PRECAST HPC POST-TENSIONED ALTERNATIVES FOR A RAILWAY CROSSING VIADUCT

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ABSTACT

Near the Luchtbal district, a 1982' long fly-over has to be designed for the high speed train line between Antwerp and the Dutch border, which crosses an existing railway line with a rather shallow angle. A first design possibility is characterized by precast, prestressed hollow slab-girders of maximum 85' 11.5'' span, which are transversely post-tensioned on site. In this design, a concrete type of C80/95 is applied. The second design, as proposed by the contractor, reduces the spans to 63' 11.72'' and foresees all girders to be cast on site. For the standard decks, only longitudinal post-tensioning is applied, and a concrete type of C50/60. The absence of transverse post-tensioning is counteracted by a 3-dimensional cable path, making this a more complex design for fabrication and calculation. For both alternatives, the solution for the decks near the crossing, with their asymmetrical bearings, is described.

Keywords: HPC, multidirectional curved post-tensioning, prestressed concrete, fly-over, railway crossing, precast elements

INTRODUCTION

This paper describes two possible solutions for the construction of the deck for a viaduct in the northern part of Antwerp, enabling the Antwerp-Amsterdam high-speed railway-link to cross with a domestic track, on its way through the Luchtbal district. The fly-over viaduct consists of spans of different lengths, spanning a total distance of 1982’ (604 m). Specific local conditions, mostly from an environmental nature, necessitated the trajectory to follow the existing motorway, leaving Antwerp for the Netherlands. To allow for this precondition, the tracks coming from the Antwerp North-South Junction[1], ending with two twin 5250’ (1600 m) long bored tunnels in the northern part of the city, need to cross a domestic passenger track.

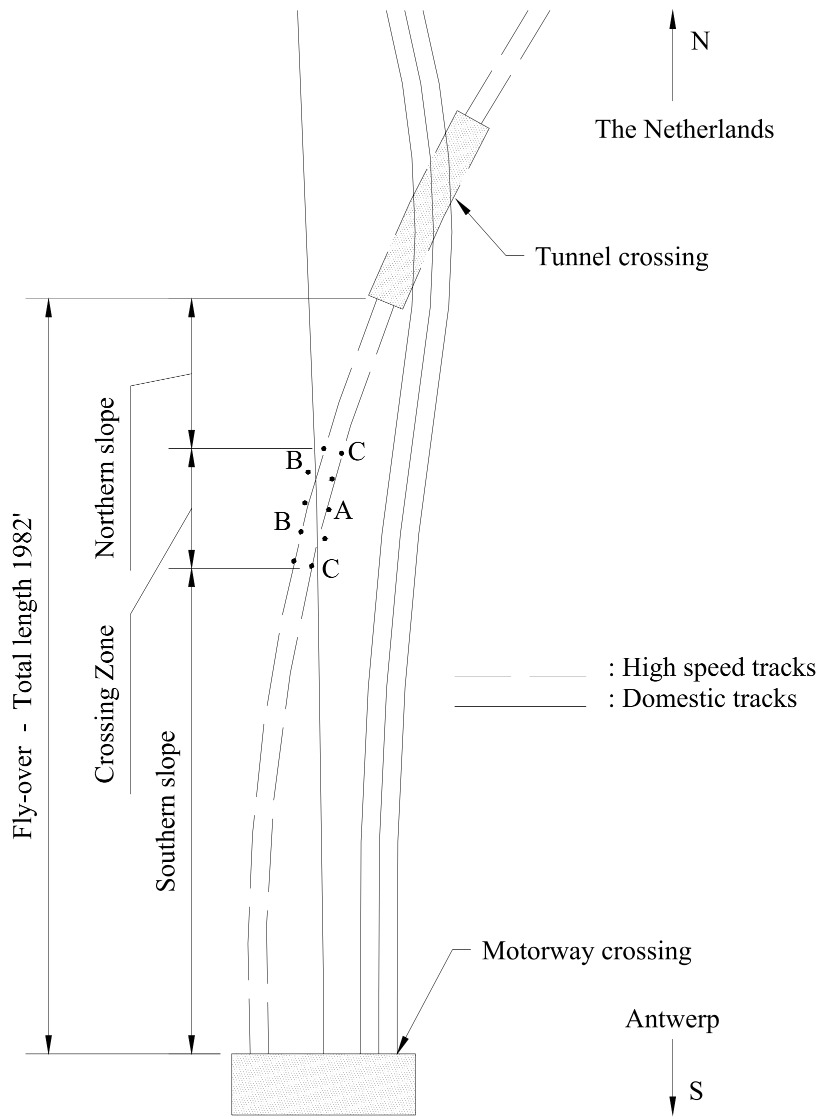

Fig. 1 Situation of the Luchtbal viaduct

The geometry of the crossing of which the plan view is shown in fig. 1, was determined by such simple necessities as a minimized construction depth and a maximum track gradient. The construction depth of the overpass superstructure was highly reduced, and although for track gradients for high-speed railway traffic higher values are allowed than for normal railway lines, they still needed to be kept within sufficiently moderate limits. The high-speed tracks must climb upon leaving the city towards the railway crossing, starting from a crossing with the motorway to the Netherlands. The domestic track going under the railway crossing needs to pass this motorway at the same location, resulting in a descend on the approach to the city for both tracks. After the crossing with the underlying tracks, the high-speed line must immediately descend to enter a tunnel, where it meets three other domestic tracks on their way to the north. This quick succession of a steep inclination, followed by a just as steep declination, necessitated the reduction to the absolute minimum of the construction depth for the viaduct to about 3’ 3.37’’ (1 m).

Since part of the argumentation behind the choice for the trajectory along an existing motorway, was the reduction of noise problems for neighboring agglomerations, this crossing through the northern part of Antwerp, full of high-rise living facilities, necessitated the installation of heavy noise barriers along both sides of the tracks on top of the viaduct, as shown in fig. 2.

The first design possibility to construct the deck of this viaduct was by using specially made, hollow section precast high-performance concrete twin elements, using three types of prestressing. This solution needed special attention in the area of high shear effects. The second possibility, which was developed at the specific requests of the contractor, consists of a deck without hollow sections in its cross-section. This deck would be constructed on site, but necessitated a three-dimensional post-tensioning, i.e. post-tensioning cables that follow a three-dimensional path along the length of the axes of the viaduct. The post-tensioning of the crossbeams, necessary in certain cross-sections proved the biggest difficulty of this design.

FIRST ALTERNATIVE: A BRIDGE DECK USING PRECAST, PRESTRESSED HOLLOW HPC ELEMENTS

GEOMETRY OF THE FLY-OVER

The viaduct, which starts in the south at the platforms comprising the crossing with the approach roads to the Antwerp-Breda motorway, should reach north at least as far as to where the crossed and adjacent domestic railway tracks are situated with a large enough distance in between to allow the construction of a normal railway installation to enter the tunnel below the three adjacent tracks (see fig. 1). The crossing of the lower track along a sharp angle clearly constitutes the decisive point for the construction’s design. For determining the necessary distance between the piers supporting the deck, one has to allow a distance of at least 10’ 7.95’’ (3.25 m) between the nearest rib of the pier and the axis of the lower track. This condition necessitates a distance of 135’ 3.62’’ (41.24 m) between the crossing of the axes of the two tracks and the first southward pier, when only a symmetrical placement of both columns is allowed. However, because of the sharp angle between the domestic and the high-speed track, it is possible to divide this distance in two equal parts, north as well as south of the critical crossing point, thus allowing the design of a sufficiently slender superstructure, but necessitating the asymmetrical placement of the columns of two piers (type B), resulting in a more complicated support structure, the piers of which are shown in fig. 1 and fig. 5. This argument results for the southern part of the viaduct, having a total length of 1425’ 6.3’’ (434.5 m) between the south abutment and the northernmost symmetrical pier, in 15 equal spans with a pier distance of 86’ 2.25’’ (26.27 m) and a span value of 82’ 4.98’’ (25.12 m). These values were also used for the northern part of the viaduct. This results in a viaduct, consisting of, from south to north, 15 equal spans of 82’ 4.98’’ (25.12 m), followed by 2 spans of 64’ 0.50’’ (19.52 m) and 2 spans of 57’ 9.70’’ (17.62 m) and finally ending with 5 spans of 82’ 4.98’’ (25.12 m). The crossing zone is then comprised of the 4 smaller spans.

STRUCTURAL CONCEPT OF THE DECK ELEMENTS

The span length derived above, combined with a limited construction depth, demands a compact cross-section. In combination with the need for a design with a minimal weight, this leads to a cross-section comprised of hollow slab elements, shown in fig. 3. The proposed solution, see fig. 2, consists of two hollow slab elements, which are assembled by transverse post-tensioning bars, while the walkways are precast elements as well, but placed on the slanted sides of the bridge deck, pinned in place in encased steel plates. The full parapets of these walkways, functioning as noise barriers and equipped with concrete cams, are fixed in place later on by concreting an edge rib on the walkways.

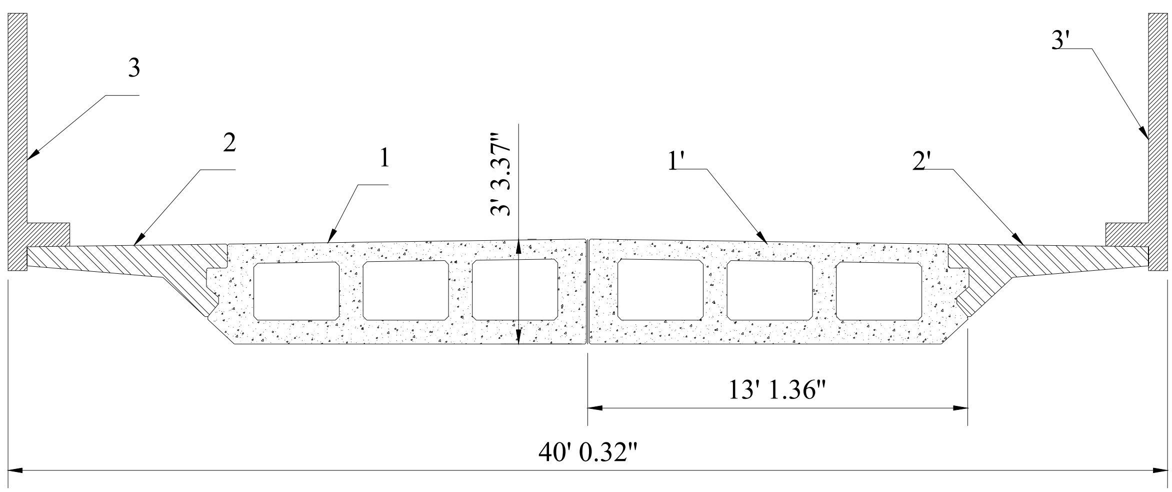

Fig. 2 The different elements of a typical cross section
(1 & 1’: precast hollow box elements; 2 & 2’: walkways; 3 & 3’: parapets)

The lower parts of the hollow box sections are prefabricated, using pretensioned strands. One precondition of this construction method is a limited number of strands, because of the maximum prestressing force of the production units. Another precondition is a limited mass, smaller than the highest allowable lifting weight of the rolling cranes, necessary to transport the elements. The practical values of these limits were a maximum of 210 prestressing strands for each element and a maximum total mass of 130 tons per element.

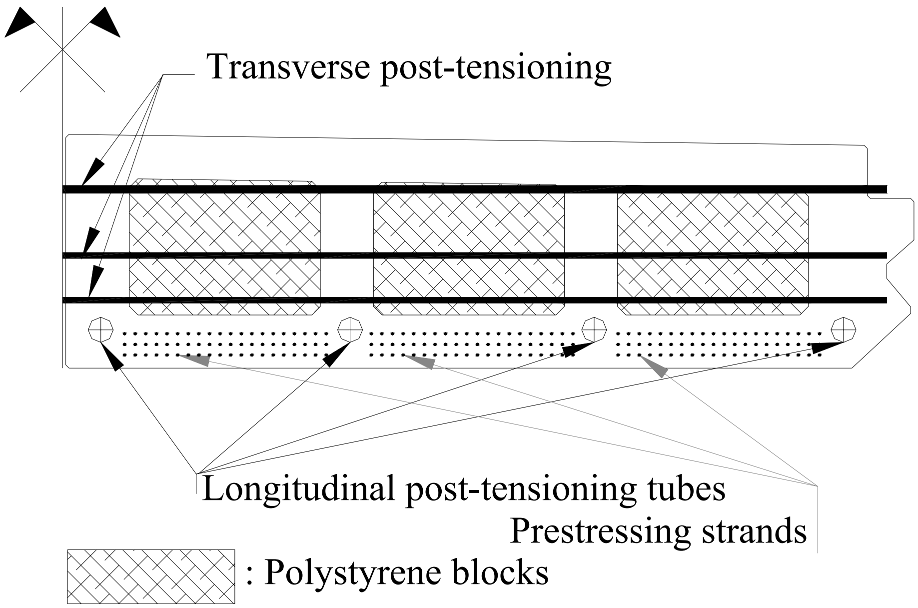

Fig. 3 Different types of tensioning in a typical precast element

The cavities in the hollow box sections are formed by placing polystyrene blocks after the prefabrication of the lower parts, so the top slab can be cast on site. After hardening of the complete box section, additional prestress is activated using 4 additional longitudinal post-tensioning cables. The total result of the pretensioning strands, combined with the mass of the first phase concrete, is a compression of the lower slab of 3144.42 psi (21.68 MPa). This compression is decreased slightly by the second phase concrete, but this effect is counteracted by the additional post-tensioning cables, who introduce supplementary compression even when allowing for a slight reduction of the efficiency of the strands, due to the bonding, estimated at 7%. The remaining stress, after the post-tensioning is 5214.11 psi (35.95 MPa). This value is reached, once the concrete of the lower slab has reached sufficient age as to limit the subsequent creep[2], i.e. during the second phase of the fabrication. This stress level can be accepted temporarily if the compression strength has reached f_{cj} = 10152.64 psi (70 MPa) at this age of the concrete. The concrete used for the precast elements is thus necessarily of the type C80/95, which is a designation according to the Eurocode[3], corresponding with a compressive strength after 28 days on test cylinders, 5.91’’ (0.15 m) in diameter and 11.81’’ (0.30 m) in height, of 11603 psi. (80 MPa)

The bearings and pier columns, which directly support the precast elements in normal sections, are not connected by a horizontal beam. Because of this, the bridge deck itself must resist the transverse bending in between the bearings, since no portal frame action in the piers can be counted on. This bending effect requires the use of transverse prestressing, assembling the two halves of the superstructure. The easiest way to supply this cross tensioning force is the use of post-tensioning bars. The transverse prestressing force is required to reach its maximum value at the bearings. The location of the prestressing strands and the longitudinal post-tensioning strands, as well as the transverse post-tensioning bars are shown in fig. 3.

THE SPECIFIC ELEMENTS NEAR THE CROSSING ZONE

The longitudinal post-tensioning is not needed in the bridge deck elements near the crossing zone, because of their smaller span.

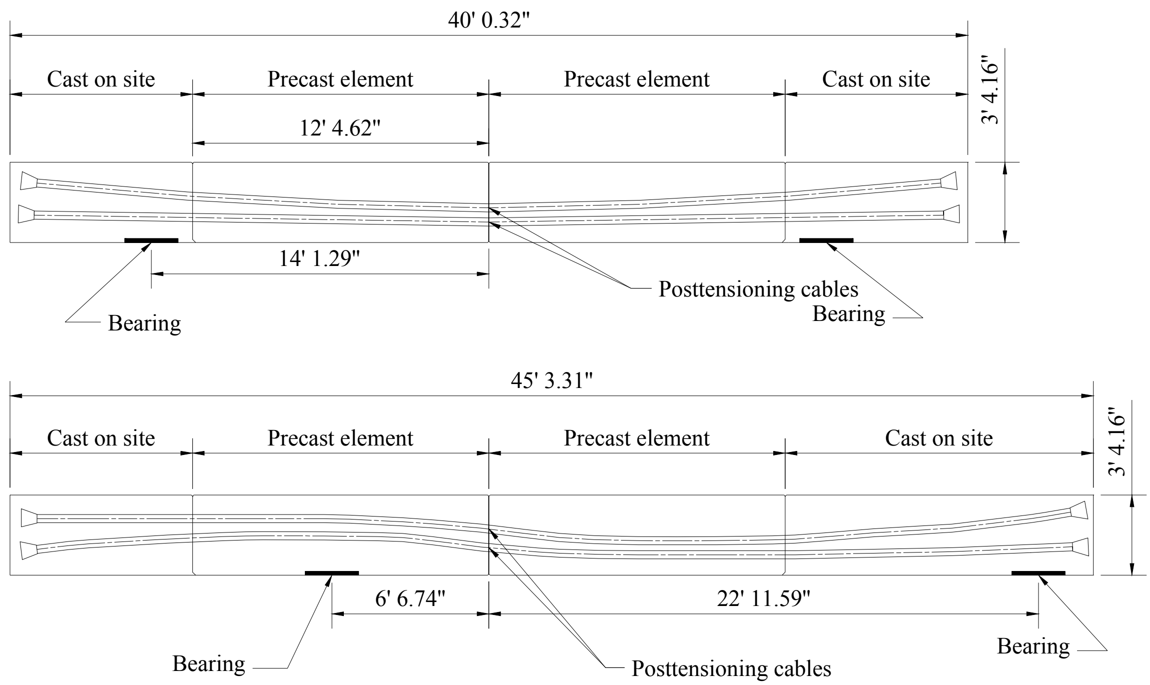

Fig. 4 Post-tensioning cables of the crossbeams at piers A (upper) and B (lower)

As the cross section approaches the crossing zone with the underlying track, the bridge deck will move away from the for the underlying track clearance acceptable position for the pier columns. The positioning of the piers can even result in a cantilevering position of the bridge deck. In this situation, the assembling of the hollow elements with prestressing bars will not be sufficient to absorb the transverse bending moments near the piers. For these spans, the use of an end crossbeam will become inevitable. This crossbeam is constructed by adding the cantilevering part to the precast elements and using transverse post-tensioning cables. Since there was no need for longitudinal cables in the smaller spans, there is sufficient room in the cross section for the transversal post-tensioning. The installation of the post-tensioning cables in the ribs of the cross section allows for complete covering and injection of the cable ducts and for the use of second degree parabolas for the cable trajectories. Nevertheless, these precast elements, have a total mass of 190 tons, once completed.

The twin columns of the piers are at a larger distance in the crossing area and thus connected by the crossbeams discussed above and shown in fig 4. Pier type A shows a symmetrical position of the columns, while at pier type B an asymmetrical installation becomes necessary. Pier types A, B and C are displayed in fig. 5 and can be located on the map in fig. 1.

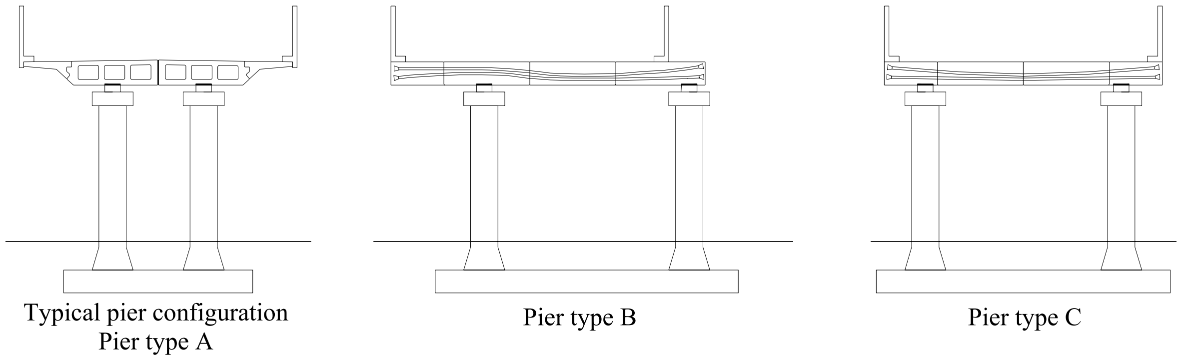

Fig. 5 Different pier configurations

The use of transversal post-tensioning bars and cables clearly increases the shear strength of the superstructure's cross section. However, the previous statement only becomes reality once the post-tensioning force is activated. Before that, shear resistance problems can exist at the deck ends. During the installation of the hollow elements, they are only supported by the element's single bearing. At this construction phase, punch can occur in the area surrounding the bearing. The by the punch effect caused tensile stresses in the element cannot yet be effectively compensated by the compression caused by the longitudinal post-tensioning cables in the direction of the bridge axis. In the perpendicular direction too, the post-tensioning bars will only compensate the punch effect, once they are activated at a later stage of the construction. The Eurocode states that the increase of the shear resistance is not proportional to the compression strength, and thus should be limited to a value corresponding with a compressive concrete strength of 7252 psi (50 MPa) or a concrete class C50/60 according to the Eurocode[3]. The effects of ballast and trainload are however not yet effective in this early situation when post-tensioning has not occurred yet, as a result of which even this reduced shear resistance, already existent in the elements, is sufficient to withstand the punch effect.

DYNAMIC CONSIDERATIONS

The viaduct is located close to the Antwerp North-South Junction, necessitating a speed limit of 80 mi/h (130 km/h). The interaction between the suspension of the railway carriages and the support structure will not cause any resonance effects, since most of the spans have an equal length and all are not much larger than one single train car. However, a study was made of the transient behavior of a train passage. The ratio of the span length to the deflection caused by "Load Model 71" [4], i.e. the relative stiffness of the viaduct, equals 780. The recommended value being 900, this classifies the viaduct as being quite flexible. However, the footway elements and the parapets on the sides, which were not considered in the aforementioned calculation, will improve the stiffness of the cross section significantly.

Passenger comfort restrictions imply that the instantaneous value of the train car acceleration remains below 3.8084 ft/s² (1 m/s²). However, even when the stiffness of the walkways and parapets is neglected in the calculations, at a speed of 80 mi/h (130 km/h) only an acceleration of 1.71 ft/s² (0.52 m/s²) is reached.

SECOND ALTERNATIVE: A CONTINUOUS BRIDGE DECK USING MULTIDIRECTIONAL CURVED POST-TENSIONING

THE GEOMETRY OF THE FLY-OVER

This solution faces the same specific problems as the previous one, more specifically the sharp angle of the crossing with the underlying domestic track and the minimal construction depth because of the steep inclination to and from the crossing.

Although this fly-over design has the same total length, namely 1982’ (604 m), the division in different spans changes because of the choice for a significantly shorter span length. This results in 20 standardized spans for the southern part of the fly-over with a span value of 63’ 11.28’’ (19.49 m) and a pier distance of 67’ 8.56’’ (20.64 m) each. This first zone is followed by the four special decks of the crossing zone, with a span value and pier distance for the first one of 63’ 11.28’’ (19.49 m) and 67’ 8.56’’ (20.64 m), 57’ 7.85’’ (17.57 m) and 61’ 5.13’’ (18.72 m) for the second one and finally 63’ 10.61’’ (19.47 m) and 67’ 7.89’’ (20.62 m) for the two last decks. All four previous decks have an asymmetrical pier positioning. The viaduct ends in the north with 6 equal decks with again asymmetrical pier positioning and span lengths and pier distances of 57’ 9.43’’ (17.61 m) and 61’ 6.70’’ (18.76 m) each. This adds up to a total of 30 spans or 29 piers, against 24 spans or 23 piers for the first alternative.

STRUCTURAL CONCEPT OF THE BRIDGE DECK

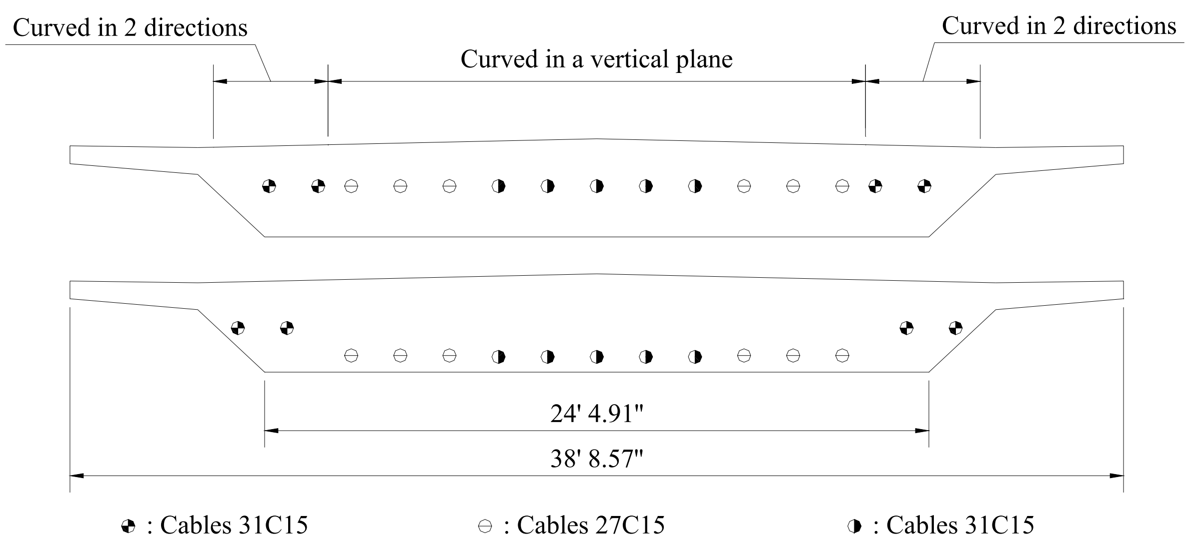

Fig. 6 Cross section at end span (upper) and mid span (lower)

The basis of this second design alternative, which was developed at the specific demand of the contractors, is a reduction of the span length to 63’ 11.28’’ (19.49 m) and the wish to cast all elements on site. These changes allow for the use of a concrete type C50/60, and do not impose the use of transverse post-tensioning in the standard bridge spans away from the crossing zone. The above mentioned concrete type corresponds with a cylindrical compressive strength of the concrete of 7252 psi (50 MPa) after 28 days, according to the relevant Eurocode[3].

However, to counteract the absence of transverse post-tensioning, it becomes necessary to install some of the longitudinal post-tensioning cables along a 3 dimensional cable path, adding an extra difficulty to this design. (See fig. 6 for cross sections of the elements and fig. 8 for a 3-dimensional view)

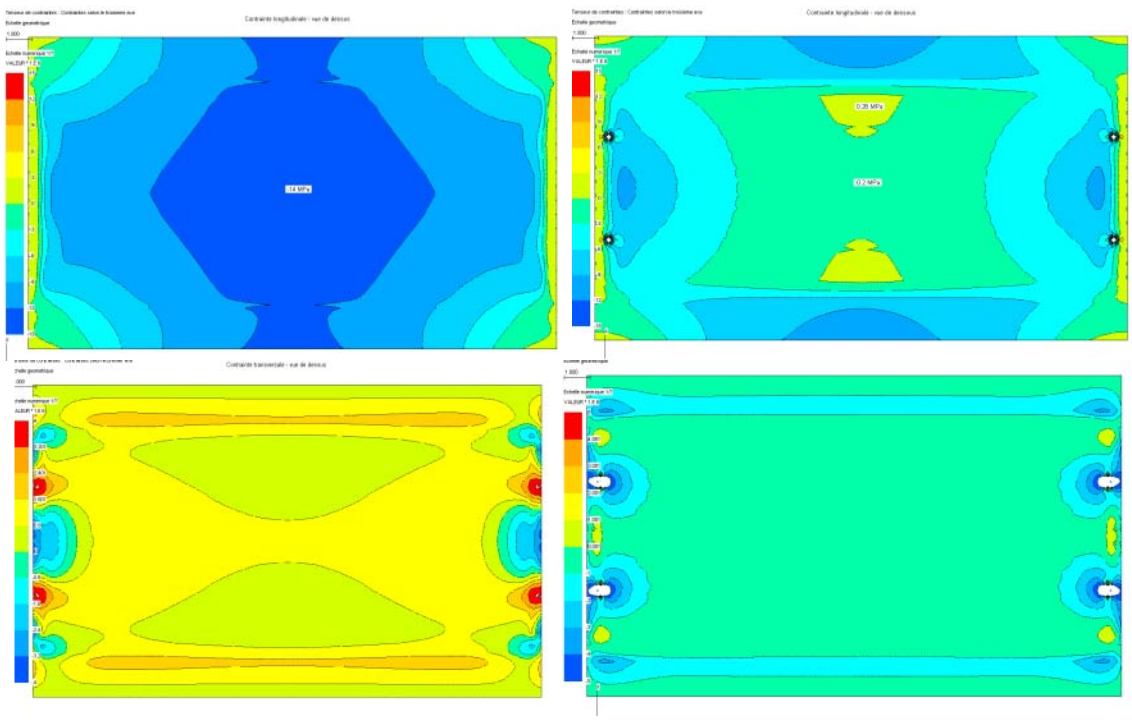

Fig. 7 Output of the finite element software: longitudinal normal stress at the upper fiber (upper left: -2176 psi to 2176 psi) and at the lower fiber (upper right: -2176 psi to 2176 psi) and transversal normal stress at the upper fiber (lower left: -580 psi to 580 psi) and at the lower fiber (lower right: -725 psi to 725 psi)

This multi-directional post-tensioning made the design much more complex, requiring the use of a detailed calculation model, using finite element software. A few examples of the output of the used program are shown in fig. 7 (values of the color scales are mentioned between parentheses). For cases like these, the Eurocode[3, 5] demands that no decompression of the concrete is allowed in the entire element.

This delivers an apt design criterion for this problem: just after the post-tensioning operation, no tensile stress is allowed at the upper surface of the section. To allow for this, the following post-tensioning strand trajectory was developed: 5 centrally placed strands of 31C15 cables, flanked on each side by 3 strands of 27C15 cables, all curving downwards at mid span, with a curvature in a vertical plane only. At both sides of the elements, two extra strands of 31C15 cables are installed. These, however, are curved in a vertical as well as in a horizontal plane, the horizontal curvature inducing a sort of "virtual" transverse post-tensioning as well (See fig. 6 and fig. 8). Strands of the types 31C15 and 27C15 are strands consisting of respectively 31 and 27 cables of the type T15, each consisting of 7 wires with a nominal diameter of 0.60'' (0.0152 m). The "C" implies the anchorage trumpets of the strand to be compact.

Fig. 8 View curved post-tensioning cables (left and right)

The illustrations in fig. 7 show the stresses in a typical bridge span with a total length of 67' 8.56'' (20.64 m). All values are given in megapascal, whereas 1 MPa equals 145.04 psi. The decks in this illustration are loaded with a combination of dead load, live load and post-tensioning forces, resulting in a compressive stress at the lower fiber of the bridge deck of 2.9 psi (0.02 MPa) and a compressive stress of 2030.5 psi (14 MPa) at the upper fiber. There is a remaining tensile stress of 40.6 psi (0.28 MPa) in a small area of the span, caused by the higher values of the live load stresses at the sides of the deck, because of its slightly lower thickness.

SPECIAL DECKS NEAR THE CROSSING ZONE

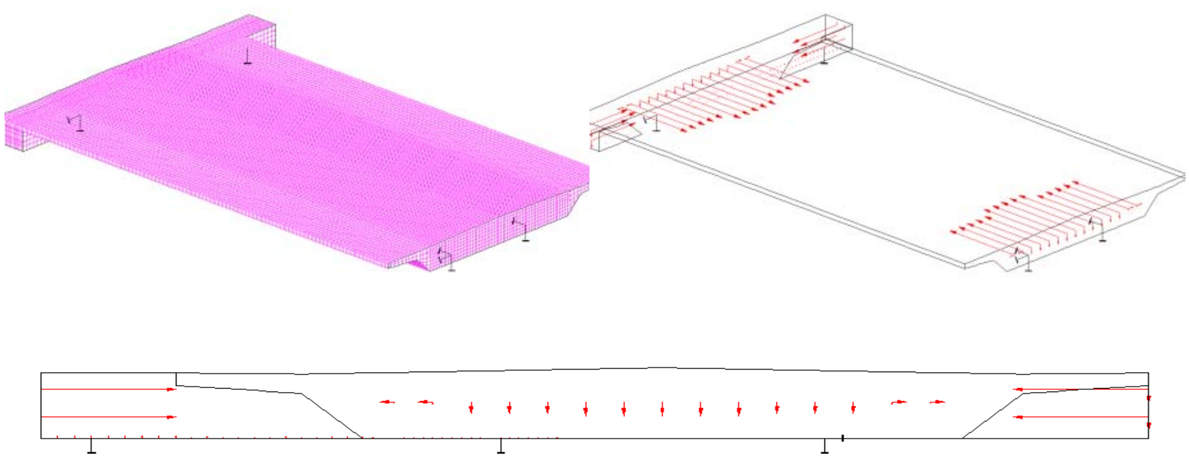

Fig. 9 Finite element model of the special decks near the crossing zone (upper left) with illustration of the vertical and horizontal forces at the end due to the post-tensioning (upper right) and more detailed in the cross section of the cantilever at the end (lower)

Due to the positioning of the piers near the crossing point and the required cantilevers, additional post-tensioning is required at these locations. The finite element model which is used to calculate the influence of the multi-directional post-tensioning in these special decks near the crossing zone is shown in fig 9. The pier positioning is comparable to the ones in fig. 5, i.e. pier types B and C.

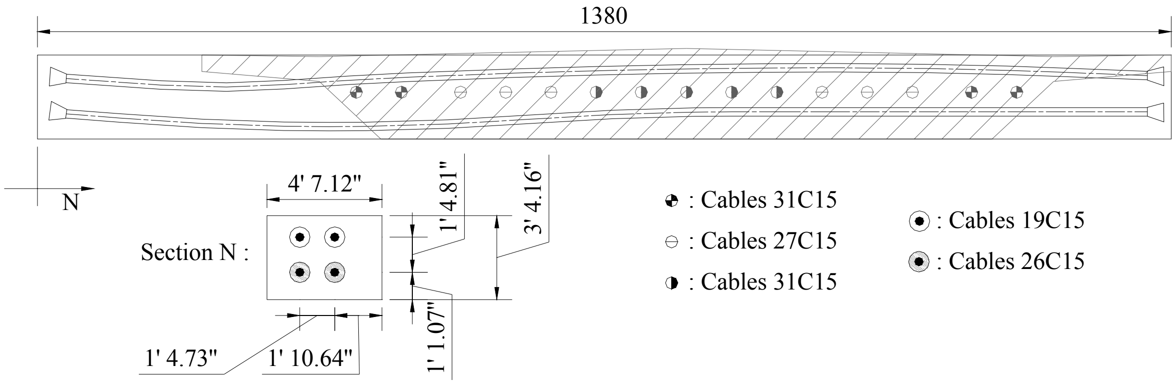

Fig. 10 Crossbeam for asymmetrical pier placement (pier type B)

The necessary crossbeams to bridge the larger column distances, are constructed by adding a beam to the end of the bridge deck element, which needs to be transversely post-tensioned. In contrast with the first alternative, this design needs to maintain its longitudinal post-tensioning in the spans near the crossing zone. Thus, the cable paths of the post-tensioning strands of the crossbeams need not only be determined by the above mentioned criterion stating no compression will be allowed, but also by the trajectory of the longitudinal post-tensioning cables, as is shown in fig. 10.

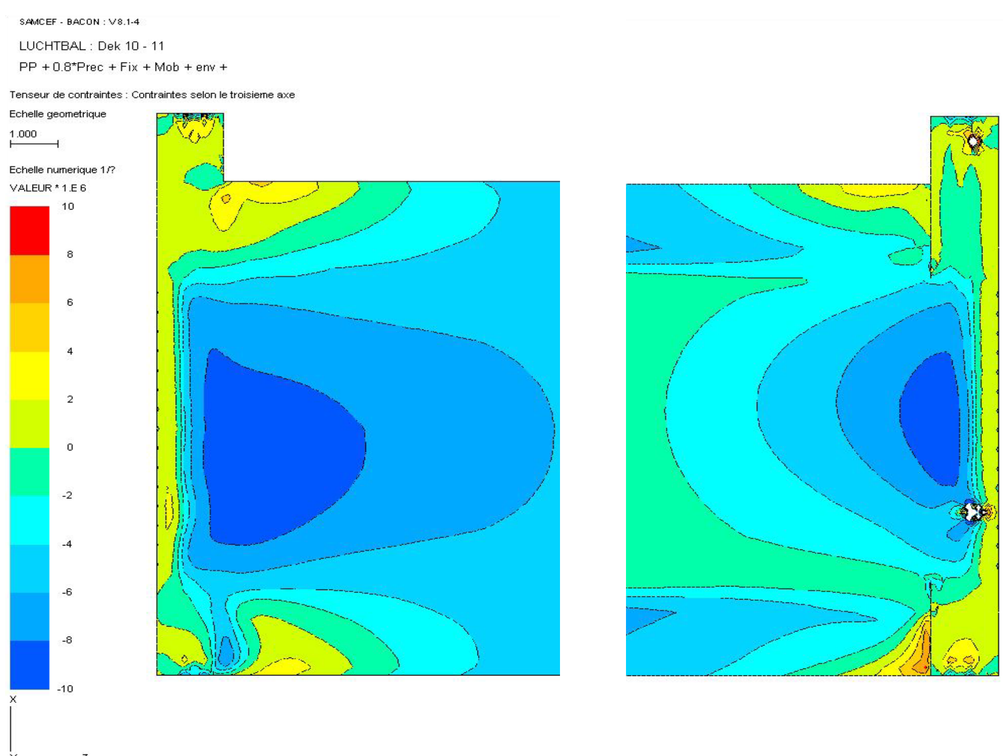

Fig. 11 Longitudinal stresses at the upper (left) and lower (right) surface
(Color scale varies from -10 to +10 MPa or -1450 to 1450 psi)

If the same types of post-tensioning strands are used as for the first alternative, the no-decompression criterion does not pose a problem at mid span for a load combination, consisting only of dead load and post-tensioning forces. Traction stresses only retain considerable values, up to 725.2 psi (5MPa), at the edges of the cantilevering part of the beam. This problem needs to be solved mainly by the installation of extra reinforcement bars in this area of the crossbeams.

The situation when the total load, i.e. dead load, live load (trains and on pathways) and longitudinal and transverse post-tensioning forces, is applied is considerably different. The effect of the longitudinal strands will not be influenced significantly by the fact that the deck elements and their support structure become asymmetric. The determining loading condition becomes the transverse tensioning force. When looking at the stresses caused by this factor alone, it becomes clear that this design will not satisfy the no-decompression demand, since it causes considerable local traction stresses of 870 to 1160 psi (6 to 8 MPa), as is shown in fig. 11. The solution for this problem lays in the augmentation of the tensioning force in the crossbeams in comparison with the beams of the precast alternative. When the crossbeams from both alternatives are tensioned using the same cable force, this force will be dispersed over a much larger area of the bridge deck in the design with the continuous bridge decks than with the one with the hollow precast elements. In order to be just as effective, the tensioning force needs to be higher, ensuring that no traction stresses remain at the upper surface of the decks after the conclusion of all tensioning operations.

DYNAMIC CONSIDERATIONS

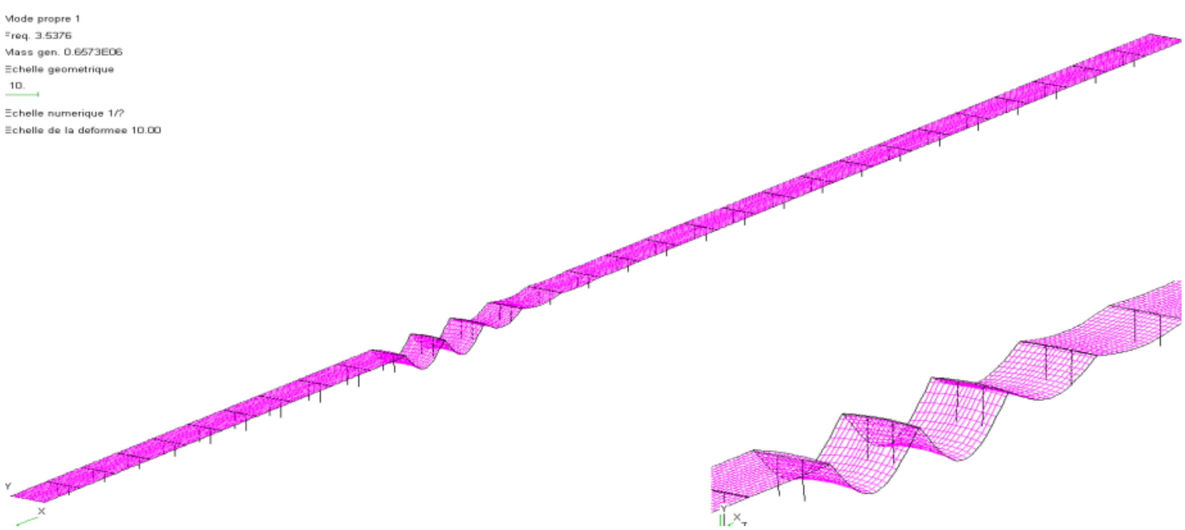

Fig. 12 Finite element model to verify the dynamic considerations

As in the first alternative, a verification of the dynamic behavior is necessary. A finite element model of the total of 30 spans (inclusive the piers) is used for this verification. Fig. 12 illustrates the model, deformed in the first mode. The first frequency is 3.53 Hz.

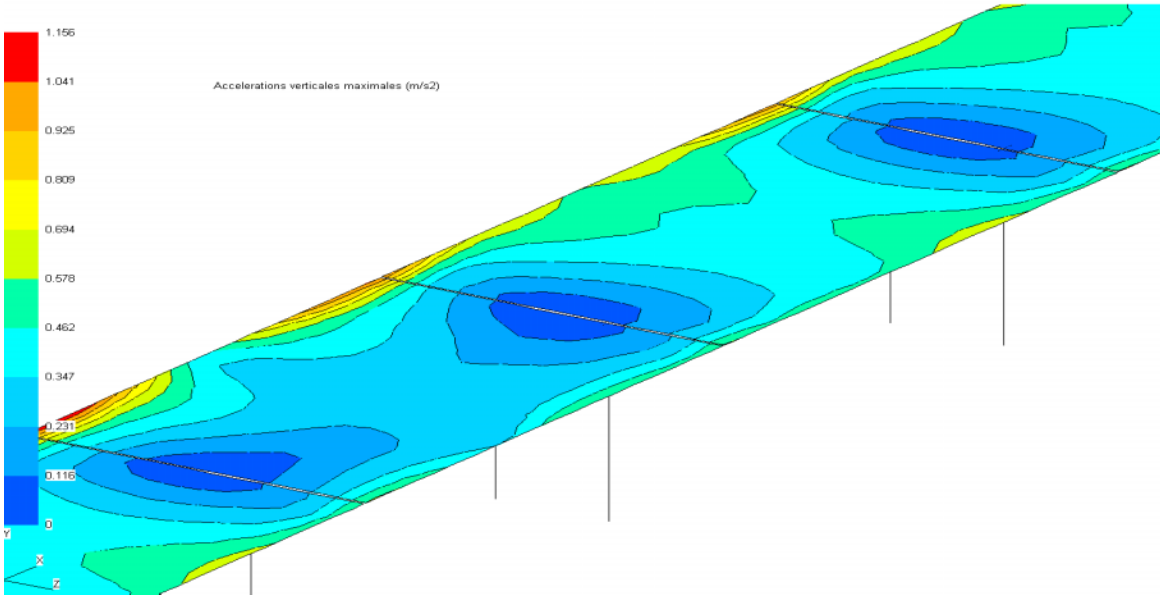

Fig. 13 Output of the vertical accelerations
(Color scale varies from 0 ft/s^2 to 3.8 ft/s^2)

Fig. 13 shows the vertical accelerations in the same conditions of a speed limit of 80 mi/h (130 km/h). At this speed an acceleration of 1.22 ft/s^2 (0.373 m/s^2) could be reached. The higher values, as shown in fig. 13, are situated on the walkways and parapets of the viaduct and can be neglected. Also in this alternative, the passenger comfort is assured.

CONCLUSIONS

This paper evaluates two alternatives for the construction of the Luchtbal viaduct, both using precast elements and/or HPC. Although the high strength concrete may not be required for the ultimate strength of the design, the criteria for angular rotations, vertical accelerations and comfort of train passengers will be influenced by the higher deformation modulus.

At first sight, the first alternative, with its different types of prestressing and post-tensioning and the use of precast hollow elements, may seem more complex than the second one, consisting of a post-tensioned bridge deck, cast on site. However, the specific conditions in the crossing zone necessitate an exceedingly complex multidirectional post-tensioning of the deck. Which one of the two different alternatives will be chosen to be constructed will most likely depend less on the complexity of the design or on the purely technical advantages of the high performance concrete and of the structural concept, but more on practical and economical arguments such as the cost and the working skill with high strength concrete and precast elements at this specific building site.

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