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REFURBISHMENT OF THE MASONRY ARCH ON MERXEM STREET BRIDGE

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Due to the widening of the Albert Canal, several railway bridges were replaced by longer spans on new embankments. With a change in overall length profile, two railway bridges needed refurbishment. The Merxem Street Bridge, a classic masonry arch bridge, had a new tubular arch bridge built adjacent. Due to time, increased traffic, ballast, loads, and volumes on the bridge since the early 1900s, it was strengthened because of the bridge's historical value and structural soundness. A new concrete deck was installed on top of the masonry arch ensuring that the higher live as well as dead loads would be spread over the arch length. This new deck was a combination of precast concrete elements with on-site concrete construction. This research focuses on this combined concrete design and its realization. The ultimate load-carrying capacity of both the existing and strengthened situation was verified using rigid block analysis. Critical failure modes were checked and the strengthening avoided the most precarious modes.

Keywords: Precast concrete elements, Rigid-block analysis, Concrete deck, On-site concrete construction.

1 INTRODUCTION

This project concerns the Merxem Street Bridge (MSB), part of a larger project to widen the busy Albert Canal, connecting the Port of Antwerp with the Industrial area around Liege and offering access to the Ruhr area in Germany. The goal was to increase the vertical capacity of the canal, allowing larger barges to travel easily with containers stacked two high. A number of railway bridges needed to be replaced by longer spans on new embankments along the entire length of the canal. Also, since the widening necessitated a larger free height for ship traffic under these bridges, the overall length profile of this railway line had to be changed. This resulted in the replacement of two railway bridges near MSB. MSB, shown in Figure 1, is a classic masonry-arch bridge, using the bonded brick concept: the span consists of multiple rings of masonry that act as if they were one (e.g., if the barrel contains "header" bonded brickwork, where certain natural stone blocks are laid "end-on" to provide a mechanical connection between rings). Ten years ago, a new tubular arch bridge was built adjacent for the construction of a high-speed railway line. Due to time, increasing traffic, ballast, loads, and bridge volume, MSB needed repairs. Strengthening and refurbishment was chosen over the construction of a new bridge because of the historical value of the existing masonry arch and since the overall structural state MSB was still quite good.

Figure 1. The existing MSB before refurbishment (*right*); the new MSB for the high-speed railway line (*left*).

2 PRE-REFURBISHMENT

The original masonry bridge was built in 1925 as part of an expansion of railway freight capacity to the Netherlands. The vault of the bridge is a three-centered (pseudo-elliptic) arch profile, which assumes that the arch profile is formed from segments of three circles. This results in the arch intrados rising vertically at both abutments, and then swiftly changing orientation to end up with a large nearly vertical middle part. The masonry vault has a variable thickness, going from 0.85 to 1.32 m near the abutments, and is constructed using local Boom clay brickwork. The vault is strengthened over most of its height by lean concrete for a thickness up to 2 m, protected by an asphalt layer on top. The actual backfill consists of fine river sand compacted on installation.

The original ballasted track bed was built immediately on top of the extrados of the masonry arch vault. The distance between the railway tracks and the extrados is only 80 cm. Since the overall length profile needed to be heightened significantly, this would have resulted in a substantial increase of the dead load acting on the arch. This is the main reason why it was decided that the masonry arch needed strengthening, even though no real defects, cracks, or arch joints had formed in the original structure. Masonry arch bridges are statically indeterminate compression structures, which resist external applied loads primarily as a result of the thickness of the masonry and their inherent self-weight. They tend to be resilient to small support movements, with these typically transforming a structure into a statically determinate form. Cracks, which might accompany support movements, are therefore not normally of great concern, making the notion of crack widths or other conventional serviceability criteria not applicable. Consequently, the ultimate limit state is always the primary focus during design. This is typically put at risk when a sufficient number of hinges or sliding planes are present between blocks to create a collapse mechanism.

This paper uses the Limitstate RING 3.0 software package (Limitstate 2013) to do a rigid-block analysis of both the original and refurbished bridge. This software automates the limit analysis by choosing the most likely mechanism of collapse from all possibilities, using equilibrium to calculate the collapse load, and trying other likely collapse mechanisms until the critical one is found. The software also models the backfill, detailed abutment geometry, possible reinforcements, and ballast structure – each resulting in a very reliable prediction of the ultimate load limit. This is characterized by the adequacy factor. This factor is basically the factor which, when applied to the normal design load, would lead to collapse of the entire bridge structure.

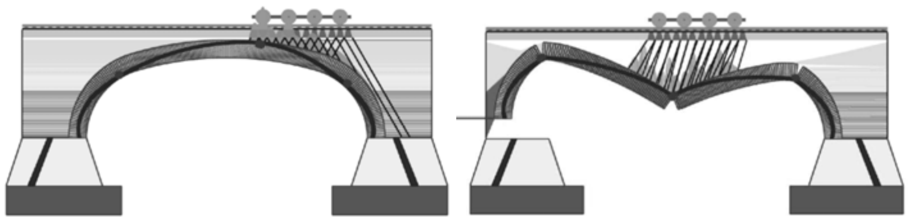

Figure 2. Failure modes of the MSB: when leaving the bridge (*left*, AF 2.11) and most critical position (*right*, AF 1.61).

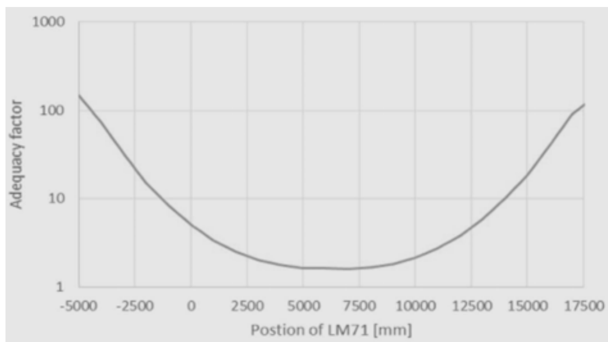

Figure 3. Adequacy factor for the original MSB for moving load model LM71.

The modeling is based on the application of Load Model 71 of the UIC. The four 250 kN axle loads at the center of this load model were placed in various positions as they run across the MSB. For each position, the adequacy factor, the location of the future hinges, and the variation of the compression zone along the length of the vault and the abutments were calculated. Two of the characterizing positions are shown in Figure 2. The first figure illustrates the situation just before the four-axle load leaves the span. The arch vault shows three possible hinge locations, typical for a pseudo-elliptic arch profile. The vertical intrados near the abutments further ensures that no hinge is introduced at that location, reducing the chance of movement of the supports. This makes this arch profile one of the more stable solutions, as well as extremely practical because of the large usable width, even at the height of the arch. The variation of the adequacy factor during the crossing of Load Model 71 is shown in Figure 3. It is immediately clear that when the adequacy factor is drawn using a logarithmic scale, the shape closely resembles the intrados of the actual arch, typical for masonry arches.

The second situation shown in Figure 2 represents the most critical position of Load Model 71. When the four axles are positioned in the middle of the span, the adequacy factor is reduced to only 1.61, which implies that the load reserve of these bridges under modern load conditions is only 61%. While this would have been more than sufficient in 1925 (when the load model would have been lighter), it is now no longer acceptable. Taking in mind the fact that the dead load will also increase because of the changed length profile and the additional backfill, the need for refurbishment and strengthening by installing an additional concrete deck atop the arch becomes clear.

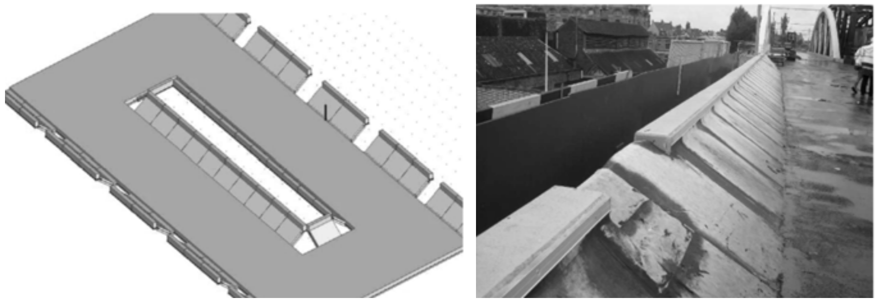

Figure 4. Illustration of the structural additions to the MSB: The structural concrete elements of the MSB before ballast installation.

3 REFURBISHMENT

To heighten the overall length profile of the railway line crossing the MSB, it was decided from a structural point of view to include a new concrete structure strengthened by prefabricated segmental concrete edge girders. The finite element model developed for the design of all structural concrete components is shown in Figure 4, consisting of prefabricated concrete elements. They are present at both edges of the bridge deck as well as in the central zone, where an opening exists in the existing bridge. The masonry arch is thus partially disconnected in the longitudinal direction of the bridge deck. The prefabricated elements have a cross-section that moves upwards, following and strengthening the existing edge members of the masonry arch. They will also act as a stiffening element for the structure, on which a cantilevering steel safety path with guardrail construction can be installed afterwards. The most important design problem for these elements was the shear connection between the individual elements, which had to be constructed on site using wired steel rods. The precast elements are shown in more detail in Figure 4. The elements, as well as the deck plate, are clearly visible.

Another difficulty when designing the additional concrete structure concerned the concrete plate, cast *in situ* in between the prefabricated elements. This concrete plate was cast on the existing backfill. It is assumed that the backfill can be characterized by medium-packed sand, with a spring constant within an elastic foundation of about 10 MN/m²/m. A sensitivity analysis was performed on the characteristics of the backfill. Because of the trainloads on the backfill for almost 100 years, it could be assumed that the sand would be very densely packed. However, dynamic train loads as well as the start of the refurbishment works could have disturbed this situation, making the assumption of a medium-packed sand a safer and better one. The length profile of the bridge deck rises along the length of the bridge because of the approach to the Albert Canal bridges close by, so the connection method of the precast elements with the *in situ* concrete varied along the length of the bridge. At one end of the bridge, as well as in the middle zone of the bridge span, a deck plate was installed at a higher level than the precast elements, allowing for the deck plate to be supported by the stiffer edge elements. The connection between both parts was realized using a hinged connection between both in the finite element model, transferring only vertical reaction forces between both structural elements.

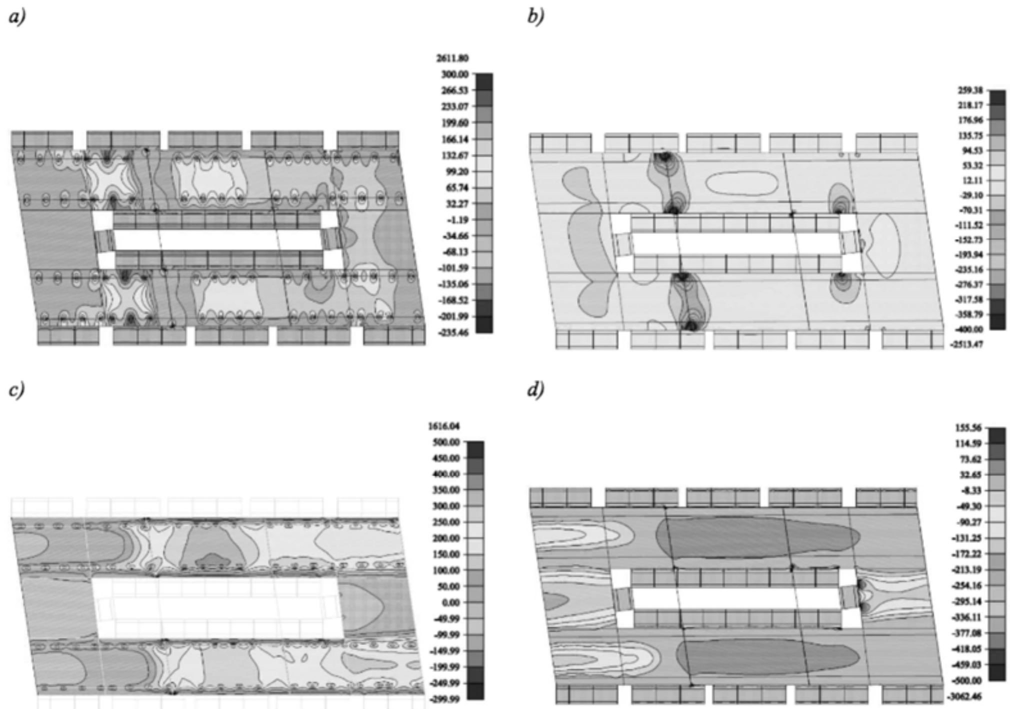

Figure 5. Bending moments in the concrete deck plate: (a) upper surface, bending along transversal axis; (b) lower surface, bending along transversal axis; (c) upper surface, bending along longitudinal axis; (d) lower surface, bending along longitudinal axis.

At the other end of the bridge, closest to the Albert Canal, the concrete deck plate was situated higher than the precast elements. No load transfer can occur between both concrete elements in this stage, so it was incorporated into the finite element model with an open joint of about 2 mm. The effect of this is clear by looking at the vertical displacements of the concrete deck under the quasi-permanent load combination. The displacements of the bridge on the right end of the bridge were significantly higher because of the connection loss with the edge members. The resulting bending moments at the upper and lower surface of the concrete bridge deck, along the longitudinal and transversal axis, are summarized in Figure 5. The influence of the load introduction of the rails at both ends of the sleepers is quite visible on the upper surface bending moments, even though a ballast thickness of about 55 cm is applied. This figure again illustrates the connection loss at the right end of the bridge between precast elements and the bridge deck: The distribution of the bending moments is clearly different and more uniform. The area where the free moving part of the deck plate is connected with the more supported area is also quite visible in Figure 5(d).

The strengthened version of the bridge deck was also subjected to a rigid-block analysis. Although the used software package does not allow introducing a concrete plate at surface level, the effect was nevertheless introduced. It was assumed that the effect of the concrete deck plate on the load introduction towards the arch vault would be comparable to a fictive ballast layer with an unrealistically high bending stiffness and angle of internal friction. This second alternative can be introduced in RING software without any problem. The resulting most critical failure mode is shown in Figure 6.

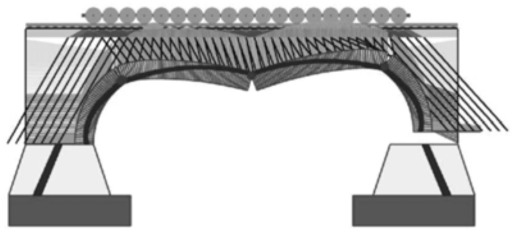

Figure 6. Failure mode of the MSB after refurbishment: AF 5.39.

4 CONCLUSIONS

The most obvious effect of the strengthening operations is that the overall adequacy factor of the bridge has risen from only 1.61 to 5.39. This increase of more than 300% allows for a safety level more in line with the design assumptions of the Eurocodes (ENV 1992-1 1992, ENV 1991-3 1991), and is realized even though the total dead load acting on the arch bridge is increased significantly, because of the additional backfill, concrete structure and ballast thickness.

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ENV 1991-3, *Eurocode 1 Basis of design and actions on structures – Part 3 : Traffic loads on bridges*, CEN Brussels, 1991.

LIMITSTATE: *Ring Manual Version 3.0f*, LimitState Ltd., Sheffield, United Kingdom, 2013.