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Design and combined rail-structure response of a new high speed railway bridge

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Summary

This paper describes the combined rail-structure response of the northern section of the new high speed rail bypass at Mechelen, starting with the work done in the feasibility stage and continues on to the final design

Keywords: High speed railway, combined rail structure response, feasibility, final design

1. Introduction

The last piece missing in the high speed railway network in Belgium is the Bypass of Mechelen, the construction of two tracks allowing the high speed trains from Brussels to Amsterdam to pass Mechelen station at 160 km/h. These two tracks follow the existing track Brussels - Antwerp as far as possible (adjustments are made in radii, the existing lines (Lines No.25 and No.27) have been designed for a speed of maximum 90 km/h), and travel through urban areas of Mechelen.

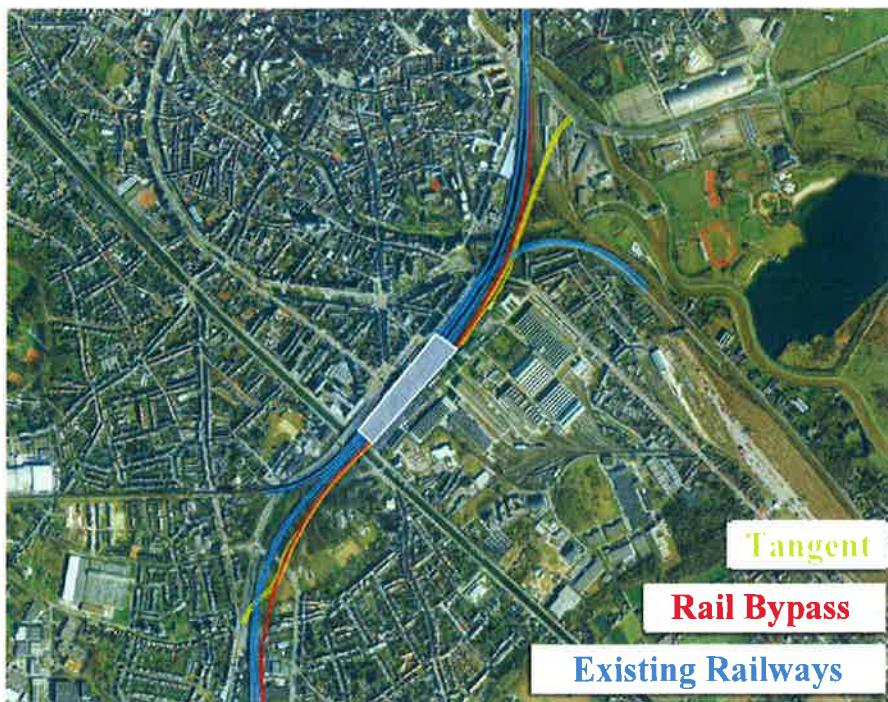


Fig. 1: Overview of the project

In order to limit expropriations and obstacles for the inhabitants of these urban areas of Mechelen, the design of a new ring road around Mechelen, called the 'Tangent' has been combined with the design of the Bypass railway infrastructure. This new ring road is a project of AWV, the Flemish Government Department of Roads and Traffic.

Since the existing railway lines are located on an embankment, as is the station, the new railway line is also situated on this level +1. The 'Tangent' road starts on level 0 for the connection to the existing road network, and dives to level -1 to pass a canal in a tunnel and connect to a new underground parking at the station. After the station, the road rises again to level 0. The zones before and after the tunnel require an integrated design for both the 'Tangent' road and the Bypass, who find themselves above each other. Several designs have been made for this combined infrastructure, one of these designs being a railway viaduct above the 'Tangent'-road. Since the infrastructure touches a lot of public space and will be very visible, the design was followed with great interest by the Flemish Government Architect, an administration brought to life as a control mechanism for the aesthetics of new infrastructure in Flanders. Especially the crossing of the Leuvense Steenweg was studied in detail as the new bridge will be built adjacent to an existing 91m long Vierendeelbridge. As a first option the idea of a viaduct was discussed with both the representatives of the Flemish Government Architect and the city of Mechelen and one of the demands was about the piers of the viaduct. They needed to be as slender as possible, without of course endangering the stability of the structure. Since the structure is a railway viaduct, the severe deformation requirement in longitudinal direction as described above requires a very stiff substructure. An optimal solution had to be sought.

After these preliminary results showed that the section of the viaduct around the Leuvense Steenweg, in order to keep the piers as slender as possible, would require using combined steel-concrete piers and spans of only 20m. Due to its length the viaduct and its piers would form a visual barrier and therefore have to great an impact on the view. The limited span would also hinder the underlying traffic flows.

Therefore an alternative design was created in which both road and railway structures were integrated in one design. In doing this several rigid abutments were integrated in the design to reduce horizontal displacement to acceptable values.

2. Theory

In general, railway track is a continuous structure with only as few expansion joints as necessary. These expansion joints are a weak point in the railway track structure, being a possible cause of derailment or other accidental incidents. Bridges on the contrary have, in general, expansion joints, the special case of integral bridges left aside. These expansion joints are inevitable for viaducts with a succession of decks. A railway viaduct has to combine both structures, and thus both perceptions about expansion joints. Longitudinal forces, traction and braking, are resisted both by the structure (via the bridge bearings to the substructure and foundations) and the rails (to the embankment behind the abutments). In this article, the influence of the use of continuous track on the design of the substructure of a viaduct is investigated.

The use of continuous track on bridges and viaducts in Belgium is subject to the following three guidelines and norms:

- Infrabel (Belgian Railway Infrastructure Manager): Circular nr.32 I (July 1992): Use restriction for continuous track [1]
- NBN EN 1991-2 (January 2004): Eurocode 1: Actions on structures – Part 2: Traffic loads on bridges [2]
- UIC Code 774-3 (December 2000): Track/bridge Interaction – Recommendations for calculations [3]

The guidelines and rules regarding the use of continuous track on bridges specified in the documents above will be treated in the paragraphs below.

2.1 Infrabel Circular nr. 32 I

The Infrabel circular gives restrictions for the use of continuous track on bridges. A distinction is made between concrete bridges with ballasted track and steel bridges with ballasted track:

- Continuous track may be used on concrete bridges with ballasted track and expandable lengths up to 70m. The allowed expandable length is reduced to 50m if the bridge touches curves with a radius smaller than 1200m. Concrete bridges with an expandable length of more than 70m, require expansion units at the mobile supports. The use of continuous track is allowed on viaducts with a succession of isostatic decks if the decks satisfy the requirements for a single bridge.
- Continuous track may be used on steel bridges with ballasted track and expandable lengths up to 50m. The allowed expandable length is reduced to 30m if the bridge touches curves with a radius smaller than 1200m. Steel bridges with an expandable length of more than 50m, require expansion units at the mobile supports.

2.2 EN 1991-2

The design of a railway bridge in Belgium, and by extension in all of Europe, is imposed to be calculated according to the Eurocodes. In EN 1991-2, a specific part is devoted to the bridge – rail interaction, paragraph 6.5.4 ‘Combined response of structure and track to variable actions’. This paragraph describes the actions to be considered, the modelling and calculation of the combined track/structure system and the requirements to be satisfied in the bridge calculation, valid for conventional ballasted track.

The design criteria are split in criteria regarding the stresses in the track and criteria regarding the deformation of the structure. The article focuses on the influence of these deformation requirements on the substructure of a viaduct, and therefore only the relevant criteria will be mentioned. There is only one criterion that affects the substructure, given in paragraph 6.5.4.5.2 (1). The relative longitudinal displacement between the end of a deck and the adjacent abutment or the relative longitudinal displacement between two consecutive decks, δ_b , shall not exceed 5 mm due to traction and braking, on a bridge or viaduct with continuous track.

2.3 UIC Code 774-3

Some of the design criteria and the calculation methods given in EN 1991-2 are only applicable in standardized bridges and viaducts, with ballasted track and specific track system. The UIC Code 774-3 gives a more extended version of design criteria and calculation methods. For example, a bridge with slab track may be calculated according to the specifications given in UIC Code 774-3. Since the criterion about δ_b as described in paragraph 2.2 according to EN 1991-2 does not change, this will be used for all types of substructures treated in this article.

In conclusion, the use of continuous track influences the substructure in two ways:

- the longitudinal forces due to traction and braking are partially transferred past the expansion joints by the continuous track
- a severe deformation requirement is given, δ_b .

3. Feasibility study

The preliminary structural design followed the aesthetical design, and was therefore bound to some conditions. Firstly, the shape of the superstructure was known, and limited the possibilities in spans for the viaduct. The superstructure consisted of a girder plate, either reinforced concrete or with both internal and external precompression cables. Because of the limited effective height, spans were limited to approximately 20m or 30m. A drawing of the section of the superstructure is given in Figure 2.

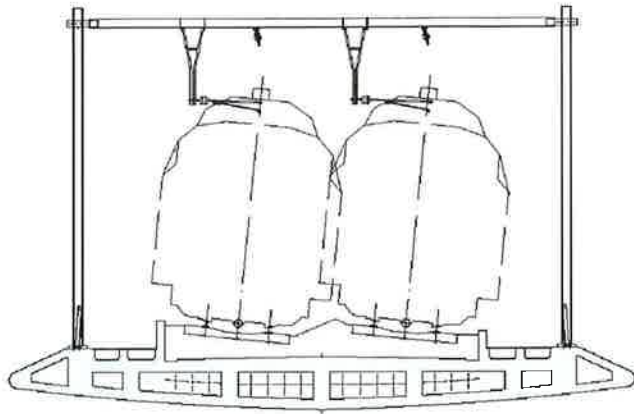


Fig. 2: Section superstructure viaduct

Secondly, the shape of the piers had to be round, and the implantation depended on the road design. Different zones were made, with the viaduct above the central reservation (two central piers), above one direction (one pier at both ends) or above the road verge (two central piers). As demonstrated in figure 3, this variable concept complicated the study with special cases like placing the columns oblique



Fig. 3: Visualisation of the piers underneath the viaduct

The following part of the article treats the calculation of a part of the viaduct, comparing different approaches in the pier – structure connection to get a pier as slender as possible, firstly from a strength point of view, secondly from a longitudinal deformation point of view.

The structure modelled consists of a hyperstatic deck of four spans of 20m, supported by 10 round piers (2 piers per support) and with a radius of 2000m. The total length of the deck is 80m, which means that the fixed bearings need to be on one of the three intermediate supports, in order to follow the rules set by Infrabel.

The height of the piers is 10m, and they are restrained at the foundations. A view of the model, made in SCIA Engineer, is given in figure 4.

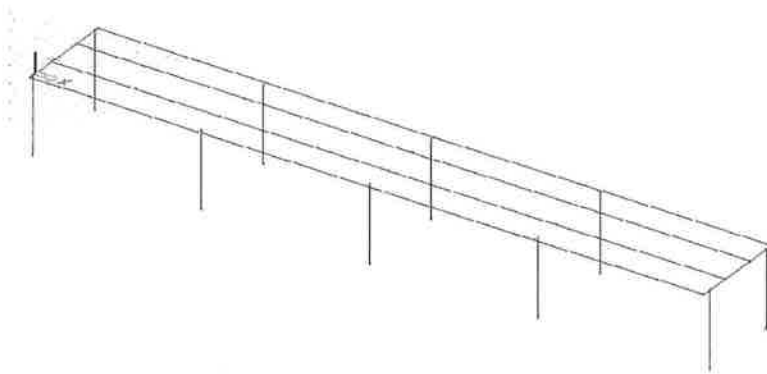


Fig. 4: Calculation model of the viaduct

Four options are retained regarding the connection between pier and deck (conception of the bearings):

- The central support has two longitudinally fixed bearings, the row of piers at the outside curve is transversally fixed (1)
- The two central piers have fixed bearings, all other piers have transversally fixed bearings (2)
- The six central piers have fixed bearings, the four outside piers have transversally fixed bearings (3)
- The six central piers have a monolith connection with the deck, the four outside piers have transversally fixed bearings (4).

Firstly, a calculation is done to define, for each option, the minimal diameter necessary for the stability of the structure. The actions considered in this calculation are self-weight, ballast, train load, braking and traction force, centrifugal and nosing force and temperature. The figure below gives an overview of the different options, with the maximal bending moment in the piers, the corresponding maximal bending moment and the minimal diameter of the pier, made of reinforced concrete and able to resist the calculated forces.

	N _s d (kN)	M _s d (kNm)	D (mm)
1	9150	20323	2000
2	9150	20317	2000
3	10444	7534	1300
4	10364	6766	1200

Table 1: Table of forces and corresponding diameters

The comparison of the minimal diameter for the different cases shows that spreading the longitudinal forces over more supports is an effective way to be able to decrease the dimensions of the substructure. This is an expected result, and of course this concept creates additional forces in the superstructure that should be taken into account.

The given diameters satisfy the strength criteria. A second calculation needs to be done in order to check if the deformation criteria, imposed by the use of continuous track, are satisfied. Now the same table is made with the longitudinal deformation caused by traction or braking. The table is given in table 2.

	D (mm)	v (mm)
1	2000	15.1
2	2000	15.1
3	1300	27.8
4	1200	11.7

Table 2: Table of diameters and corresponding deformations

This table shows that the minimal diameters according to strength are not at all sufficient to limit the deformation according to the design criteria for the use of continuous track, and the difference is significant. Other measures need to be done in order to create a structure that satisfies all the

demands. In order to get slender piers, as demanded by both the representatives of the Flemish Government Architect and of the city of Mechelen, plain reinforced concrete was no longer an option and the use of mixed steel-concrete structures would be necessary. This case study shows that when designing a substructure of a railway viaduct, even in preliminary design, the design criterion δ_b is of great importance and should be taken into account. In this case however, the discovery, combined with several other elements caused a complete change in the project's design.

4. Final design

The final design does no longer have one single type of structure, but is adapted to each location specific needs.

The first section, between the station and the Leuvense Steenweg, is an area where the Tangent road is still partially underground. On the side of the existing railway embankment a large earth retaining wall is required. By using this wall and adding some columns the railway viaduct can be designed using mainly structures already needed for the road tunnel to carry the train load.

Upon reaching the Leuvense Steenweg the Tangent road has shifted to one side and is no longer situated under the railway. The intersection itself however is much wider and is once again partially under the railway. The small area where the road is adjacent to the railway allows for a decent abutment to be built. This abutment will serve as a fixed point that can handle the braking and acceleration force exerted on the bridge across the intersection. This bridge over the Leuvense Steenweg is a 3-span posttensioned trough bridge will be built. The form was a consequence of the environmental impact assessment which indicated the need for noise barriers on the viaduct of at least 2.4m above the rail. As these noise barriers would already form a visual barrier. This height was used to add rigid beams and create a trough section. In doing this the deck was rigidified, allowing the larger spans needed to cross the intersection. The deck is posttensioned used 16 cables, 8 of them are within the sidebeams of the deck allowing a larger eccentricity.

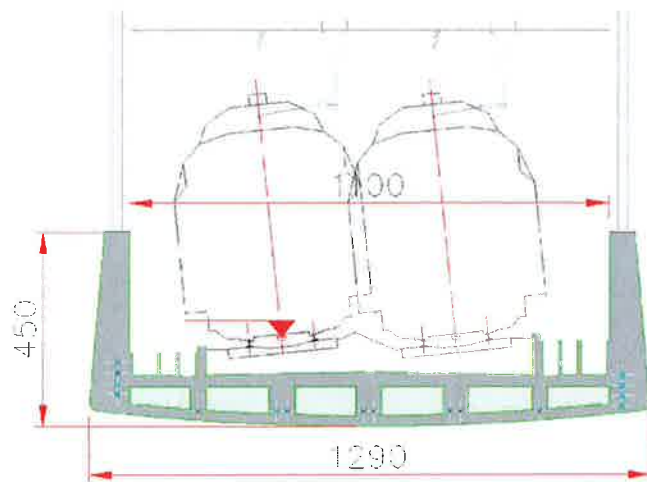


Fig. 5: Adapted cross-section of the viaduct

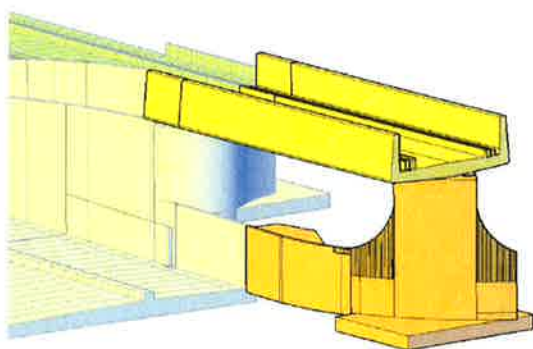


Fig. 6: Model of the first span

The substructure consists of two abutments at both ends and 2 piers placed on two corners of the future intersection. The fixed bearings are placed on the southern abutments as this was the only structure capable of handling the required horizontal force. The two piers are placed at an angle relatively to the deck, in order to follow the intersections border. The piers are 2m thick. This was required to enable the placement of the bearings as they will be around 1.2m in diameter. But is also made the pier able to resist the horizontal friction that can occur in these moveable bearings.



Fig. 7: Visualisations of the new design: Crossing of the Leuvense Steenweg

Once passed the intersection the tangent moves further away and no longer hinders the railway viaduct. However in this area the railway still has to cross an existing railway line (L53) and the river Dijle. Due to the limited clearance between both railway lines a mixed steel-concrete bridge was designed. The river Dijle was crossed using a steel clamped arch. Past the river the railway continues on a new embankment. In the part of the project, it was possible to place regular abutments.

This design once again modelled in SCIA Engineer as one combined structure and subjected to the breaking and acceleration forces. As the superstructures are very diverse over this design, the stability of each was checked in an individual model. An overall 1D-model was created only to verify the horizontal deformation.

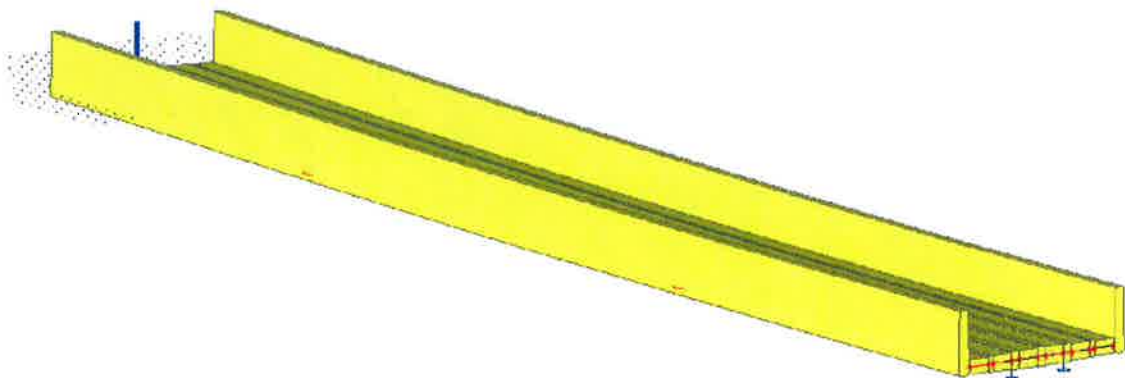


Fig. 8: The detailed 3D-stability model



Fig. 9: The "horizontal deformation"-model

The results clearly showed that the use of 3 abutments in this area as well as the earth retaining walls were sufficient to reduce the deformations to an acceptable value of 1,2 mm. This maximum occurs at the movable bearing of the deck across the Leuvense Steenweg

5. Conclusion

This study shows that the deformation limit imposed by EN 1991-2 can be a determining factor in the design of a railway-viaduct, especially in cases where the design attempts to be as slender as possible. In those cases the horizontal deformation can be a far more severe criterion than stability and should therefore be taken into account when determining the cost estimate of a project.

In the case of the Leuvense Steenweg this has been one of the reasons to completely change the design during the feasibility study. The new design, with the classical abutments, showed the great beneficial impact of some rigid elements in the substructure, resulting in this case in automatically being compliant to the limit of deformation.

6. References

- [1] THIELEMANS J., DE PAUW B., "Influence of Combined Track-Structure Response on the Substructure of a Railway Viaduct", 2012, The First International Conference on Railway Technology: Research, Development and Maintenance