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CRITICAL ANALYSIS OF THE DESIGN AND CONSTRUCTION OF THE SHEPPEY CROSSING

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Abstract: This paper will provide a critical analysis of the Sheppey Crossing. Constructed in 2006, the bridge is a dual carriageway viaduct carrying the A249 over the Swale Channel, designed to provide un-interrupted access to the Isle of Sheppey. This paper will analyse the aesthetics, design solution, construction technique, loadings, strength, main piers and ship impact and bridge detailing issues.

Keywords: fixed link, double curvature, composite deck, launching, ship impact

1 Introduction and Purpose of Bridge

The Sheppey Crossing is part of a £100 million improvement scheme by the Highways Agency to improve the A249 between the M2 and the port of Sheerness. The £30 million bridge acts as a fixed, high-level road crossing the Swale Channel between the Isle of Sheppey and mainland Kent (Fig. 1). It is to provide an alternative to the existing Kingsferry lifting bridge. This was previously the only method of access to and from the island, however has two major drawbacks. It only has a single lane of traffic available for users in both directions, and has to be lifted on a regular basis during the day for shipping traffic to pass. Both these issues cause inconvenience to users, by creating long delays to traffic.

Figure 1: Sheppey Crossing, with Kingsferry Bridge running along side on the right hand-side

The new crossing, designed and constructed by Carillion, aims to provide major benefits by reducing journey times and congestion, as unlike the Kingsferry bridge, is not required to be lifted to allow for shipping traffic to pass. Furthermore, the new crossing carries two lanes of traffic in each direction, allowing for greater volume of traffic. However, it should be noted that the bridge only permits motorised vehicle users, and is a clearway and no stopping is allowed. Pedestrians and cyclists are not permitted to cross the bridge.

2 Final Design Solution and Geometry

The new 1.3 km long composite bridge consists of 19 steel deck spans, which sit on 36 reinforced concrete columns, which range between 7 metres to a maximum clearance of 29 metres height at midspan. The deck comprises of four main plate girders that follow the plan curvature, with cross girders in-between. In Ref. [1] is it explained that the final construction of the Sheppey Crossing used 10,000t of fabricated steel plate girders and 60,000t of structural concrete.

The bridge has a curvature in plan and vertical direction. This is to allow shipping traffic to pass freely under the midspan of the bridge, as well as to stick to the narrow corridor confinements imposed by the surrounding environment. From Ref. [3] it is highlighted that across seven of the spans, the plan geometry is 1500m radius and over 11 other spans the radius is 3500m. The overall deck width is 22.4m, which provides two standard carriageways in both directions. It should be

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noted that the spans grow in length gradually from the abutments. To account for this increase, the deck thickness is also increased proportionally from 1.6m at the abutments to a maximum of 3.6m at the main central span, which is 92.5m long,

There may be certain aspects of the bridge design and construction that is unclear or lacking information. In these circumstances, appropriate assumptions will be made to assess the bridge appropriately.

3 Bridge Aesthetics Considerations

To assess whether this bridge is aesthetically pleasing is difficult to quantify. However, using Fritz Leonhardts rules on bridge aesthetic it is possible to appraise how successful the Sheppey Crossing has been at fulfilling visual criteria. The bridge will not be evaluated on all ten aspects of Leonhardts aesthetic rules, but those deemed most appropriate to apply to this structure.

The use of a composite bridge, which in essence in this case is a combination of steel girders with a concrete slab on top of piers, seems an appropriate decision to have been made. Considering the relatively short span across the Swale and the need for a cost-effective solution that does not cause visual obstruction the chosen bridge type is more than appropriate.

Comparatively, taking into consideration the criteria to minimise environmental damage and budget, other solutions like cantilever bridges or suspension bridges would have cost far too much, and would not have fitted in successfully with the surrounding context. Furthermore, to achieve the brief requirements for a fixed link that allowed shipping traffic to pass freely using other bridge types would have required huge and visually obtrusive designs.

At first glance, the overall visual appearance of the bridge is one of a bland and uninteresting appeal. It seems like another typical steel and concrete composite bridge, lacking colour and character to set it apart from other similar structures, with its presence seeming only to provide a new form of access across the Swale.

However, by keeping the colours of the Sheppey Crossing to its original construction material colours has allowed the bridge to immerse its self into the surrounding area gently, integrating itself with the natural environmental surroundings successfully. It is only with in-depth assessment and understanding that the bridges true colours begin to reveal themselves.

A striking aesthetic feature of this bridge is the curvature provided in the plan and vertical direction (Fig. 2). Ref [6] explains that bridge structures with curvature can have higher costs compared to those with straight alignments, due to increased design, fabrication and construction costs. However, it adds aesthetically to the bridge in numerous ways, and can be cost-effective with the associated functional improvements weighed up.

Figure 2: Elevation showing the Sheppey Crossing subtly blending with landscape

The double curvature of the Sheppey Crossing provides character for the bridge, giving a more pleasing appearance. It gives the structure an elegant elevation that subtly blends into the surrounding landscape along with the unusual arrangement of the substructure. The sweeping curve of the bridge brings a sense of excitement to the design, compared to a traditional straight girder deck bridges.

However, the use and need for the vertical curvature does pose an issue. The degree of curvature does not seem to allow road users to assess visibly where they are going (Fig. 3). This can make road users anxious travelling across the bridge. This is a basic requirement that has been compromised by the need for a high-level fixed link that allows shipping traffic to pass.

It should also be noted that Ref. [6] explains that in a structural aspect, the use of curvature in the design gives an inherent need for appropriate torsional strength against vertical loads acting on a curved bridge that cause twisting moments. This can in turn lead to an increase in deck thickness or columns, and thus increase in costs.

Figure 3: Steep approach to the bridge

Although this structure may not be complex, with little defining features, it does provide a simple and elegant design that is understandable. It has been designed as a robust bridge, successfully rendering a sturdy impression of stability for users. It portrays its function and reveals its structural form and purpose. This is achieved by using appropriate aspect ratios between column sizes and deck spans and thicknesses. It should be noted that use of a varying span length and column height adds an elegant touch to the bridge, portraying good use of proportions.

By subtly tapering the girder depth to match the varying spans has enabled the designers to maintain a sense of proportion and structural efficiency for the bridge. It should be noted that it is also possible to achieve this look by curving the bottom flange upwards. This is further reinforced by the use of an uninterrupted and continuous fascia beam (Fig. 4). It is highlighted by Ref. [2] that by eliminating vertical stiffeners from the edge elements has allowed the designers to minimise the number of bolted splices required, giving a smoother finish to the fascia.

However, relative to the overall size of the bridge, a change of 2m in deck depth is not completely visible or noticeable. In contrast, where the deck span begins to decrease as you move towards the abutments and column locations become closer together is more visible, and may cause people to question why they have done this, as the decrease in deck depth has not been emphasised. This issue could be solved by using loud colouring in the fascia beam to highlight the change in depth.

Figure 4: View of fascia and parapet

The superstructure can also have aesthetic implications from beneath the deck. For the Sheppey Crossing the use of steel girders for the superstructure has had aesthetic drawbacks, creating numerous lines and edges on the soffit (Fig. 5). It seems for the Sheppey Crossing, the soffit has been a victim of excess repetition of elements. Along with the cross girders, the soffit, aesthetically, has too many components, creating numerous edges, and possibly leading to mental disquiet as explained in Ref. [4]. However, the soffit is rarely gazed upon due to its location and thus not an important issue, compared to the structural benefits achieved using this construction type. The benefits will be discussed later in the paper.

Figure 5: Soffit view, order has been compromised through excess repeating of elements

The substructure can also have a significant visual effect on the overall aesthetic appearance of the bridge. For example, using twin-tapered columns in the design has added aesthetic quality to the bridge. This method, according to Ref. [4] was used by the Greeks, and prevents the optical illusion that the top of a column looks wider than its base, which is required to take more bending. The use of only two columns has helped prevent creating any oblique barriers across the main spans over the Swale, but becomes more apparent as you reach the abutments where the columns are in closer proximity to each other. However, the latter is of less importance to have achieved, as the main visual view is present over the midspan crossing the Swale.

In summary the overall aesthetic appeal of the bridge is positive. It has been successfully integrated into the environment considering the other forms of bridge that could have been used. It has character, derived from its subtle curvatures and elegant proportions, with geometric balance between depths and spans. It may not be a loud or complex bridge, but this is to its advantage considering its surrounding environment, as it blends in well for its set context.

4 Construction

4.1 Reasons for Bridge Deck Type

There is scope for the Sheppey Crossing to have been constructed using a different bridge deck type. However, because of the length the bridge was required to span and the environmental restrains imposed on construction techniques, it seems that a composite bridge would be the most cost-effective solution. Other solutions available could have been concrete or steel box sections. However, it seems that the use of girders for the deck was based mainly on the advantage that they could be purchased easily as prefabricated products. It should be highlighted that uses of box girders also have their advantages.

The main advantages of box girders come from the torsional rigidity of the closed cell, which becomes important as spans increase. Box girders have a high torsional stiffness, which makes them efficient to resist bending relative to the material used. However, fabrication is more expensive than plate girders, and thus a major drawback for a small budget project.

When considering steel box sections an issue of instability arises, as they are prone to buckling problems. In order to stiffen steel box sections, often the top flange is topped with a concrete slab, forming a composite section. Hence, in the scope of things it seems easier to use a plate girder composite system, which is cheaper, and easier to construct. For example, if the concrete slab is not provided on the steel box section during construction, it can often lead to collapses.

4.2 Prefabrication

For the Sheppey Crossing, using plate girders to create a composite deck allowed the use of prefabricated elements. The girders, as factory made, are likely to be of a higher quality. However, elements that were launched into position were welded and joined together on site, which may have provided less quality. Sections that were lifted into place were welded or joined via splice bolts as lifted individually into position. The issue with transporting these girders to the site is that it cannot only be expensive but also hazardous, as large vehicles are required to carry huge segments of the deck.

4.3 Construction Technique

It should be noted that because of the bridges varying plan and vertical geometry, its location over water and limiting footprint available due to environmental considerations has created major constraints for the type of construction possible to use for erecting the bridge steelwork. Considering the type of bridge and constraints implicated in the project the only viable solutions would be either launching or lifting the bridge into place.

If a lifting mechanism was used to erect the steel bridge components in to position either a heavy lifting vessel would be required, similar to the "Svanen" used in the construction of the Oresund Bridge or a jacking lifting system (Fig. 6), similar to the one used on the Shibanpo bridge. Using either of these methods would eradicate many of the dangers and issues involved with launching the steelwork. Because of the curvature and small size of the bridge, it is unlikely that a jacking lifting system would be used, but use of a heavy lifting vessel could be appropriate for this project. However, this method would cause inference with river traffic as well as being expensive. The lifting process will also be dependant on river conditions, as strong currents could make lifting dangerous and inaccurate.

Figure 6: "Svanen" used for the construction of the Oresund Bridge

Although the process is very expensive, using a lifting system does allow the bridge deck to be smaller, as there are no compression forces exerted on the deck during construction, as would be present during launching. However, because of the bridge's location, lifting sections into place is not a practical option, whereas a sequential launch program could be more advantageous (Fig. 7). When considering the narrow construction corridor available for construction to take place, it seems sensible to use launching as the construction technique where possible. It should be noted that Ref. [3] states that the launching process requires that the girders have constant width bottom flanges, with no split bottom-bolted splice cover plates for smooth construction over launch pads.

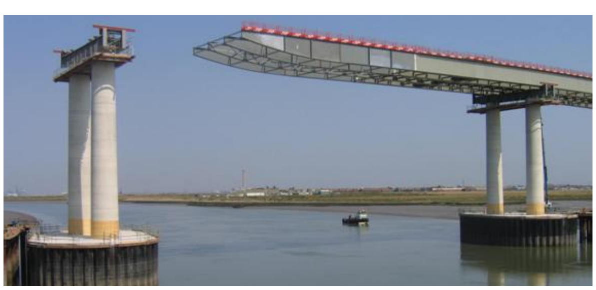

Figure 7: Launching of the deck using a steel nose across the central span of the bridge

An issue with the construction technique chosen for the Sheppey Crossing is that because of its double curvature and changing horizontal radii it required two new steel noses to be fabricated for the job as it was also to be launched from both abutments. This is expensive and seems impractical, as it may only be suitable for this job, which is not a cost-effective solution, and a waste of materials. Because of the extra weight of the deck to carry the loading imposed during launching, it meant that a lighter steel nose was used to minimise the overall weight. It seems that the process becomes more complicated and intricate by using launching as the construction technique.

As the bridge does not have a constant curvature, it means that it would not be possible to launch all steelwork components, but would require several sections to be erected via a crane. This then involves ensuring that the ground the crane sits on is strengthened to take the extra weight (i.e. hard standings). This begins to increase costs, and demand intricate management to ensure all issues are pre-planned and understood to allow for a smooth construction process.

An important aspect that must also be considered during launching is the possibility of misalignment. If during the jacking process, the deck misses the bearings there is the possibility that the column could puncture the steelwork where stiffeners have not been provided. The VSL pulling system is the most likely technique used to launch the bridge. This method is inherently safer than pushing the structure using a lift-and-push system, where it is more likely to offset the deck from the bearings on the columns.

To allow the deck to slide over the piers smoothly, preventing any friction occurring between the deck and the column, a low-friction polytetrafluoroethylene (PTFE) pad is most probably used on top of a stainless steel sheet placed on top of the column. Then once the entire deck has been positioned the permanent bearings are added on to each pier as the bridge is slightly lifted. It should be noted that wind loading during this process can be hazardous.

Another consideration that must be made using launching for the plate girders is the temporary stability required during the launch. For the Sheppey Crossing they have considered this at the worst locations by providing plan bracing between the two inner main girders.

4.4 Construction Process

Launching the deck as the principle method of construction has also had its disadvantages. As the central span requires the deepest deck it meant that for the construction process to be successful would require the deck to be launched from two points, working towards the middle.

If the deck was launched from one side, because the deck is deeper closer to the nose, and thinner further down the deck, it would cause issues, as when it crossed the central span it would have likely cracked under the tremendous stresses generated from the deeper, heavier deck next to the launching nose (Fig. 8). It is therefore more practical and cost-effective to have launched from two sides than use temporary formwork/cables or to have increased the overall deck thickness to take the stresses generated during launching.

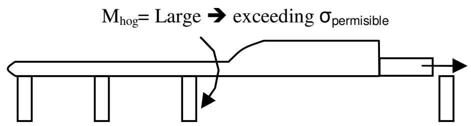

Figure 8: Schematic diagram showing if deck was launched across the whole central span

It should be noted that for the purpose of this construction a secondary steel nose was fabricated due to changing radii and to allow for the deck to be launched from both abutments to meet in the centre. Therefore, it must have been a far more cost-effective solution to fabricate two steel noses and launch from both sides than use another construction technique, even though they both incur high costs. The construction of the superstructure occurred in five phases, and is summarised in Fig. 9.

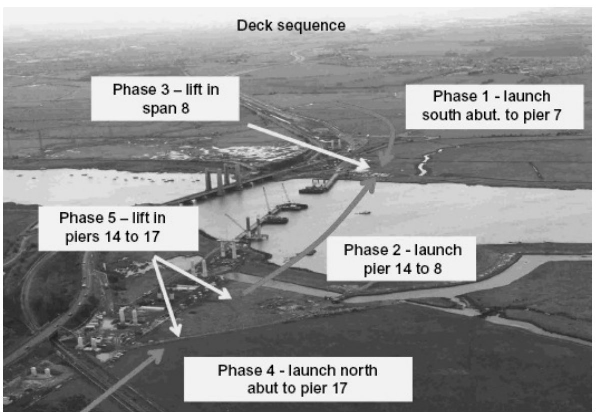

Figure 9: Steelwork erection phases - Ref. [3]

It is explained that in Ref. [1] that fifteen of the spans were launched, while the other four where erected via a crane once the launching program was complete (Fig. 10). The sections that were erected using a crane required hard standings to taken the heavy loadings. However, by reducing the number of sections that required heavy duty ground improvements, saved time and costs. The erection process was also carefully considered, being undertaken during the night to minimise disruption caused to traffic during the day by the heavy lifting equipment required. Once the steel skeleton had been erected the pre-cast concrete planks where then added to the deck, and surfaced with a low-noise road surface, considering the surrounding natural environment.

Figure 10: Lifting of deck girders into place

4.5 Analysis of Launching Loads

During launching of the bridge, every cross-section experiences the maximum hogging and sagging moments. This means that the deck may be required to be larger to account for construction loading. A simple check can be performed to assess whether the deck is required to be larger due to construction loading compared to in-service loading of the completed bridge in use.

It will be assumed that the uncompleted deck, comprising of only the steel skeleton, weighs 350 kN/m, using a weight of 7850 kg/m³ for steel, and the light steel nose used during launching weighs 1000kN, spread uniformly over its length of 30m (Fig. 11). The largest moments are most likely to occur at the midspan, which is the longest span of the bridge at 92.5m. However, because the deck is being launched from two locations the deck will not be launched the complete distance, so it is assumed that it is launched to just over half the central span length of 50m to give a more conservative value. The second moment of area of the steel deck has been calculated as 4.8 x 10^{12} mm^4, assuming sizes of the four steel I-beams used at midspan, which are 3.6m deep. A y-value of 1750mm has also been taken.

It should be noted that the steel is assumed as grade S355, as this is most commonly used in bridge construction. This is because of the cost-to-strength ratio is lower compared to other grades as explained in Ref. [7].

The maximum hogging in the deck during construction is going to occur when the deck is cantilevering 50m over a column, shown in Eq. (1):

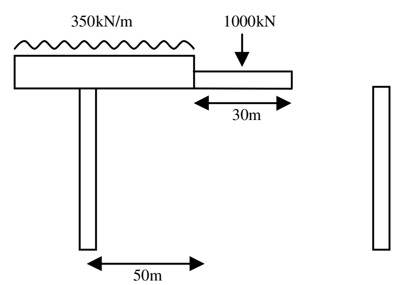

Figure 11: Maximum hogging experienced in deck during launching

Dead Load	= 350 kN/m
Dead Load Factor, γ_{fl}	= 1.15
ULS for steel, γ_{f3}	= 1.10
Factored Dead load	= 443 kN/m
Nose loading	= 1000kN
Erection factor	= 1.15
Factored Nose Loading	= 1150kN

$$M_{hog} = M_{deck} + M_{nose} \,. \qquad (1)$$

$$= (443 \times 50 \times 25) \times (1150 \times 30) = 587 \times 10^{3}\, kNm$$

Using the hogging moment generated from Eq. (1) when the deck is launched over half the central span it is possible to calculate the stresses generated in the deck and compare it to the allowable permissible stress of the steel deck:

$$\sigma_{actual} = \frac{M_{hog} \times y}{I_{xx}}. \quad (2)$$
$$= \frac{583 \times 10^6 \times 1750}{4.8 \times 10^{12}}$$
$$= 215 N / mm^2 < \sigma_{permissible} = 355 N / mm^2$$

As shown in Eq. (2), the stress generated during the deck launching is far less than the allowable permissible stress in the steel deck. If the deck hadn't been launched from two different directions the launch would have to cover a span of 92.5m, which would have produced stresses of 326N/mm², which is relatively close to the allowable limit. Therefore it has been a clever initiative to have launched the bridge from two directions, reducing the stresses experienced in the deck during the launch. Later in this paper the maximum moment caused via a completed in-use deck shall be calculated and compared to the stresses generated during the bridges launching.

5 Main Piers and Ship Impact

5.1 Foundation & geotechnics

It should be noted that the main piers constructed in the Swale Crossing, is mostly likely to comprise of alluvium and London clay. The most likely form of foundations used are conventional piles below all intermediate pier and abutment locations. Ref. [3] explains how piles can act in both skin friction and end-bearing in the London Clay. This adds to their resistance against ship impact.

5.2 Main Piers

To construct the main central piers, it was firstly required to construct temporary jetties in-order to enable heavy plant equipment to reach the locations. Once the jetties were constructed cofferdams where put in place to allow work to continue undisrupted in temporary watertight enclosures (Fig. 12). The central piers where then constructed, as caissons where constructed directly on the London clay. This provides sufficient resistance concerning ship impact. The piers were then cast in-situ.

Figure 12: Construction of cofferdams via temporary jetties

5.3 Ship Impact

As the bridge crosses a busy shipping traffic route consideration must be made for the possibility of ship collision into the piers. If assumed a similar impact capacity to that of the Severn Crossing of 36MN from Ref. [3], it is possible to check whether the central piers and could withstand a ship collision of 6500 dead tonnage weight (DWT) using Eq. (3):

$$F_{impact}(MN) = 0.44\sqrt{(DWT)}. \quad (3)$$
$$F_{impact} = 35.5MN$$

Therefore, it seems the piers would withstand a ship collision. However, considering the possibility that one of the main central piers did collapse, leaving half the deck to be simply supported between approximately 130m span, it would be essential to check whether generated stresses within the deck exceed the permissible (355N/mm²). It is assumed that there is no traffic loading in this case, only dead load over half the deck, the maximum sagging is calculated in Eq. (4):

Dead Load (in-service)	= 380 kN/m
Dead Load Factor, γ_{fl}	= 1.15
Factored Dead load	= 437 kN/m
SI dead	= 40 kN/m
SI Dead Load, γ_{fl}	= 1.75
Factored SI load	= 70 kN/m
ULS for steel, γ_{f3}	= 1.10
Total Factored load	= 558 kN/m
Over half the deck	= 297 kN/m

$$M_{sag} = \frac{W_{deck} \times L_{deck}^2}{8}. \quad (4)$$
$$= \frac{279 \times 130^2}{8}$$
$$= 590 \times 10^3 kNm$$

$$\sigma_{actual} = \frac{M_{sag} \times y}{I_{xx}}. \quad (5)$$
$$= \frac{590 \times 10^9 \times 1750}{2.4 \times 10^{12}}$$
$$= 429 N / mm^2 > \sigma_{permissible} = 355 N / mm^2$$

Considering if one of the central piers were to be knocked out, it seems that the deck would be unable to support its self between supports as the stresses generated in Eq. (5) are far greater than the permissible allowed in a steel deck. However, it seems unlikely, unless due to terrorism that a pier can be taken out as it has been constructed to take very high loading.

6 Loading

6.1 Dead load

The dead load considered for this paper has been assumed to comprise of the steel girder superstructure and the concrete slab deck. It has been estimated that the total dead weight is approximately 380kN/m.

6.2 Superimposed Dead Loading

The superimposed dead loading on the bridge is assumed as $2kN/m^2$, and thus taken as 40kN/m across the longitudinal length of bridge. This includes loading from finishes and surfacing, including the concrete parapets.

6.3 Live Load

In order to calculate the traffic loading on a highway bridge, we must define the notional lanes present on the bridge. It should be noted that for the purpose of this paper the deck will be assumed to act as a composite deck, and not as single girders. The Sheppey Crossing has an approximate carriageway width of 22m carrying two different directions of traffic, giving a total of 6 notional lanes in both directions. There are two forms of live traffic loading that need to be applied to the bridge that are HA and HB loading.

HA loading consists of a uniformly distributed load (UDL) and a knife-edge load (KEL). The loading depends upon the loaded length of the bridge, which is logical since it is statistically less likely that a long bridge will be loaded to the maximum intensity over its entire length. The loaded length will be applied at various positions along the length of the bridge, possibly with different load patterns, in order to evaluate the worst load case.

HB loading represents an abnormal vehicle loading on the bridge and consists of a group of wheel loads being applied in place of the HA loading in one lane of the bridge, in the worst place. Only one HB vehicle shall be considered on any one superstructure.

6.3.1 HA Loading

The loaded length of the bridge is greater than 380m, therefore giving a nominal unfactored HA UDL of 9kN/m of notional lane. The method of loading the bridge is done by putting two full HA loading on two of the bridges notional lanes, with the remaining four notional lanes being having 1/3rd of the full HA loading. A Knife Edge Load of 120kN per notional lane will be applied transversely across a line on the deck along with HA loading.

6.3.2 HB Loading

Concerning HB loading, it is assumed that a vehicle of four axels, each with 4 wheels, is placed on one notional lane that is deemed to give the most onerous loading case. For each axle, a 10kN unit loading weight is taken, with full HB loading being taken as 45 units. Concerning HB loading, it should be noted that 25m clearance before and after the HB vehicle should not be loaded. Full HB loading therefore comprises of 450kN for each axle load.

6.4 Natural frequency

It should be noted that because this is solely a highway bridge the effects of vibration due to traffic have not been considered as significant.

6.5 Creep

Creep has been ignored for this paper as this is a predominantly steel bridge, and therefore effects of creep have been considered insignificant.

6.6 Temperature

Temperature fluctuations can be of a huge concern considering the main structure of the bridge is steel, creating high stresses in the bridge. This risk is coupled if the bridge design is poor and the bridge is made too stiff, and thus unable to accommodate for movement. The two temperature effects that occur in bridges include effective temperature, where the overall temperature increases and decreases, and temperature difference, where the temperature below and above the deck differ.

If we consider effective temperature changes in the bridge, we can assess the stresses generated by the overall bridge expansion or contraction. Using U.K. isotherm maps we can estimate the range of temperatures likely to occur for the Sheppey Crossing as between –10°C and 31°C, giving a range of 41°C. Assuming there are expansion joints provided at quarter point locations, giving a length of 650m in between them, and taking the coefficient of thermal expansion of steel as 12 x 10^{-6} we can calculate the movement of the deck in Eq. (6):

$$\Delta L = \Delta T \times L \times \alpha \ . \tag{6}$$
$$\Delta L = 41 \times 650000 \times 12 \times 10^{-6}$$
$$\Delta L = 320mm$$

Relative to the length of deck between the expansion joints assumed, the movement is high. However, the use of expansion joints allows horizontal movement to occur within the deck, so that compressive stresses do not build up. A method that may have been used in this bridge to reduce longitudinal temperature stresses is to use piers that consist of two separate columns that are stiff in bending but flexible to move laterally at their tops. This then prevents high longitudinal stresses from being developed in the deck.

However, the use and location of expansion joints for the Sheppey Crossing is unknown. Therefore, the worst case will be analysed, which would be if no expansion joints were provided or if there were clogged, causing them to be ineffective. The bridge would be unable to move, and compressive stresses would be generated within the bridge. The compressive stress generated in this case is calculated below in Eq. (7):

$$\sigma_c = T \times \varepsilon \times E \ . \tag{7}$$
$$\sigma_c = 41 \times 12 \times 10^{-6} \times 200000$$
$$\sigma_C = 98N / mm^2$$

Considering that steel can take about 100N/mm^2 before buckling, this force calculated in Eq. (7) is huge, taking up most of the capacity available for other loading. The other form of temperature fluctuation causing further loading on the bridge is the temperature difference between the bottom and top of the deck.

This is because the bottom elements of the bridge are at the same temperature as the air, and the upper elements are exposed to solar radiation, wind and precipitations, causing a temperature difference throughout the deck. Using BS 5400 it is possible to estimate the temperature difference between the top and bottom of the deck as 14°C and thus calculate stresses generated in Eq. (8):

$$\varepsilon_{max} = \Delta T \times \alpha = 14 \times 12 \times 10^{-6} = 168\mu \quad (8)$$
$$\sigma_{max} = \varepsilon_{max} \times E \,.$$
$$\sigma_{max} = 168 \times 10^{-6} \times 200000 = 34N / mm^2$$

In order to calculate the resultant axial load and bending moment in the steel deck, caused by this difference in temperature, the centroidal axis is required. For the purpose of this paper, it will be it will be assumed that the centroidal axis is 2/3 up from the soffit. Figure 13 shows diagrammatically the axial and bending moments:

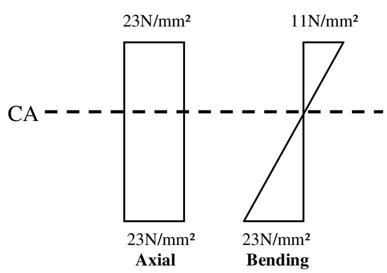

Figure 13: Axial and Bending moments generated in deck due to temperature change

6.7 Aerodynamic Considerations

Using the Design Rules for Bridge Aerodynamics, BD49/01, which forms part of the UK Highways Agency's Design Manual for Roads and Bridges, Ref. [3] explains how the bridge was assessed to check that the edge profile of the bridge complied with the geometric constraints outlined. The bridge failed the assessment because the design rules did not apply as the edge profile lied outside the range of bridges tested during the compilation of the code.

As the bridge failed the assessment, because the rules did not apply in this case, it was required to check the edge profile using a wind tunnel test to ensure the requirements for vortex shedding and divergent amplitude response were adequate. The wind tunnel testing proved successful, as the edge profile was deemed to be within the limits required by BD49/01 for vortex shedding and divergent amplitude.

6.8 Wind

Wind loading on a bridge acts in several different ways through horizontal drag, vertical lift and torsional forces, with their relative magnitudes depending on the geometry of the cross-section. This section will focus on only horizontal drag forces, ignoring vertical uplift/downward forces, because the Sheppey Crossing is a robust bridge. It should be noted that wind loading will be assumed to be most critical at the maximum height, which is approximately 35m high at the midspan, at which has a span length of 92.5m.

Wind loading magnitudes vary depending on the location of the bridge. For the Sheppey Crossing, which is located in Kent, we can assume that the mean hourly wind speed is approximately 26ms^{-1}, using an isotachs map from BS5400. In order to calculate the maximum wind gust that could occur on the bridge requires the wind coefficient, K_1, and hourly speed factor, S_2. These are 1.66 and 1.24 respectively and related to the height of the bridge. Note that a funnelling factor, S_1, is not considered as it is deemed not to affect this bridge. Therefore, the maximum wind gust, v_c, is from Eq. (9):

$$v_c = v \times K_1 \times S_2 \,. \quad (9)$$
$$v_C = 53.5m^{-1}$$

The horizontal wind forces acting on the bridge can therefore be calculated using the dynamic pressure head, projected unshielded area of the deck and the drag coefficient. The section of deck that is considered for this calculation is at midspan, which has a depth of 3.6m and span length of 92.5m, giving an area (A_1) of 333m². Note that the parapet is solid and the height of the area is taken from the bottom of the girder to the top of the live load. Using BS5400 Cl. 5.3.3.2.4 "two or more plate girders" the relative drag coefficient, C_D, is given as 2.2. Therefore the horizontal loading is calculated in Eq. (10):

$$P_t = q \times A_1 \times C_D \,. \quad (10)$$
$$q = \left(0.613 \times v_c^{\,2}\right)$$
$$P_t = (1755) \times (3.6 \times 92.5) \times 2.2$$
$$P_t = 1288kN$$

This is the maximum horizontal wind loading that will be present on the bridge, occurring at the midspan, which is the tallest point of the structure. This is the horizontal force that is required to be resisted by the bearings on the main span.

It is possible to perform a similar calculation to evaluate the wind loading on the piers. Taking the piers at the midspan, and assuming dimensions of height 29m and thickness 3m a drag coefficient can be estimated from BS5400, "Table 9". With a height to breadth ratio of approximately 10, and assuming they have a rough surface, C_d will be taken as 0.9. Using Eq. (10) again we can find the load on the pier:

$$P_t = \left(0.613 \times 53.5^2\right) \times (3.6 \times 29) \times 0.9 \quad (11)$$
$$P_t = 138kN$$

This load from Eq. (11) can be translated into a UDL loading occurring on the pier of 4.8kNm.

6.9 Numerical Analysis of the Sheppey Crossing

Using the worst load case, which is assumed as all permanent loads and primary live loads acting on the Sheppey Crossing, the bridge will be analysed using Eq. (12 & 13).

For the Sheppey Crossing the following traffic loading combination will be applied as it is considered the most severe longitudinal loading on the deck:

- 1 lane with full HA Loading
- 4 lanes with 1/3 HA Loading
- 1 lane with a HB vehicle of length 10m centre of the span
- KEL acting midspan on all lanes

Considering Longitudinal Loading on Bridge over 92.5m span for chosen load combination:

Factored Dead load	= 437 kN/m
Factored SI load	= 70 kN/m
Traffic Load	= (1x9x92.5) + (4x1/3x9x92.5) + (4x450) + (6x120)
	= 4453 kN
	= 202 kN/m
Traffic Loads, γ_{fl}	= 1.30
Factored Traffic load	= 263 kN/m
Total Load	= 770 kN/m
ULS for steel, γ_{f3}	= 1.10
Factored total load	= 845 kN/m

$$M_{sag} = \frac{W_{total} \times L_{deck}^{\ 2}}{8}. \tag{12}$$
$$= \frac{845 \times 92.5^2}{8}$$
$$= 905 \times 10^3 \, kNm$$

$$\sigma_{actual} = \frac{M_{sag} \times y}{I_{xx}}. \tag{13}$$
$$= \frac{905 \times 10^9 \times 1750}{4.8 \times 10^{12}}$$
$$= 330 N / mm^2 < \sigma_{permissible} = 355 N / mm^2$$

The calculated stress in the deck due to in-service loading and traffic loading is less than the allowable stress showing the deck is of adequate strength. The stress generated is greater than the stresses due to hogging during launching of the bridge. This means that the deck depth is not dictated by construction loading, but by final dead and traffic load. This shows good design principle, and good initiative to launch the bridge from two directions, as it has produced a more cost-effective solution.

7 Bridge Detailing Issues

When it comes to designing a bridge all aspects of design, construction and use must be covered. However, in the case of the Sheppey Crossing unnecessarily features of the bridges use have been ignored. The following is some detailed aspects taken from Ref. [3] and expanded upon, explaining their implications on road users and overall bridge design. This section will focus on the detailing that was missed.

A significant issue that has been noticed is that there is no provision of lighting on the bridge, or emergency lighting (Fig. 14). Considering if a motorist breaks down during night-time then emergency services attending the incident will have reduced safety whilst dealing with the incident on the bridge. Lighting is not provided on the basis that TA 49/86, "Appraisal of New and Replacement Lighting on Trunk Roads and Trunk Road Motorways", which is one of the standards in the Design Manual for Road Bridges, does not identify the Sheppey Crossing as requiring lighting. If lighting were to be provided, it would best be located on the outer side of the parapet, to avoid compromising its function in the event of a vehicle impact. However, this would degenerate from the slender visual the bridge has in elevation. So it would be preferable to install lighting along the inside of the parapet.

Figure 14: No provision of lighting provided

Considering the curvature and gradient of the bridge, it may be considered that the speed limit of 70mph, especially during the night where there is no provision of lighting, is dangerous. However, this is deemed a satisfactory speed limit in accordance with TD 9/93, "Highway Link Design". It is possible to calculate the Safe stopping sight distance for a highway using, Eq. (14), total breaking distance = $d_1 + d_2$ (m), assuming:

Design speed, *V*	= 110km/hr (equivalent to 70mph)
Reaction time of driver, t	= 2.5 seconds
Coefficient of friction, μ	= 0.30

Reaction distance, d_1 = *t x v (m).*
= *2.5 x 0.278V*
= *76m*

Breaking distance, d_2 =
$$= \frac{V^2}{254.\mu}.$$
$$= \frac{110^2}{254 \times 0.3}$$
$$= 159m$$

Therefore, S
$$= d_1 + d_2. \tag{14}$$
$$= 76 + 159$$
$$= 236m$$

Under current, the speed limit allowed on the bridge, vehicles travelling at 70mph require 236m to come to a halt. When combining this with lack of light and the issue of not being able to see clearly this is a very long distance, and increases potential dangerous of disasters occurring.

It should be noted that the bridge has been designed solely as a vehicle crossing. It therefore has no pedestrian walkways, but also no hard shoulder. Therefore, there is no refuge for broken down vehicles. Danger can arises when high volumes of fast flowing traffic approach a broken down vehicle. In instances where this has occurred, the bridge has had to shut temporarily while the situation is fixed to prevent any fatalities. It was argued that because this is deemed a dual carriageway it is common not to include a hard shoulder. To have increased the bridge width to accommodate for a sufficient hard shoulder was stated as inappropriate. It was explained that 6 metres would have been added on to the deck width, increasing the impact of the bridge on the environmentally sensitive marshland and significantly increasing the cost of the bridge. Instead the Kingsferry Bridge is required to be present to account for any closures on the Sheppey Crossing.

Another detail that has been ignored is the provision of an emergency telephone on the bridge. Although this may seem insignificant, can be a major issue. When combined with the lack of hard shoulder and absent lighting it is possible to understand the hazards a motorist may be subjected to in an emergency, having to walk 1.3km along the sides of the roads until they come off the bridge. This may just cause additional delay in alerting emergency services and allowing hazards to be present for longer, leading to opportunity for another accident to occur.

As the bridge has prohibited pedestrians and cyclists to cross it, the handrail has been constructed to a height of 1.2m, in accordance with the Highways Agency's Standard. However, it is still possible to access the bridge by foot, and already several attempts of suicide have been made. Whether the parapet has been constructed to low is difficult to assess because the bridge has been deemed as a vehicle crossing only, therefore pedestrians should not be allowed to use the bridge. Therefore, from a cost point of view it seems the correct measure to take.

However, is the bridge is to last decades, and allows the opportunity for suicides to occur then maybe it would have been more beneficially to construct the bridge with higher parapets, although at an increased cost and aesthetic issues of a less slender bridge, it would save lives in hindsight. However, if a taller parapet were applied it would have adverse effects on the structure of the bridge. The extra height of the parapet would have increased the weight and horizontal wind loading, requiring a stronger more expensive structure. There has been a balance made between benefits and cost.

Furthermore, concerning the bridge parapet, it has been designed to withstand direct impact from a car or light van, but not a heavy goods vehicle (HGV). Only where the bridge crosses the railway line has there been the provision of a stronger parapet to prevent loss of life of a secondary event occurring. Again, bridge cost must have been a dominating factor that eradicated fitting the entire bridge with high containment parapets.

Using an impulse-momentum calculation, it is possible to show that the current parapet is unable to deflect a HGV off the parapet, which should deflect up to a maximum of 200mm to resist the impact. However, the current parapet in use is likely to deflect 3mm for every 1kN of force applied to it. Therefore, if we assume a HGV of 40 tonnes striking the current parapet transversely at 4m/s, taking into consideration that 90% of the momentum of the truck is lost in crumpling, we can show the deflection of the current parapet using Eq. (15-17):

$$\begin{gathered} v = u + at. \\ 0 = 4 + at \\ a = -4/t \end{gathered} \tag{15}$$

$$\begin{gathered} s = ut + 0.5at^2. \\ 0.2 = 4t - 2t \\ \Delta\ (t) = 0.1\ secs \end{gathered} \tag{16}$$

$$\begin{gathered} F.\Delta(t) = m.\Delta(v). \\ F = \frac{(40000kg \times 4m/s \times 10\%\ )}{0.1secs} \\ F = 160kN \end{gathered} \tag{17}$$

Therefore actual deflection of current parapet is: 160 x 3 = 480 mm > 200 mm permissible. Thus current parapet unable to take HGV loads.

Although many of the above issues are deemed insignificant when applied to a trunk road, it just shows poor bridge design considerations at a detail level. Even though it may not state in standards for the use of certain detailing, each bridge should be considered for its use and practicality to save on costs. However, something that may not be considered during the design is the negative externalities of the bridge closing and causing delays and thus requiring the Kingsferry Bridge to stay operational.

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