INTEGRATED STEEL VIADUCTS FOR RAILWAY IN EXTENSION OF A HISTORIC MULTIPLE-ARCH CONCRETE VIADUCT

Bart De Pauw
Ghent University, Belgium

Philippe Van Bogaert
Ghent University, Belgium

ABSTRACT

The existing railway line between Brussels and Ghent crosses the valley of the Pede by a 523 m long historic viaduct. This structure of the 1930's consists of 16 three-hinged reinforced concrete arches of 32 m span, reaching a maximum height of 40 m and supported by hollow concrete piers. The railway company started infrastructure works in increasing the number of tracks of the railway line from 2 to 4 tracks. Widening of the structure by two additional lateral viaducts was only acceptable if these new constructions are respectfully integrated in the historic work of art. The paper describes the final design of new steel viaducts and its supports consisting of steel hollow ribs, which are fading gradually into the existing piers. In advance, the existing foundations were considerably reinforced by additional grouting piles. The design of the superstructure dealing with severe criteria for deformations and accelerations for railway structures consists of a steel box with variable hollow section. The box section is rectangular at the piers and the upper flange is constant along each span. However, the lower flange rises as it reaches the span centre obtaining less section height, while the flange is twisted about a horizontal axis and becomes wider. This created a waving pattern of the steel structure, both in a horizontal plane as in the front view complying with the existing arches. Per track, the total steel viaduct consists of 4 hyperstatic structures having a total length of 130.75 m each, thus being continuous over 4 spans of the historic bridge. On top of the upper flange a concrete deck plate carrying ballast is foreseen. Recently, the construction of the modern structure, showing clearly the contrast with the ancient arches, has been started.

1. INTRODUCTION

A project of extension of suburban and regional railway lines around the capital Brussels is now under construction in Belgium. One axis of this network is in the Western direction and consists of extending a 25 km double track line to 4 tracks. The existing railway crosses the valley of the river Pede, where soil conditions are very poor. The valley is presently crossed by a 523 m long historic viaduct, which was built from 1933 to 1936. The viaduct was chosen over large backfills in the valley since the poor soil conditions. At present the viaduct cannot be seen over its full length of 523 m, although it is imposing in the surroundings. Figure 1 gives a view in front of the central part of the concrete structure. The viaduct is a benchmark in the rural environment, through its dominance midst the gentle slopes of the Pede valley. The viaduct itself consists of 16 concrete 3-hinged arches with 32 m span. Although they are scarcely seen from below, the arch hinges are remarkable since they do not contain any device and are completely of reinforced concrete. Four arches are an independent group, since they are separated from the rest of the viaduct by double pier structures, allowing for compensation of the thrust force of each group. This is not apparent, since the viaduct is seen as a continuous structure of 16 arches.
In addition to the structural characteristics, the viaduct is remarkable for its materials and texture. The concrete is of a yellow colour, unfortunately now becoming black and suffering from carbonation. The concrete surface also is rather rough and still shows the wooden formwork used while building the viaduct. The reinforcement is of smooth bars with hoop anchoring, of mild steel. All of these characteristics make this viaduct an excellent example of early reinforced concrete building practice from the 1930's. A detail of the piers and arches is given in Figure 2. Extension of the railway facilities to 4 tracks will need to widen the structure by two additional lateral viaducts. In view of the rather exceptional qualities of the viaduct, this operation will need taking care of the existing structure and integrate any new element respectfully into the historic work of art. In extending the viaduct, the concrete structure will be renovated simultaneously.

2. VIADUCT TRANSFORMATION

2.1 Criteria for Viaduct Transformation

In dealing with the problem of widening this structure from 2 to 4 tracks, an important condition seems to be the presence of the existing piers as well as keeping the view of the massive structure. In addition, the arch structure also should be left apparent, as much as possible. Also, the arches should keep their function and continue to behave as a four-span group. This may be achieved by looking for alternatives which do not imitate the arches nor try to find modern versions of this structure and its materials. Important guidelines in making the new design are the contrast of old and new technology, keeping the four-span static behaviour with the repetition of 32 m spans, bearing in mind the special texture of the concrete and leaving the characteristic view of the hollow piers. The new structure must not be of imposing character.
2.2 Final Design

During the pre design, two alternatives with independent pier structures as well as two alternatives with integrated cantilever pier structures were considered. It could be concluded that in respect to the existing structure, the latter option was to be chosen. The final design consists of a steel superstructure having variable hollow sections. The box section also is continuous over 4 spans and it is characterised by waving patterns, both in the plan (Figure 5) as in the cross-sections (Figure 4). The upper flange of the box section is constant and stays horizontal along the structure’s length. The lower flange has variable width, minimum 3.65 m at the piers and maximum 5.15 m at the span centre. In addition, the lower flange climbs according to a sine wave from the supports towards the span centre obtaining less height and is twisted about a horizontal axis as it becomes wider. As a result, the vertical cross web near the existing concrete arches has variable height, whereas the outer web incidentally shows torsion along the bridge axis. This can be clearly seen in Figure 4 and Figure 13. This created a waving pattern of the steel structure, complying with the existing arches, both in a horizontal plane as in the front view. These waving patterns match with the concrete arch repetition.

![Simulated view](image1.jpg)

Figure 3. Final alternative with steel box section

![Cross section](image2.jpg)

This alternative is qualified as being an honest structure, showing as much respect for the historic viaduct. A simulation is illustrated in Figures 3a) and 3b). Since a steel structure is used for the bridge deck as well as for the infrastructure, a contrast of materials is being created between the rough concrete and the smooth modern steel construction.

The new superstructure is supported by a steel corbelling construction, fixed to the existing piers. This has seemed a better choice than a concrete pier, as the concrete texture would not match with the existing one. The steel piers are joined by a steel transversal beam, located in the hollow parts of the existing piers. The vertical pier has a conical shape and fades into the lower part of the concrete pier. Three years ago, works has started. In a first phase, the strengthening of the foundations by extending the existing foundation slab has been carried out. In a second phase, the construction of the steel piers and superstructure has been started a year ago. The paper describes the design of the strengthening of the foundations, the piers and the superstructure and gives a view of the current stage of the construction.
3. FOUNDATIONS AND PIER STRUCTURE

3.1 Strengthening of the Foundation

Since the new superstructure is supported by a construction which is fixed to the existing piers, strengthening of the existing foundation is needed due to the additional weight. The 15 impressive concrete piers of which 12 piers have a thickness of at least 3.5 m and 3 piers have a thickness of 5.5 m (further mentioned as "double pier structures"), are founded on concrete footings showing a cross-section of 9.1 x 12.1 m in the case of a standard pier and 9.1 x 15.1 m in the case of the double pier structures. These footings are founded on concrete rectangular piles of cross-section of 0.35 x 0.35 m. In addition, the individual footings are joined together by 6 connection beams. These beams are situated at a level of 2 m under the ground. The abutments are similar since they are founded on approximately the same level as the piers. Research of the ground characteristics learns that a ground layer with sufficient carrying capacity is situated on a depth of 11.5 m up to 17.5 m under the ground. Likely, the existing piles are founded on that layer. In the symmetrical extension of the viaduct, strengthening of the existing foundations is realized by grouted piles, drilled around the existing footings. This technique has the advantage of making foundation elements without large construction wells or work platforms and limits thus the hindrance to the environment. The grouting piles can be made with small machines and no remarkable lowering of the groundwater in delicate zones is needed. In this solution, the grouting piles are designed to replace the existing concrete piles. To carry the additional load, the grouting piles are founded on a deeper level as in the case of the existing piles. After drilling the grouting piles as can be seen in Figure 6b), the existing foundation slab was extended by a new concrete slab around the existing foundation. Both new and existing slab were put together by post-tensioning cables. This is illustrated in Figure 6a).
3.2 Pier Structure

As earlier mentioned in designing the extension of the piers, a steel corbelling conical pier construction was chosen over several other designs. Showing respect on both the geometric as well as the static functions, a steel pier construction fit better than a concrete pier. In the final design, a rectangular box section in conical shape has been designed. These steel piers are illustrated in Figure 7. The conceptual design of this pier has to deal with both vertical and horizontal forces. At first for horizontal stability in transverse direction, a transverse steel connection structure between the two cantilever piers and located in the hollow parts of the existing piers is needed. As for the horizontal stability in longitudinal direction, the horizontal traction and braking forces of the new structure on each cantilever pier is incompatible with the slender design. Therefore, the new structure is made continuous over 4 spans. This makes it possible to lead the high horizontal forces to the piers with higher thickness (or the double pier structures having a thickness of 5.5 m). These double pier structures are situated each 3 standard piers of 3.5 m thickness, thus making the new steel superstructure continuous over 4 concrete arches or 4 spans.

In designing the pier construction, the limitation of the transverse distortion of the pier is decisive. Due to the eccentric railway load on one track, the portal frame bends on the side where the charges are applied and rises at the unloaded side. This is illustrated on the scheme in Figure 8a) as well as in Figure 9 showing the results of the
deformations of the finite element model of the double pier structure under mobile forces. Also a considerable horizontal displacement appears. In the initial design, only an upper transverse beam connecting the two cantilever piers appears to be insufficient.

![Diagram](image1)

a) Deformations in the finite element model

![Diagram](image2)

b) Final design

**Figure 8. Pier structure**

Therefore a solution is found in using an internal framework to be placed in the internal hollow part of the existing pier. This is illustrated on Figure 8b). The framework cannot be made infinitely rigid by using for instance a cross connection in the inner part of the central frame. Also such a cross connection is unacceptable regarding the respected view through the hollow part of the existing piers. It could be concluded that regarding the limitation of horizontal and vertical displacements due to an asymmetric loading of the frame, the stiffness of the framework is decisive. Therefore, in the final design, the influence of the superstructure at the unloaded side is taken into account. At that side, the deformation of the portal frame is not entirely free, experiencing the favourable influence of the unloaded viaduct. The bearings of the adjacent viaduct, which are fixed in a transverse direction, obstruct a horizontal and vertical distortion of the pier. This influence is charged by means of a horizontal spring with horizontal stiffness $k_h$ and by means of vertical spring with vertical stiffness $k_v$, which is determined by the elastic deformation capacity of the superstructure. Figure 8 gives an overview of the simplified FE-model (Figure 8a)) and the design of the conical pier construction (Figure 8b)). In reality, Figures 10a) and 10b) illustrate the outer and inner parts of a single pier structures under construction. At the present, some single piers have been installed as can be seen in Figure 11.

![Diagram](image3)

**Figure 9. Vertical deformations of the “double” pier structure under mobile forces**
4. STEEL SUPERSTRUCTURE WITH VARIABLE HOLLOW SECTION

4.1 Cross Section

The specific design of the superstructure determines in large degree the cross-sections. This design of the new superstructure is acceptable if the total construction height remains limited in order to guaranty a full front view on the existing viaduct. Evidently, the restriction is not unlimited dealing with the strict serviceability criteria and dynamic analysis for railway bridges according to the Eurocode (ENV 1991-3). In the further design, these criteria are decisive comparing with the criteria of strength. It concerns not only the restrictions in vertical deformations for safety purposes, but also the transverse deflections, especially the twist of the deck, and the end rotations of the deck. Figure 12 gives the cross-sections of the superstructure at the piers and at mid span. As can be seen on the illustration on the left, the outer web plate of the steel superstructure shows torsion. During construction, this outer plate will be pre-deformed by torsion as shown in Figure 13. In this configuration, the restriction of the vertical deflection $\delta/L$ to a maximum value of 1/1090 for a train speed of 160 km/h and the restriction of the rotations at the end of the deck to $6.5 \times 10^{-3}$ radians for ballasted track can be fulfilled. The deformation of the dead weight is illustrated in Figure 14. This is made possible due to the continuity of the superstructure over 4 spans and the use of
a concrete deck-plate. A disadvantage of this continuity is that the influence length for traction and braking forces is higher. Since the slenderness of the standard steel pier structure, it seems logical to lead the additional horizontal traction and braking forces to this existing massive pier structures (double pier structure) having a thickness of 5.25 m. For this pier, the use of a wider new steel pier structure each 4 spans does not disturb the normal pattern, one remains in the same philosophy of the original design.

Figure 12. Cross sections

Figure 13. Pre-torsion along the bridge axis of the outer web plate of the superstructure

An additional stiffness is realised by providing a concrete deck plate of a thickness of 0.25 m as can be seen in the cross-sections of Figure 12. As well, the concrete deck foresees in spreading the charges, avoiding direct traffic loads on the upper steel flange. So, local fatigue problems can be limited. However, in the area near the supports, on the part of the concrete deck under uniform tensile stresses cannot be counted. Therefore, the parts of the concrete plate of the continuous structure under tensile stresses larger than the admissible concrete tensile strength are omitted in the calculations (iterative process). However, the zones of traction remain local.
4.2 Transverse Distorsion

Due to the restricted construction height, the slender cross-section at mid span has limited strength to torsion effects. This brings concerns about the transverse distortion, such as the twist of the deck which must be strictly limited for railway bridges. To raise the torsion stiffness, internal stiffening is used. In the final design, the use of longitudinal stiffening by internal webs, as well as the use of diaphragms on very short distances, has been investigated. The last solution is clearly preferable, also due to its favourable influence during the construction of the superstructure. This construction method exists in constructing the steel deck upside down. First, the internal diaphragms can be welded on the horizontal “upper” steel flange. Secondly, the curved “under” flange can be pushed towards and welded on the diaphragms. This can also be applied for the curved outer web. A deck strengthened with internal webs would lead to complex and uneconomic cut outs of the web plates in following the curving. The contact length is lesser than in the case of diaphragms. Moreover, the increase of the torsion stiffness is small in comparison with strengthening with diaphragms. For example in the case of an isostatic deck with a span of 31.35 m, on the one hand strengthened with diaphragms and on the other hand strengthened with internal webs of a comparable steel quantity, the relative distortion Δy of the two outer points of the under flange is 3 to 4 times larger in the case of web stiffening. A comparison is given in table 1.

| Table 1: Comparison of the transverse deformation of an unstiffened and stiffened cross section |
|-----------------------------------------------|-----------------|------------------|
| Thickness stiffening [mm] | Deformation on the left y_A [mm] | Deformation on the right y_A [mm] |
| Without stiffening | - | 39.8 | 116.9 |
| Longitudinal Web Stiffening | 20 | 54.6 | 66.8 |
| Longitudinal Web Stiffening | 30 | 54.4 | 63.8 |
| Diaphragm | 20 | 63.1 | 65.8 |

In the final design, an "optimum" for the diaphragm distance has been investigated, avoiding unnecessary adding of steel. In the graph of Figure 15, the relative deflection Δy=y_A−y_A of the two outer points of the under flange is reflected to the diaphragm distance. This is expressed in the thickness of the diaphragm (in millimetre) diverted by the diaphragm distance in meter. The transverse distortion increases exponentially regarding to the “relative” diaphragm distance. In the design, diaphragms of 20 mm thickness on a distance of 2 m were chosen. These diaphragms are foreseen with openings for construction and inspection reasons.

198-9
Figure 15. Relative transverse distortion at mid span of an isostatic deck with different internal stiffening.

5. REFERENCES

