High speed railway (Reino Unido) Duddeston Junction Viaduct. Desafíos del diseño.

High Speed Railway (UK). Duddeston Junction Viaduct. Design challenges.

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RESUMEN

El Viaducto de Duddeston Junction, en la línea de alta velocidad ferroviaria entre Londres con Birmingham, contiene diferentes elementos estructurales singulares, como un punto fijo ubicado en un estribo de hormigón armado de grandes dimensiones, requiriendo un análisis detallado mediante diferentes modelos de bielas y tirantes y un modelo sólido de elementos finitos. La llave de cortante, en acero corten, necesaria en el tablero para transmitir las cargas al estribo, ha requerido duplicar sus elementos para poder cumplir con los límites de fatiga. El artículo también describe el apoyo del tablero sobre un pórtico metálico cuya viga transversal superior es integral con el tablero debido a limitaciones de gálibo.

ABSTRACT

Duddeston Junction Viaduct carries the high-speed train line between London and Birmingham, and includes a few structural singular elements, such as a reinforced concrete abutment used as the fix point of the viaduct, which has to sustain large longitudinal forces, and that has required a detailed analysis with different strut and ties models and a solid FEM. The shear key at deck level, in weathering steel, transmitting loads to the abutment has required duplicating the number of steel beams to fulfil fatigue requirements. The article also describes a steel portal, supporting the viaduct over rail tracks below, which cross beam is integral with the deck due to clearances constrains.

PALABRAS CLAVE: mixto mixto, punto fijo longitudinal, llave de cortante, pórtico metálico, bielas.
KEYWORDS: steel viaduct, longitudinal fix point, steel shear key, steel portal, concrete thrust block.

1. Introduction

Duddeston Junction Viaduct is one of the structures designed as part of the High Speed 2 rail line connecting London and Birmingham. It carries two tracks over existing infrastructure, including a Network Rail corridor and the River Rea, as the alignment approaches the Curzon St. Station terminus on the Birmingham West Spur. The line design speed in the area is 100km/h and the design life of all structural elements, including parapets, is 120 years, in accordance with the UK National Annex to EN 1990:2002 – Basis of Structural Design [1].

The viaduct has a total length of 706m divided in fourteen continuous spans, with the following arrangement (in m): 46 + 52 + 32 + 46 + 50x2 + 55x7 + 45. Grid lines numbering goes from GL23 (where the expansion joint with the adjacent Viaduct – Curzon 1 – is located) to GL37 (abutment end).

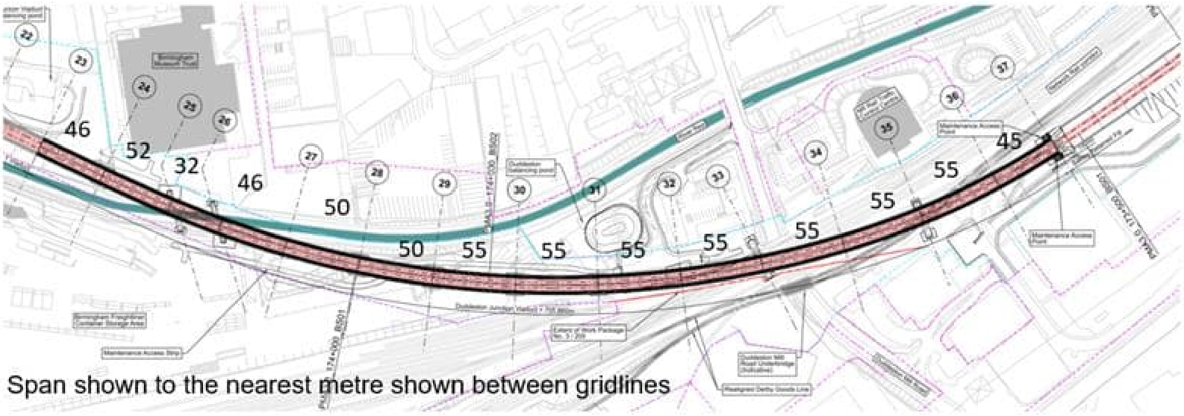

Figure 1 Plan view of structure, with span lengths indicated.

From GL24 to 37 the structure is curved in plan, with a series of different radii. Between GL23 and GL24 it transitions from a straight alignment to a 4300m radius curve; between GL24 and 26 the radius varies from 1630m to 695m (radius changes at a setting out point – SOP- in the vicinity of GL26); from GL26 (latter SOP) to 34 there is a constant radius of 659m. From GL34 (SOP located between GL 34&35, but close to 34) to GL36 (SOP close to 36, but between 36&37) the radius transitions from 583.5m to 508m, and from the latest SOP to GL37 is constant and equal to 670m.

The steel deck is composed of two fabricated weathering steel plate girders of constant 4m depth and at 6.5m lateral centres, carrying cross girders at approximately 3.0m spacing. Both the transverse and the longitudinal girders act compositely with the reinforced concrete deck slab by using shear connectors.

A plan bracing consisting of Z profiles in a K arrangement connects the bottom flange of the main longitudinal girders. Vertical cross bracings made of I section are also provided to prevent the distortion and the warping of the cross-section.

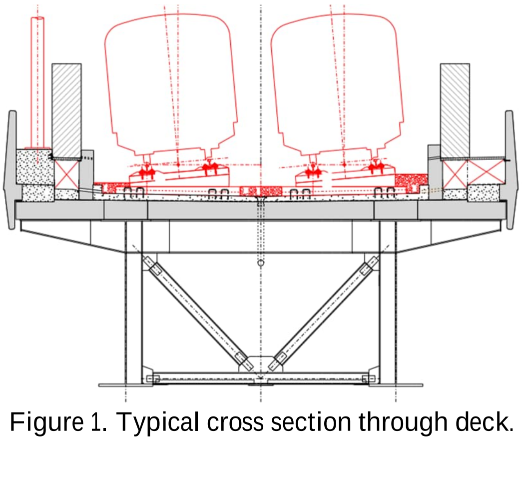

Figure 1. Typical cross section through deck.

The viaduct is supported on a mixture of standard reinforced concrete piers and steel piers or portal frames that span transversely over River Rea (GLs 25 and 26) and Network Rail tracks (GLs 32 to 35). All steelwork is in weathering steel. The sizing of these supports varies; portals clear spans are 22.46m, 22.46m, 23.20m and 18.70m for GL 25, 26, 33 and 35 respectively.

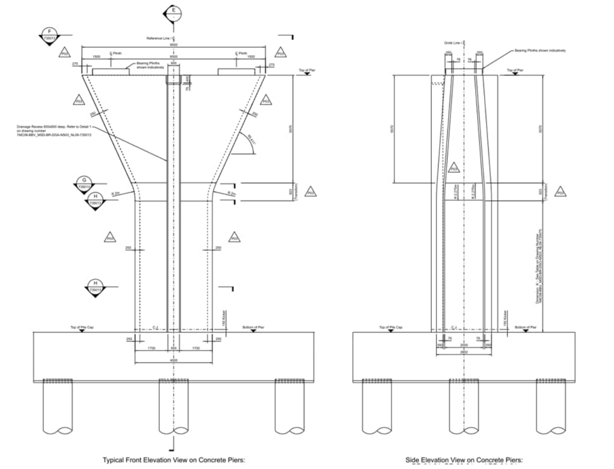

Figure 3. Concrete piers geometry.

Due to vertical clearance requirements over Network Rail tracks, the horizontal cross beam of the steel portal at GL35 is integral with the deck. The complex structural behaviour of this integral portal beam has been analysed with a Finite Element Model (FEM) and is further described in section 2 of this article.

All viaduct piers and abutment will be supported off reinforced concrete bored piles and pile caps. Piles' diameters used are 750mm, 1200mm and 1500mm. Since bentonite will be used for pile construction, the piles' shaft friction capacity has been reduced by a 20%.

A reinforced concrete abutment at GL37 allows for a transition between the viaduct and the adjacent retained fill. The geometry of the abutment body is dictated by the fact that the longitudinal fixity of the viaduct is provided at this location. Special consideration has been given to its structural analysis due to the associated large horizontal forces that has to withstand, which is further developed in section 4 below. To lock the steel deck to the fix abutment a steel shear key has been introduced at the bottom flange level of the main girders; this design is described in detail in section 3.

At the opposite end of the fix abutment at GL37, an expansion joint has been provided at GL23, separating Duddeston Junction Viaduct from the adjacent one, Curzon 1 Viaduct. The movement range at the free end of Duddeston Viaduct is -740/+685mm for Serviceability Limit State and -1100/+998mm for ULS , where negative values represent deck contraction and positive ones deck expansion.

The articulation arrangement of the viaduct consists of spherical bearings which are longitudinally free in all the supports (two each) but with one of the bearings at each gridline being transversely fixed. The longitudinal guide is oriented tangentially to the curve in plan.

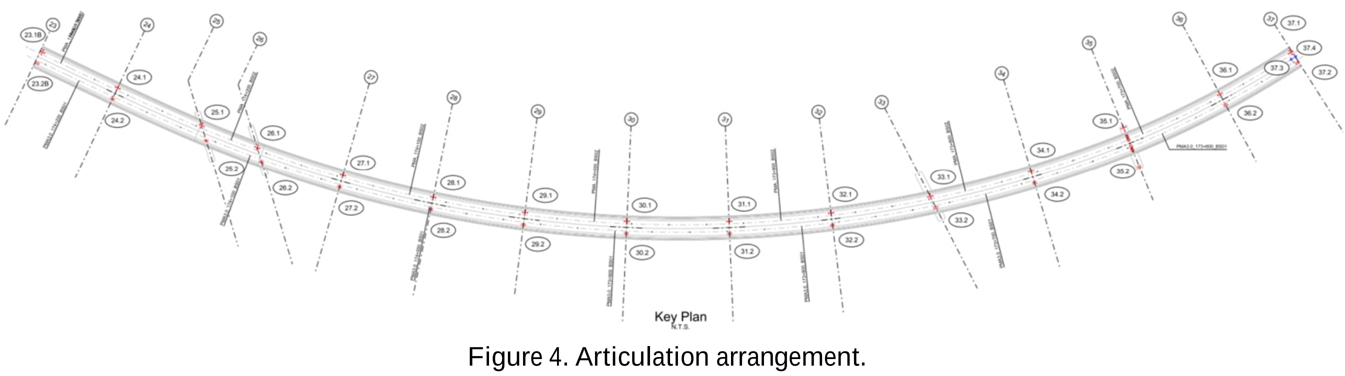

Figure 4. Articulation arrangement.

2. Integral portal beam at GL35

At GL35 a portal frame spanning transversally over Network Rail tracks is required. The portal was solved with two pier shafts at each side of the tracks with a clear span of 18.70m (22.20m between bearings) and a cross-beam with a rectangular box cross section of 4.00m wide by 3.30m deep. Due to vertical clearance requirements, the vertical alignment of the supported tracks and the 4-meters-deep longitudinal steel girders, the portal cross-beam had to be designed integral with the deck.

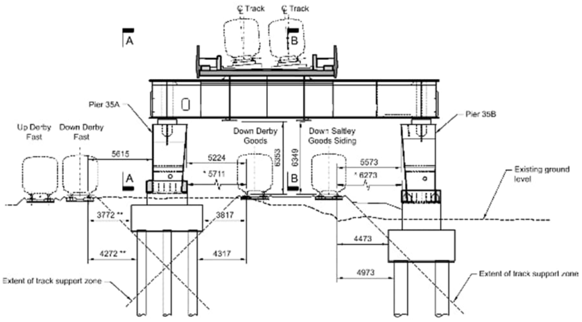

Figure 5. Steel portal at GL35 geometry.

The initial concept design proposed the bottom flange of the portal beam to be aligned with the bottom flange of the main girder to avoid a clash between the deck parapet downstand and the portal cross-beam.

The reinforced concrete slab was designed as precast units between main girders and cross girders, due to the need of a quick installation whilst working over the existing railway. Therefore, the cross girder at the gridline is required for the installation of the precast units.

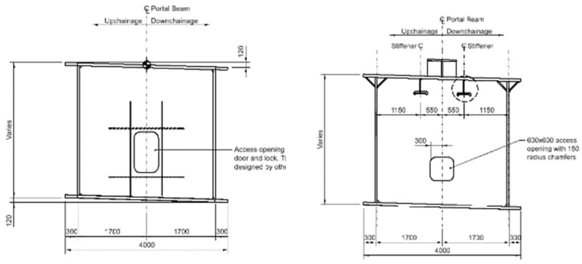

Figure 6. Transverse integral portal beam. Cross Sections A-A and B-B.

2.1 Challenges

The design was clearly governed by fatigue limit state and at the first attempt of designing the portal beam with the bottom flange aligned with the main girder bottom flange it was seen that the combination of longitudinal stresses from the main girder and transverse stresses from the portal beam was resulting in fatigue stresses well above the limits. Therefore, it was obvious that to achieve an optimal design the flanges of the main girders needed to be independent of the flanges of the portal cross-beam, as shown in Figure 7 below.

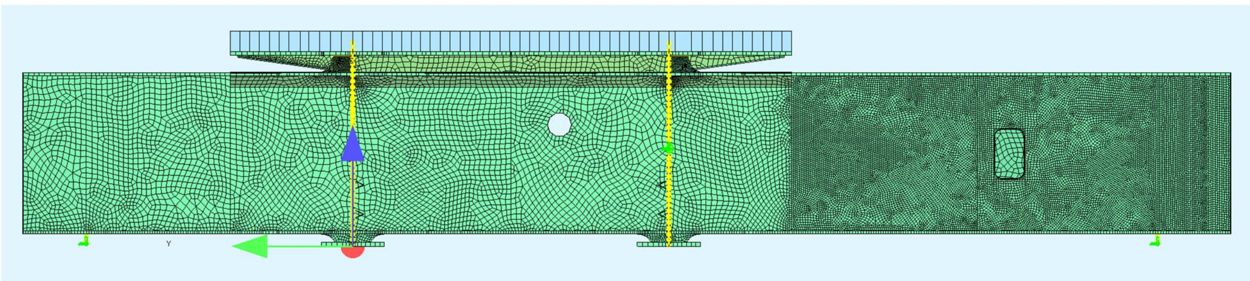

Figure 7. Transverse integral portal beam. Section.

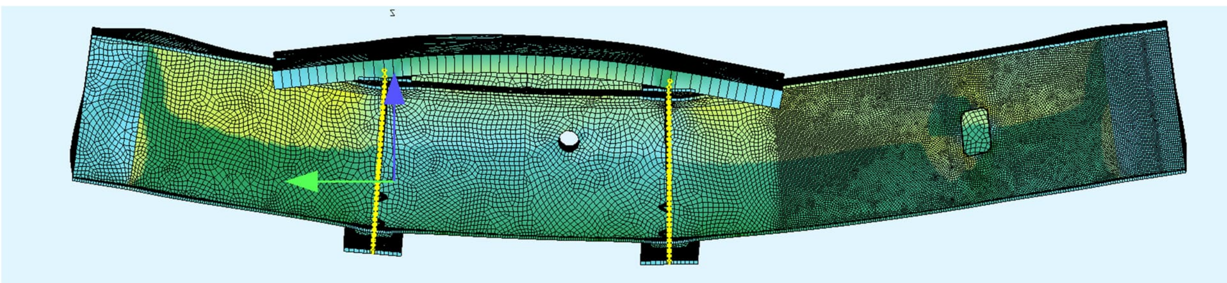

Figure 8. Transverse integral portal beam deflection.

Additionally, as the deck is not centred within the portal beam with one main girder almost at midspan whilst the second main girder is approximately at one-quarter of the span, the sagging deflection of the integral portal beam causes significant distortion of the main girders which adds further fatigue stresses to the system (see Figure 8 above). At the same time, the concrete slab connects the top flanges of the main girders, fixing their relative displacement, and leading to high local stresses at the main girder webs.

Besides the structural challenges defined above, the following challenges due to non-structural constraints have also been faced:

- The portal beam, as a closed space, shall be accessible for inspection and maintenance. So, an opening for accessing the box needed to be introduced away from the railway tracks with a position then right where the shear stresses are higher (see Figure 8).
- There is a drainage carrier pipe running longitudinally in between the main girders, below the concrete slab, requiring a circular opening at the beam webs.
- The portal beam shall drain the water outside the Network Rail tracks. Consequently, the frame was provided with a double pitch crossfall towards the piers.

2.2 Parametric design

To resolve the above mentioned structural challenges, several design iterations with a detailed shell model of the portal cross-beam was going to be required. For that reason, it was decided to use a script in grasshopper that populated the model. This script was capable of automatically generating the model based on various parameters and geometrical inputs. By utilizing this parametric design approach, the designer was able to efficiently explore and evaluate multiple solutions. This method not only allowed having a detailed analysis of several models but also facilitated easy comparison of different design alternatives, thereby enhancing the overall efficiency and effectiveness of the design process.

The script allowed to change the geometry of the elements such as the cross girders, include and remove elements such as webs or stiffeners, increase/decrease the thickness of every plate, etc.

3. Steel shear key design

The purpose of this section is to explain the process followed to design the steel shear key at the level of the main girders bottom flange which will transfer the longitudinal forces of the viaduct into the fix abutment (longitudinal fixed point of the viaduct). Fatigue stresses have governed the design with a maximum fatigue force of 12.2MN, which needs to be considered on both longitudinal directions. Due to the stress range created by these forces (reversable force in opposite directions), the stress range was doubled of the stress generated by the above longitudinal force, which posed a significant challenge for designing the shear key. Several options were attempted until a solution was found adequate.

In the following paragraphs, three of these considered options are explained and summarized. Apart from these options, several other modifications and variations were considered, what was possible thanks to the methodology used. The geometry of each option was defined with Grasshopper software using an algorithm to parametrize the geometry. The structural model was studied using the software Sofistik. The Sofistik FEM mesh geometry, materials, loads, and boundary conditions were created directly from the algorithm. This enabled quick changes to test each option (ie adding stiffeners, adding structural elements, modifying their locations, changing sizes of structural elements, reduce or increase the mesh sizes, etc).

3.1 Shear key box defined under each main girder (Option 1).

This option consisted on defining a steel shear key box under each main girder. The steel shear key box comprised 7 webs and two flanges and it protruded upwards and downward of the bottom flange of the main girder. The main challenge with this solution was the high concentration of stresses at the main girder bottom flange and the intention was to achieve a wider spread of the load by engaging the main girder web in this load transfer. On each flange of the steel shear key box a neoprene bearing would be installed to transfer the longitudinal forces to the concrete shear key block defined in the abutment, as shown on the sketch below in Figure 9.

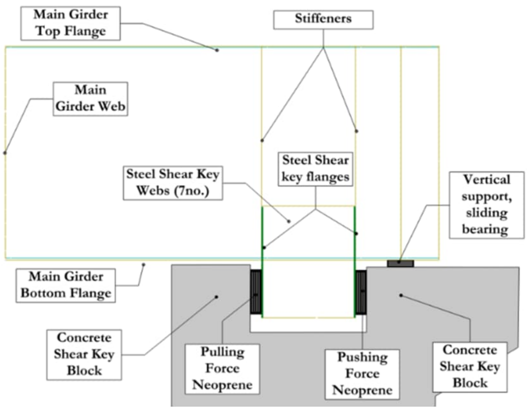

Figure 9. Sketch showing the concept solution for Option 1.

The algorithm in Grasshopper was defined to include the geometry of the shear key box so that parameters such as its relative position to the main girder, size, number of webs or plates sizes could be easily changed (See Figure 10).

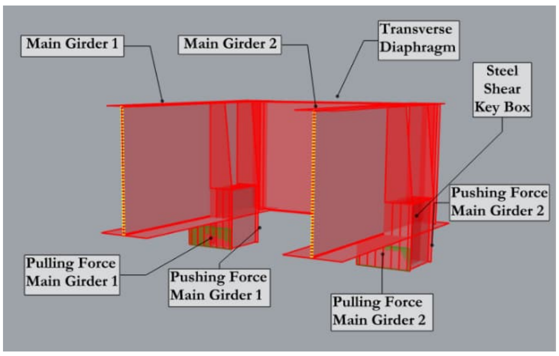

Figure 10. Model Geometry defined using Grasshopper algorithm shown on Rhino (Option 1).

On this model the reactions were applied as patch loads to the faces of the steel shear key box (as indicated on Figure 10).

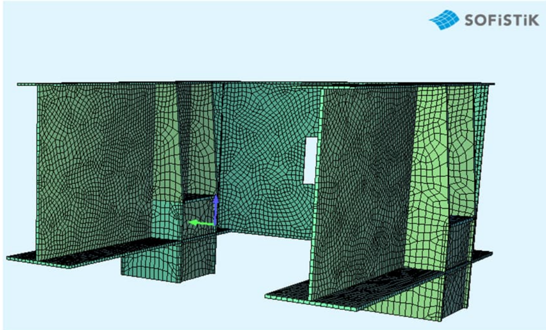

Figure 11. Sofistik FEM defined using the algorithm in Grasshopper (Option 1).

To obtain the reactions for this option, a change in the global model of the Viaduct was required, as it was originally thought with a single longitudinal fix point. The global model was modified to include a longitudinal support under each girder. However, with this solution the rotation in plan was now restricted and as a result the reaction under each girder was 8MN instead of half of the 12.2MN.

3.1.1. Conclusion and reasons for exclusion of this option.

The reason for not considering this option was the failure of the main girder bottom flange in fatigue. At the contact point between the shear key flange and the main girder bottom flange the stress range was above the limits defined on BS EN 1993-1-9 table 8.4 construction detail 8 ($\Delta\sigma_c$ defined on this table as 71 MPa).

$$\gamma_{Ff}\ \gamma_{Mf}\ \Delta\sigma_{E,2} < 71 \text{ MPa} \quad \gamma_{Ff} = 1.0 \quad \gamma_{Mf} = 1.1 \tag{1}$$

$$\Delta\sigma_{E,2} = \lambda\,(|\sigma_{max}| + |\sigma_{min}|)\text{MPa} \tag{2}$$

$$\Delta\sigma_{E,2} = 0.65(\,|107| + |-110|)\text{MPa} \tag{3}$$

$$\Delta\sigma_{E,2} = 141 \text{ MPa} \tag{4}$$

$$155\ MPa >> 71 \text{ MPa} \quad \textbf{(Fail)} \tag{5}$$

The modified stress range for this element $\Delta\sigma E,2$ considered a damage equivalent factor λ of 0.65, calculated as $\lambda 1^* \lambda 2^* \lambda 3^* \lambda 4$ as per BS

EN 1993-2 and HS2 Standards, considering an influence length of 45m ($\lambda1=0.64$), 119 million gross tonnes per annum ($\lambda2=1.37$), 120 years working life ($\lambda3=1.04$) , and a ratio for the case with one track loaded and two track loaded of 0.5 for this elements stress range ($\lambda4=0.71$).

Figures 12 and 13 show the values of σmax and σmin for the longitudinal stresses in the main girder bottom flange at the location of the attached shear key flange.

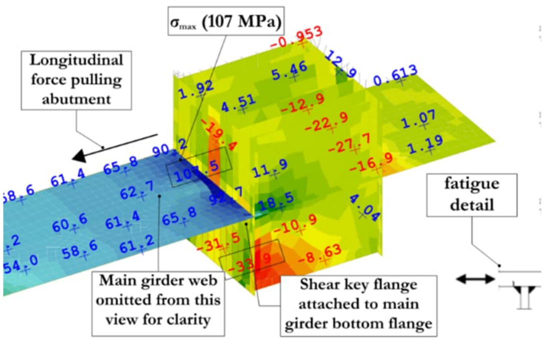

Figure 12. σ_{max} stress peak (107 MPa) at the main girder bottom flange. Longitudinal force pulling abutment (Option 1).

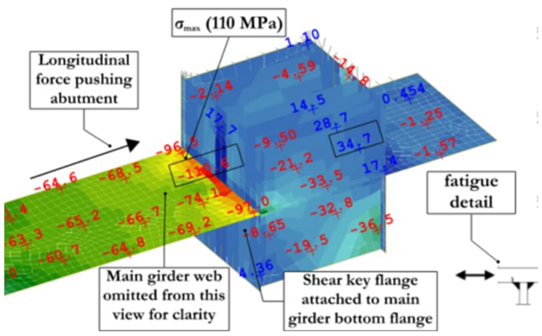

Figure 13. σ_{min} stress peak (-110 MPa) at the main girder bottom flange. Longitudinal force pushing abutment (Option 1).

3.2 Shear key beam defined under transverse diaphragm (Option 2).

This option consisted of a rigid steel beam defined under the transverse diaphragm that connected the bottom flange of both main girders at the vertical support location. The structure in the global model was defined with a single longitudinal fixed point located in the middle of both main girders. The maximum reaction for fatigue for each direction applied at each face of the shear key beam (as a patch load to represent the neoprene area) was 12.2MN.

The aim for this option was to avoid the concentration of stresses on the main girder bottom flange by achieving a wider spread through the webs of the shear key beam (see figure 14).

The algorithm was modified to include this shear key beam in the geometry. The resulting FEM model is shown in Figure 14. The area marked in red shows the location where the patch load was applied for the longitudinal pulling force; for the pushing scenario the load was applied on the opposite side of the shear key beam.

For option 2, three additional flanges were added over the main girders bottom flange to distribute the stresses and reduce the stress range on the critical elements.

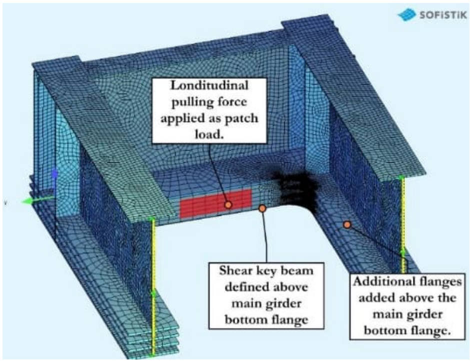

Figure 14. FEM of shear key beam defined under transverse diaphragm and above main girder bottom flange.

2.2.1. Conclusion and reasons for exclusion of this option.

This option was discarded due to the failure of several regions adjacent to welds on the shear key beam and the region connecting the shear key webs and the main girder flanges (see figure 15 and 16). Welds were located outside this region, however, at the contact point between the shear key webs and the main girder flanges the stress range was still above the limits defined on BS EN 1993-1-9 table 8.1 construction detail 5 ($\Delta\sigma_c$ defined on this table as 125 MPa).

For this element:

$$\Delta\sigma_{E,2} = 122 \text{ MPa} \qquad (7)$$

$$135\ MPa > 125 \text{ MPa} \quad \textbf{(Fail)} \qquad (8)$$

Figures below show the maximum and minimum stresses at the location of failure. The concrete block and elastomeric bearings are indicative only (were not included in the model but shown here for clarity).

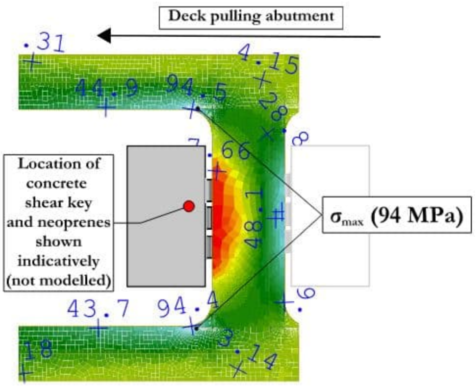

Figure 15. σ_{max} stress peak (94 MPa). Longitudinal force pulling abutment (Option 2).

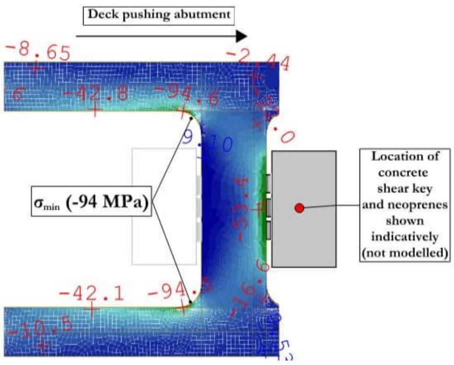

Figure 16. σ_{min} stress peak (-94 MPa). Longitudinal force pushing abutment (Option 2).

2.3 Two shear key beams (Option 3).

This option derives from option 2, so as to create different loads paths for the pushing and pulling scenarios. While this option only requires one concrete block, it requires more steel.

In this Option, neither of each critical region is affected equally by both loading situations (pushing and pulling) which reduces considerably the stress range. Thanks to this, an optimization was possible adding only two additional flanges (rather than three) and reducing the width of the shear key beam compared to the Option 2.

The stresses on this option for each critical location comply with fatigue limits.

$$\Delta\sigma_{E,2} = 0.65(\ |126| + |-7|)\text{MPa} \qquad \textbf{(9)}$$

$$\Delta\sigma_{E,2} = 86 \text{ MPa} \qquad \textbf{(10)}$$

$$95\ MPa < 125 \text{ MPa} \quad \textbf{(Pass)} \qquad \textbf{(11)}$$

The two figures below, show the stress peaks at the critical location (the model optimized with two shear key beams).

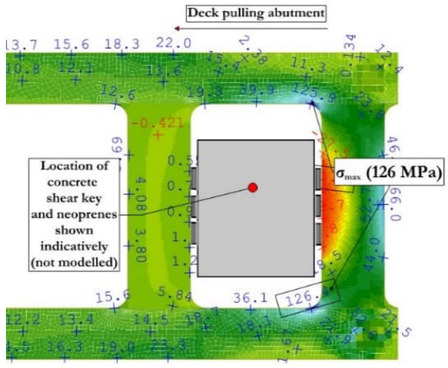

Figure 17. σmax stress peak (126 MPa). Longitudinal force pulling abutment (Option 3).

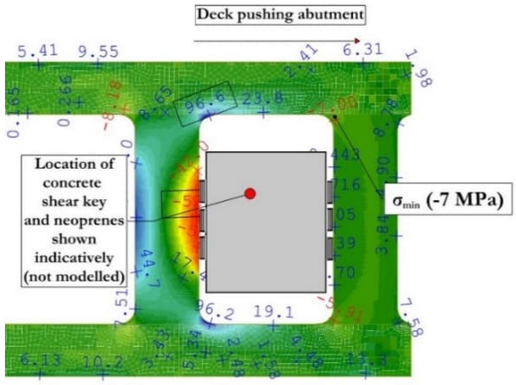

Figure 18. σmin stress peak (-7 MPa). Longitudinal force pushing abutment (Option 3).

4. Concrete abutment design

As explained previously, this long railway viaduct is longitudinally fixed at one abutment, with a reinforced concrete shear key (see brown upstand above the bearing shelf in Figure 19) which needs to resist 24.5MN/12MN of longitudinal and reversable force for the ULS and Fatigue cases, respectively. The design of this reinforced concrete abutment can be split in two distinct features: the shear key itself and the rest of the abutment.

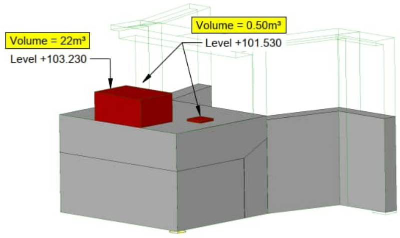

Figure 19. 3d view of the abutment and shear key.

4.1 Shear Key Design.

The shear key is designed as a corbel following a Strut and Tie (S&T) methodology for all design checks: ULS, SLS and Fatigue. Due to the required construction joint at the bottom of the shear key which requires the use of couplers, fatigue check was the governing case.

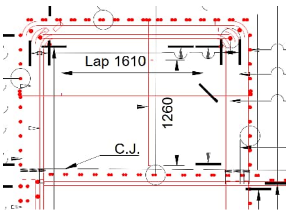

Figure 20. Shear key reinforcement, showing construction joint (C.J.) and couplers

4.2 Abutment Design.

The rest of the abutment body is designed as a simplified 3D S&T model which considers a combined mechanism of a horizontal load path (load transferred horizontally into the wing walls and then down into the foundation) and a vertical load path (load transferred vertically down through the abutment wall into the foundation) by means of a horizontal and a vertical S&T model. This simplified design approach was later validated with a full 3D brick model of the abutment, as shown in Figure 21 below.

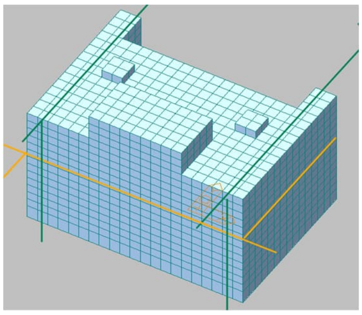

Figure 21. FE volumetric (brick) model

This brick or solid FE model was showing that 60% of the load applied at the shear key was transferred horizontally (and thus 40% vertically); and the horizontal part was mainly taken by the top portion of the abutment (see Figure 22 below). A hand calculation was also performed considering the vertical stiffness of the wall combined with the horizontal stiffness of the top 2m of the wall and was showing a 50% load distribution between horizontal and vertical load path, respectively, which is very close to the FEM results.

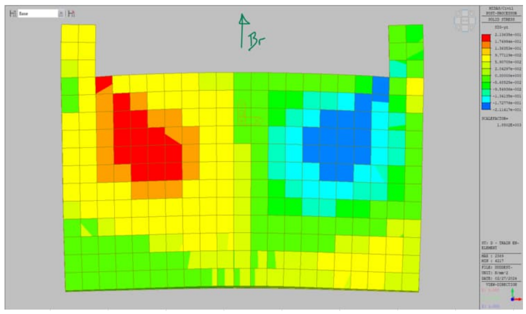

Figure 22. Horizontal slice from the brick model taken at the top of the bearing shelf, showing the compression struts going into the wing wall

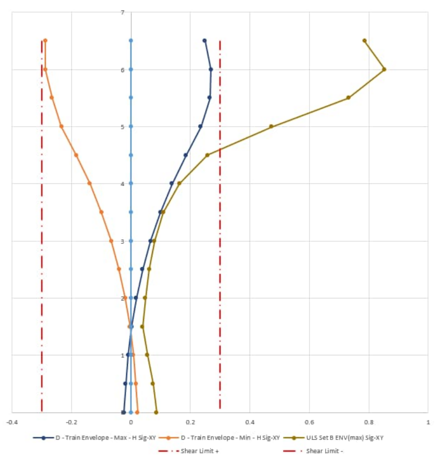

Figure 23. Vertical plot of horizontal stresses taken near the wing wall showing how the horizontal part of the load is mostly transferred by the top 1.5 to 2m.

Then, two separate S&T models were developed and resolved for the horizontal and vertical load paths, figures 24 and 25 respectively, to inform the reinforcement requirements and very importantly the points beyond which the main rebar should be anchored or lapped. Generally speaking due to the use of couplers or the rebar bends, fatigue checks were also governing here.

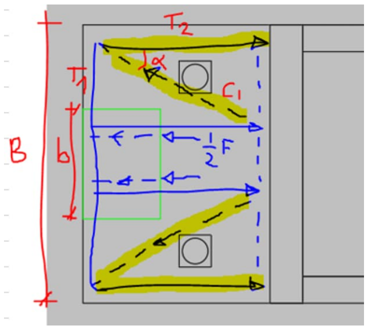

Figure 24. Horizontal S&T model for the top 2m below the bearing shelf

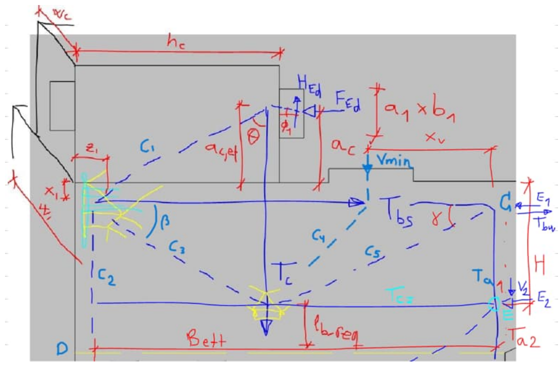

Figure 25. Vertical S&T model

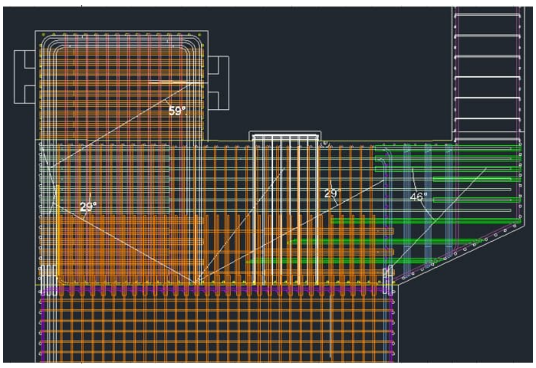

Figure 26. Abutment section with reinforcement and angles of the compression struts considered for the vertical S&T model

5 Discussion, Conclusions and Acknowledgements.

The paper delves into some of the design complexities of a high-speed composite viaduct in Birmingham (UK) whereby Fatigue has generally been the design governing case for all the steel elements but also for the reinforced concrete abutment, as per [3] and [2], respectively. In both cases, FE models with shell or solid elements were required, which normally takes large amount of computational time and effort; however, these challenges were largely overcome in an efficient manner and reasonable timeframes thanks to the use of parametric modelling.

Due consideration of the construction process and methods was directly affecting the design, which makes evident how crucial is a good interaction/collaboration between design and construction teams in these designs so that a successful and safe design is achieved.

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6 References.

[1] BSI British Standards. BS EN 1990:200 Eurocode - Basis of Structural Design.

[2] BSI British Standards. BS EN 1992-2:2005 Eurocode 2 – Design of concrete structures Part 2: Concrete bridges – Design and detailing rules.

[3] BSI British Standards. BS EN 1993-1-5:2024 Eurocode 3 – Design of steel structures Part 1-9: Fatigue