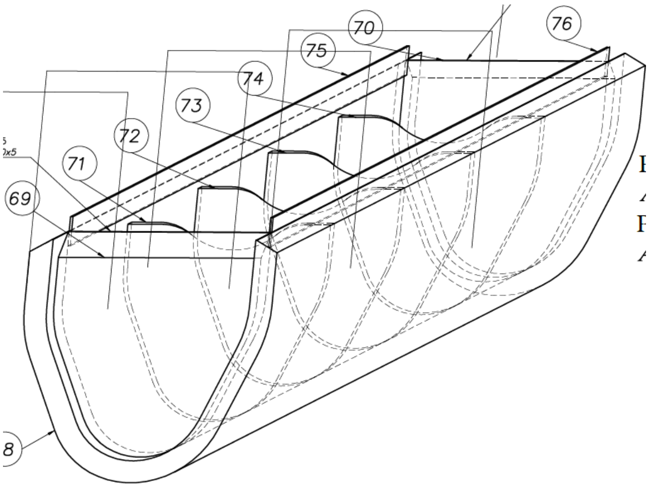

Figure 5. Arch spring hinge with internal stiffening

Evidently, the curved box end must at all times remain into the lower steel sockets, also during rotation of the hinges. The contact angle can also be derived with the Hertz-formulas and was in this case 6.033°, the maximum angle of rotation due to loads and temperature being 2.596°. To be certain that the arch will not move out of the steel socket, an opening angle of the latter of 130° was taken.

Arch erection

Because of the size of the steel part of the bridge, the 5 parts were welded on site to the 4 strut elements on fixed scaffolding. Since the arch top vertical deformations, due to the dead weight, are limited to less than 8 mm, no particular additional rise was given before fabrication. A procedure to fit the various parts together has been worked out. The principle

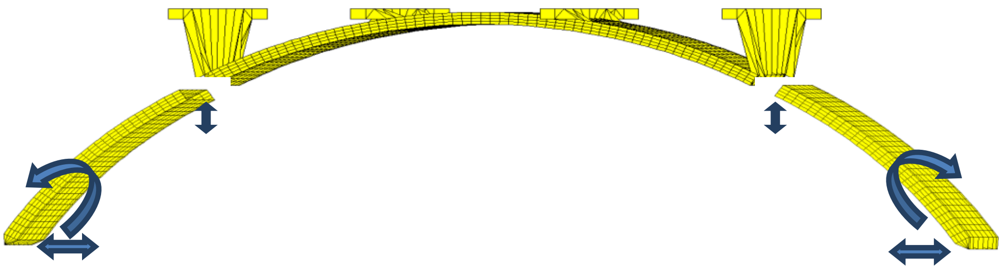

Figure 6. Principle for compensating angular displacements during assembling

is shown in Fig. 6. The lateral parts had to be turned by an angle of 0.1407°, corresponding to lower the arch springs by 12.71 mm and moving them towards the centre by 14.16 mm. As the central part with both smaller struts were already assembled at the workshop, this correction was the only one needed and allowed perfect fitting of the welds, connecting the larger struts to the 3 arch box sections.

The steel structure was subsequently hoisted by a single crane to be placed on the supporting steel sockets, which had to be encased in the concrete abutments. Obviously, the structure's geometry will be different than in the final situation. Fig. 7 shows the deformations when the structure is being hoisted. However, the vertical deformation certainly

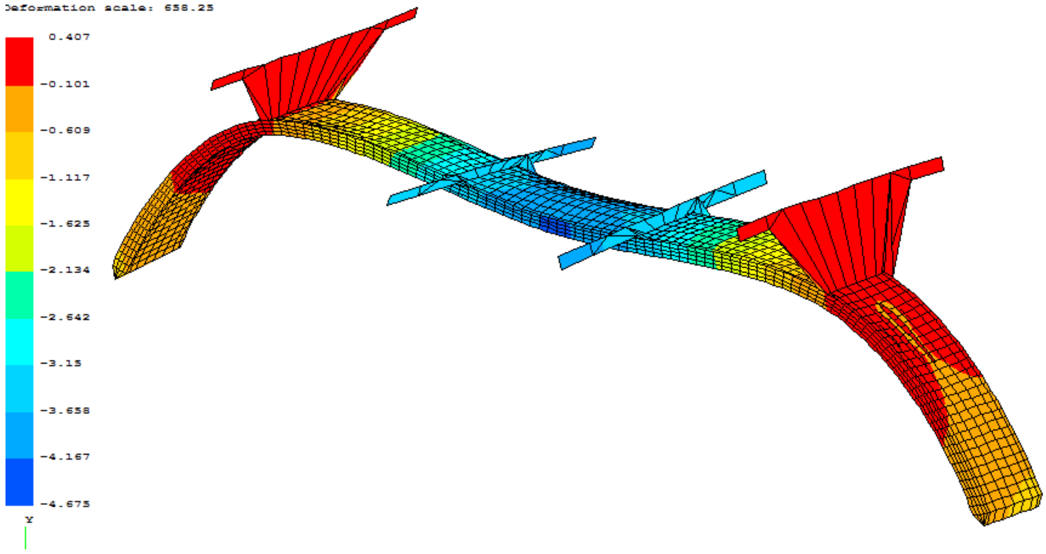

Figure 7.   Vertical deformations during hoisting of steel structure

is not the real issue for obtaining perfect fitting of the springs in the abutment encased steel sockets. Due to the skew angle, the obtuse and sharp ends of each arch spring have different values of the aforementioned displacements. These differences have been calculated and the sharp edge is found to be 0.191 mm lower during hoisting and 0.261 mm more towards the centre than the obtuse angles. The forces to compensate for these gaps have been determined as 12.58 kN in the horizontal direction and 7.996 kN in the vertical direction. In practice, there was no actual need for implementing a particular process to compensate these gaps.

A second problem concerns building tolerances, since the steel sockets are placed in concrete abutments and tolerances for this type of construction are usually much larger than for steel structures. Should these tolerances prove to be too large, the arch springs can still be forced into the sockets, but this will introduce additional residual stress.

In the case of Industry Lane bridge, this effect has been quantified. Referring to Fig. 8, there are basically 3 types of tolerances, the most evident one being a discrepancy between the distance from arch spring to spring and the identical distance of the concrete encased steel sockets. This type of tolerance (called Δx) may be compensated by applying identical horizontal forces F_1 on the arch. Should the distance between springs be too large, an increase of the arch thrust force will result, additional stresses being found from compression and bending.

Another type of tolerance may rise from horizontal non-parallelism of both steel sockets, which must be compensated by a vertical axis torque, or opposite horizontal forces F_2 located on one spring side in the obtuse angle, and on the other spring side in the sharp angle. This type of tolerance (called $\Delta\theta$) mainly causes distortion stress at the arch centre.

The third type of tolerance is caused by vertical non-parallelism of the supporting sockets. This may be compensated by a horizontal axis torque, introduced by two opposite vertical forces F_3, each located on one spring side at different edges, similar to the previous one. This last type of tolerance (called $\Delta\varphi$) mainly causes shear and torsion stress in the outside fields of the arch, as well as additional bending stress near the arch top and struts. These compensating forces and torques introduce residual stresses, the largest effect being found in the box section flanges at the obtuse angles. If the magnitude of the additional stress

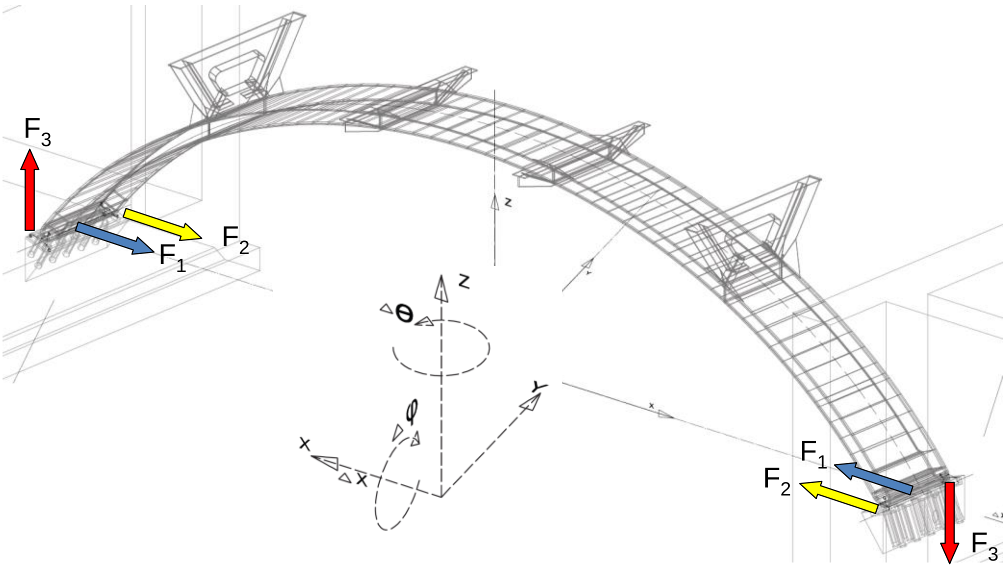

Figure 8. Compensating various types of tolerances

is limited to a predefined quantity, 3 equations may be determined to test whether the actual values of the tolerances Δx, $\Delta\theta$ and $\Delta\varphi$ are within this condition. These equations are not mentioned herein, since they have no general meaning and must be determined for each structure separately. The main question is to determine the acceptable magnitude of the residual stress. Normally, designs do not systematically allow for calculated values of additional stress of a permanent character. In the present case a safety margin existed and the magnitude of 20 MPa was decided in an arbitrary manner.

Load test

In December 2013 a load test was carried out on the Industrial Lane bridge. As the railway tracks were not installed yet, heavy 5-axle lorries have been used. Possibly, in the future the test might be repeated with actual train loading. Table 2 shows the mass of the lorries used during this test. Depending on the loaded length, these lorries represent around 18.5% of the design live load, although the effect of a single axle may well approach 36% of the local effect of the design model LM 71. The lorries were placed in various positions during the static test, whereas during a braking test, one single lorry was braking hard while crossing the bridge. The results of these tests are being discussed.

Table 2. Lorry characteristics : axle loads and total mass.

Lorry Nr	Axle 1	Axle 2	Axle 3	Axle 4	Axle 5	Total mass
1	6760 kg	10130 kg	9480 kg	10030 kg	9800 kg	37300 kg
2	6250 kg	8970 kg	8140 kg	11020 kg	10960 kg	45340 kg
3	6140 kg	10250 kg	9520 kg	9340 kg	9150 kg	44400 kg

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Looking at the position of the gauges in a current arch section, measured at 4 different cross-sections at locations shown in Fig. 8, the measurements demonstrate that there is only a minor difference between the left and right hand side of the box section. Hence the skewness has little influence on the stress state.

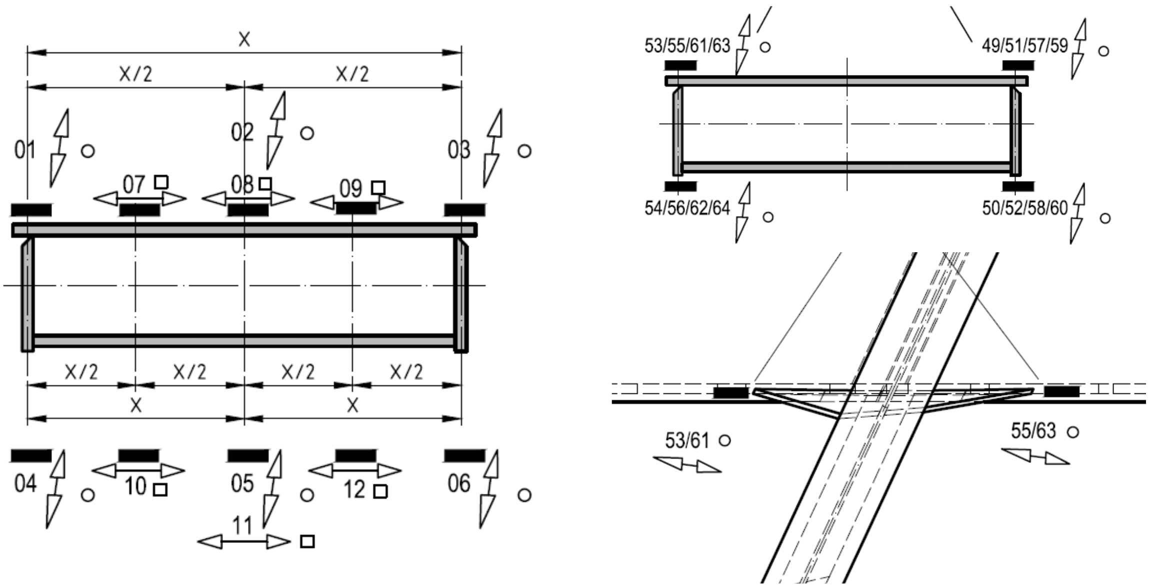

Figure 9. Strain gauge locations box section and strut sections

In addition, at the top and bottom flange centre of the box section the transverse stress is consistently somewhat larger than the longitudinal stress, although the values are really close. This implies that the effect of lateral pressure, due to the arch curvature is important, in spite of the plate reinforcement. Fig. 9 right indicates the location of strain gauges at sections near to the strut elements. The gauges at the upper corners of the box section are at a distance of 10 mm to the weld tip. Fig. 10 shows the evolution of the stresses near the weld toe and at the lower corner.

The measurement graph clearly indicates that at this particular location the maximum stress corresponds to -15.02 MPa, whereas the stress variation equals 20.37 MPa. For the lower corner of this cross-section those figures are respectively -6.93 and 12.91 MPa. In addition, the homogeneous compression almost disappears once the load is no longer on top of the strut and the stress state becomes almost pure bending. It is becoming obvious that stress variations are the most important effect of moving loads and the structural resistance is being determined by the fatigue strength, as was already found during the initial design.

The latter has become overwhelmingly clear during a braking test. The stress variation due to a single lorry equals 21.31 MPa. However, the location of the strain gauge is at a distance of 10 to 12 mm from the weld toe and is already inside the area where the stress concentration due to the built-up member has started. According to the ECCS-recommendations [3], the area where the concentration builds up is between 30.25 mm and 18 mm from the actual weld toe.

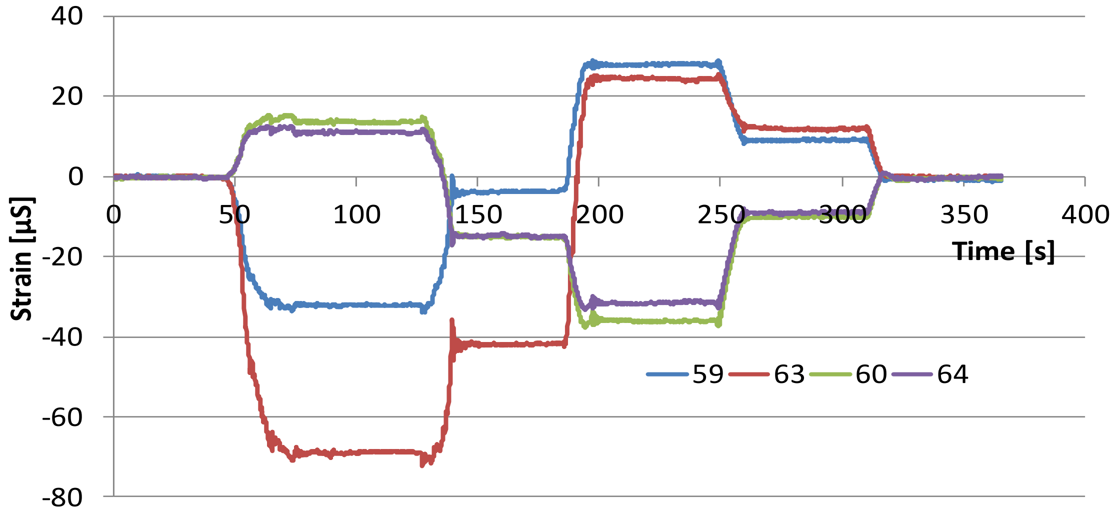

Figure 10. Strain variations near the largest strut element

Admitting that the effect of the lorry is due to the larger axle load of 110 kN and since the rail loading of LM 71 comprises the knife loads of 250 kN, to be increased by the dynamic factor of $\Phi_2 = 1.23$, the equivalent stress variation would then reach 250 * 1.23 * 21.31/ 110 still to be multiplied by the λ-factor [4] in this case 0.8, resulting in $\Delta\sigma_e = 47.7$ MPa. The fatigue category for this detail equals 56 MPa. Consequently, even if the concentration effect is not being considered, the test demonstrates that fatigue resistance is sufficient.

Conclusions

The original design of the Industry Lane steel two-hinged arch bridge was already 15 years old when its construction was actually started. Updating of the analysis made it clear that the low radius of curvature introduces an effect of lateral pressure, needing stiffening of the flange plates. However, for practical reasons this was replaced by more heavy flange plates. During the loading test, the effect of lateral pressure was actually measured.

The compensation of erection tolerances was analyzed in detail, according to the erection procedure as modified by the contractor. This demonstrated that the compensation was almost negligible. The load test also proved that stress variations and fatigue is the main issue with short steel bridges. Mainly those areas where bending is large are prone to fatigue damage. In the present case this location was near to the larger struts, which does not coincide with the maximum of ultimate limit stress.

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