CRITICAL ANALYSIS OF THE SHOOTS BRIDGE, CENTRAL SPAN OF THE SECOND SEVERN CROSSING.

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Abstract: This paper critically analyses the design of the central Shoots Bridge of the Second Severn Crossing. Its suitability to carry traffic is assessed against the superseded BD37/88 code (to which it would have been built) and the current BD 37/01. Presenting design capacities using both standards identifies any weaknesses in the crossing according to today’s design highlights areas for future improvements.

Keywords: Severn Estuary, cable-stayed, balanced cantilever, caissons, suspended maintenance gantry.

Figure 1: View of the Shoots Bridge on the Second Severn Crossing

1 Introduction

Since the inception of the Severn Tunnel over a hundred years ago, engineers have been designing to carry more freight across the Severn Estuary year on year. Having undergone strengthening, the M48 Severn Bridge was at full capacity once again and the new Second Severn Crossing was commissioned.

After four years of construction, the £330million M4 Second Severn Crossing (SSC) finally opened on 5th June 1996. The crossing links England and Wales across the Severn Estuary.

Spanning a total length of 5,128m and comprising two approach viaducts, the central cable-stayed Shoots Bridge is 848m long and has a main span of 456m. The central pylons are 137m tall and the cables are arranged in two planes of 120 cables in semi-harp formation. There are three lanes plus a hard shoulder in both directions.

2 Background information

Prior to construction of the SSC, the Severn Bridge, located 8km upstream, carried the M4 motorway. The M4 route is now carried by the SSC. A survey in 1984 showed traffic in both directions to be 35,000AADT (Annual Average Daily Traffic) - just more than the flow in and out of Bath every day. Seasonal and weekend fluctuations saw traffic peak at 48% above AADT. This survey also revealed that 75% of traffic from the bridge either started or ended in the Cardiff/Newport/Swansea direction, whilst 25% went to Chepstow/Wye Valley, further justifying this new build.

In the event of total closure, the alternative route across the Severn Estuary (without the SSC) is an 80km diversion. A second crossing would, therefore, provide back-up in case of closure of either crossing. The delays and diversions would be unacceptable because of bottlenecks at the approach roads and the impact on local economy.

The SSC could actually have taken the form of a tunnel. Ground conditions for a bored tunnel along the eventual route were favourable, and more importantly, proven by the railway tunnel under the Severn Estuary.

A tunnel would be preferable to a bridge here because:

- Minimal visual impact on landscape
- Less affected by adverse weather
- Another tunnel can be built adjacent if required. This is much more difficult to achieve with a bridge.

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Of the three points, a bridge can only mitigate that of the weather. Structural stiffness would limit deflections, and anti-corrosion outer layers can be added to bridge elements. The final decision for a bridge instead of a tunnel is presumably down to cost – both capital and operational, and fire safety.

The approach viaducts are mentioned in parts of this section because they have significant aesthetic features which help contextualise those of the Shoots Bridge.

Commenting on bridge design, Leonhardt preaches that “quality and beauty must be united”[2]. In this, he postulates ten criteria for satisfactory design of bridge aesthetics which form the outline of the analysis in this section.

It is important to note that this bridge is not open for public pedestrian access. Not only does this eliminate need for vandalism protection (and as a result, this topic is not discussed in this paper), it also means that details visible in the longitudinal direction are only seen by vehicles travelling at high speed. Any minute features near the centre of the bridge are seen only by boat (non-commercial as freight ships dock downstream) or riverbank viewers with telescopic equipment.

The SSC’s location allows plenty of space and potential for expression of character. Both shores are quite flat and consist of small towns and undeveloped fields. The SSC’s low height/length ratio appearance allow for a careful merge into these surroundings. Its architecture, however, appears modest and does not make full use of the available potential.

The SSC’s aesthetics lack any obvious anthropometric form. However, consider its location. The crossing links Bristol and (eventually) Cardiff – two strong industrial cities in the UK. The SSC takes pride of place over a fierce current and the second largest tidal range in the world. Such minimalist architecture shows mankind and engineering overcoming natural forces in a simple, efficient way.

A curved plan layout can be seen from the shores. Together with the parabolic suspension cables from the adjacent Severn Bridge, the linear elements of the SSC (especially the pylons and viaduct deck) would have harmonised better had these features been curved in elevation.

3 Aesthetics

Careful siting for the new bridge was required. Design considerations needed to include:

- Aerodynamic interaction between old and new bridges. Too close together and downstream vortices could cause unacceptable movements.
- Foundation interaction between the two bridges. Possibility of heave or settlement.
- Length of new approach motorway network to be built.

2010 was taken as the design year for vehicular traffic, only 15 years after the opening of the bridge [1]. A somewhat proximate date, traffic was predicted at 55,000AADT. Figures from 2007 measured an AADT of 64,885, representing nearly two-fold growth since 1984 and 10,000AADT above prediction. Heavy Goods Vehicles (HGVs) constitute 11.2% of traffic across the SSC in 2007, affirming the economy’s reliance on this link. If traffic continues to grow at a high rate, then major strengthening works may be required to this bridge.
direct and reinforce the navigation channel under the main span.

There are secondary cables attached to the main cables on the Shoots Bridge. These were used to prevent unwanted oscillations of the main cables and were painted black to appear inconspicuous. This ensures structural form is not confused and shows that the deck is only carried by the main stay cables. Both primary and secondary cables are visible in Figure 4: Pylon on Shoots Bridge.

All components of the Shoots Bridge feel adequately slender and proportioned (see Figure 1: View of the Shoots Bridge on the Second Severn Crossing). The pylons are not too thick or tall to feel overpowering and makes the Shoots Bridge feel integral to the whole SSC. If anything, the stay cables are too thin, but the compromise is struck from its expression of simple structural form.

The stay cables are arranged in a semi-harp pattern. This offers a good compromise between structural efficiency of fan-formation and aesthetics of harp formation. However, as the bridge is always viewed either longitudinally or obliquely, the cables will arouse mental disquiet from the non-regular weaving pattern of the cable planes overlapping, as opposed to harp formations which form similar trapeziums when overlapped. (See Figure 4: Pylon on Shoots Bridge)

The pylons are tapered to add visual weight to the base. (See Figure 4: Pylon on Shoots Bridge) This prevents the tower appearing top-heavy. Also, the top cross member of the pylons deepen at the connections.

Whilst not entirely attractive, its purpose is clearly to increase section area for flexure.

The cable-stayed structure adds interest and variety to break rhythm and prevent monotony from the approach viaducts. To distinguish the two sections, there is a complete change in structural form. Aside from the obvious cable-supported deck, there is a clear variation in deck form; changing from twin box girder to truss (see Figure 5: Interface between viaduct and Shoots Bridge). Such detail is difficult, but possible to discern from the shores as in Figure 1: View of the Shoots Bridge on the Second Severn Crossing.

The change in structure is not limited to shape, but accented in colour too. The viaducts are creamy in colour and the Shoots Bridge (except the pylons which has the same colour as the viaducts) is painted pale green (see Figure 5: Interface between viaduct and Shoots Bridge ). Both colours stand out well under cloudy or sunny conditions. Even under dark storm clouds, the pale green appears to self-illuminate the bridge.

The base of the piers have suffered discolouration from algae. This detracts heavily from the overall aesthetics of the SSC. (See Figure 5: Interface between viaduct and Shoots Bridge).

Viaducts, piers and pylons are of smooth, matte finish. When viewed alongside the outlying bedrock at low tide, this gives a conflict of textures. The bedrock lies only on the English (and not Welsh) side and is not seen at high tides. When this is covered by high water, the smooth finish of the entire SSC blends seamlessly into the panorama of water, fields and sky.

4 Loading

The SSC was designed using BD 37/88 loadings [3], published in 1988. This code was updated in 2001 and there are several notable differences between revisions. Using BD 37/01 [4] gives results acceptable to today’s practice. This would then support judgments to perform strengthening or any other future works.

This paper will analyse the SSC against both old and new codes, aiming to analyse any weaknesses in the crossing according to today’s design, if any.
Without information to suggest otherwise, the SSC must be assumed to have a design life 120 years under BD 37/88.

### Table 1: Loads considered for loaded width = 26.6m

<table>
<thead>
<tr>
<th>Load Type</th>
<th>BD 37/88 Value</th>
<th>BD 37/01 Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dead</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bridge Deck (width=36m)</td>
<td>260(-/10) kN/m</td>
<td>260(-/10) kN/m</td>
</tr>
<tr>
<td>Live</td>
<td></td>
<td></td>
</tr>
<tr>
<td>HA UDL Loading</td>
<td>33 kN/m</td>
<td>85.8 kN/m</td>
</tr>
<tr>
<td>HA KEL Loading</td>
<td>440 kN</td>
<td>672 kN</td>
</tr>
<tr>
<td>HB Loading (45 units)</td>
<td>4x450 kN</td>
<td>4 x 450 kN</td>
</tr>
<tr>
<td>Braking loads</td>
<td>8kN/m plus 200kN</td>
<td>8kN/m plus 250kN</td>
</tr>
<tr>
<td>Accidental wheel load</td>
<td>250 kN</td>
<td>300 kN</td>
</tr>
</tbody>
</table>
| Parapet collision          | Variable       | 500 kN (trans.)
| Wind                       | 7.87 kN/m (trans.) 19.4 kN/m (ver.) | 3.56 kN/m (trans.) 8.79 kN/m (ver.) |
| Superimposed               |                |                |
| Road surfacing             | 83.8 (+/- 28) kN/m | 83.8 (+/-28) kN/m |
| Road furniture             | 39.9 kN/m      | 39.9 kN/m      |

These values are discussed further in respective sub-headings.

### 4.1 Dead loads

The estimate for wire diameter is based on observations from photos (see Figure 15: Jacking the stay cables), a quoted 0.6 inches (15.2mm) from [5], and the author’s engineering judgement.

Based on this information and the longest cable length of 253m, each cable comprises 75 strands of 7 wires. Each wire is 15mm diameter. Approximate calculations for the weight of cables thus yield 8kN. Their weight is, therefore, negligible compared to the deck.

### 4.2 Live loads

#### 4.2.1 HA load

The most significant change to BD 37 is the HA uniformly distributed load (UDL) live load and the partial lane factors. The superseded code states a value of 9kN/m per notional lane and 0.33 for the partial load factor. In the new code, loaded lengths past 1600m require agreement between consultant and client to establish a HA UDL live load. In the absence of this information, the unit load per notional lane is calculated using the formula:

\[ W_{HA} (kN/m) = 36 \times (1/L)^{0.1} \]  

where L represents the loaded length. Here, L = 5,128m since this must also include the approach viaducts. Two notional lanes are loaded with full HA, while the remaining notional lanes are loaded with 0.6HA.

The widths of the notional lanes change from 3.8m to 3.65m between codes. Accordingly, there are 7No. lanes in BD 37/88 and 8No. lanes in BD37/01.

Pedestrian live load is not applicable because the SSC does not allow public pedestrian access. Servicing loads are negligible compared to HA and HB loads.

#### 4.2.2 Centrifugal loading

The Shoots Bridge is straight along its entire length, and so centrifugal loading is negligible. The approach viaducts, however, would have been designed for this.

### 4.3 Superimposed dead load

Road surfacing is assumed to have a unit weight of 21kN/m³ and a thickness of 150mm (+/-50mm).

Road furniture is assumed to take 1.5kN/m² across the entire width and length. This scenario is conservative and in reality cannot be combined with full HA loading, as an entire deck of road furniture means vehicles cannot use the crossing.

### 4.4 Ship impact

A government requirement is to design against impact from vessels of 6,000DWT (Dead Weight Tonnage). The force, in MN and from [6], is calculated from:

\[ F_{ship} = 0.44 \sqrt{DWT} \]  

\[ F_{ship} = 34.1 \text{ MN} \]

This value is equivalent to 70 times the transverse loading on the parapet from vehicular collision. The other transverse loading is due to wind, but when considering the loading on the Shoots Bridge, this is equivalent to 7.87kN/m x 848m = 6.7MN. Therefore, ship impact is the dominant transverse force.

### 4.5 Natural Frequency

The natural frequency of a bridge is especially important for both collapse and serviceability analysis. If the natural frequency is too low (<5Hz), then the design must counteract resonance due to wind, or vehicles may lose road traction from bridge motion. Too high (>75Hz) and physiological problems can be experienced by users – for example motion sickness.

Assuming the composite section is transformed into steel, infinite rigidity at the backspans and uniform
cross section along the entire length, the central span can be crudely modelled as a 456m beam built in at both ends. These assumptions are discussed later.

The bridge’s vertical fundamental angular frequency is calculated as [7]:

\[ \omega_n = (\beta_n l)^2 \sqrt{\frac{EI}{mL^4}} \]  

(3)

Where E = 210 x 10^9 N/m^2, I = 1.058m^4, M = 39.2 x 10^6 kg/m, L = 456m. For n=1, \((\beta_n)^2 = 22.37\) assuming both ends are clamped. This gives \(\omega_1 = 0.254\) rad.s^{-1} and therefore natural frequency \(f_0 = 2\pi \omega_1 = 1.67Hz\). This is below the optimum 5Hz for the 1st vertical mode. This proved to be critical as excessive excitations due to wind were recorded soon after opening. Details of this are mentioned later.

The assumptions are based on a uniform beam of the smallest known depth (and therefore lowest I and M values in (3)). In reality, the deck deepens at the pylons and pier supports, thus increasing the stiffness and the natural frequency too.

The condition of infinite rigidity at the backspans is impossible in reality and will therefore create a more flexibility condition. This assumption therefore overestimates the natural frequency. Additionally, the piers add further stiffness, thus altering the mode of vibrations for the SSC. Their fixity means that the bridge should only resonate at harmonics of the fundamental.

The stay cables effectively restrain the deck from downward vertical oscillations. This change of support conditions would introduce new terms (be it linear or non-linear) into the equation. For greater accuracy of natural frequency analysis, further iterations using computer analysis would be required.

4.6 Wind

Both standards are designed to a 120 year return value of wind speed. The current BD 37/01 states a maximum unfactored design gust speed of 35m/s. This is a dramatic reduction compared to the calculated design gust speed of 52m/s in BD 37/88.

The superseded BD 37/88 is a crude calculation involving only a few factors; mean hourly wind speed, height of bridge, loaded length of bridge, gust factor, tunnelling factor (if applicable).

The current BD 37/01, in addition to the above factors, include: orientation of bridge, terrain friction, fetch of wind and shielding from buildings. This upper cap seems bizarre at first, but when multiplied by a ULS factor of 1.4, the resultant 49m/s is faster than most recorded gusts from the Great Storm in 1987. According to [8] storms of this magnitude have a return period of 1 in 200 years. It therefore follows that this upper cap is enough to protect bridges against collapse and that the BD 37/88 was conservative with its design load.

The transverse force on a bridge is calculated at:

\[ P_t = 0.613v_d^2A_1C_D \]  

(4)

Where \(v_d\) is the design gust speed, \(A_1\) is the solid horizontal projected area and \(C_D\) is the drag coefficient. \(C_D > 1.3\) for a highways bridge.

It is widely known that wind speeds are higher at long-span bridges than on-shore. Predictions from [9] state up to 24% greater speeds on merchant ships. For even taller structures like the SSC, this factor is likely to increase.

The SSC was installed porous 3m high barriers to defend traffic against wind. As shown in (4), for a given area, the horizontal force experienced is proportional to the square of the speed. When cars cross bridge without windshieding, a 24% increase in speed relates to a 54% increase in force. The 50% porosity (and hence 50% windshielding) of the barriers reduces this force to below experienced forces at the unprotected approach roads. Therefore, if cars could reach the SSC, they could cross it.

4.7 Temperature

The design guidelines for temperature do not vary between old and new versions of BD 37.

The River Severn area has a mean annual of 10-11°C. [13] The SSC has an assumed design-life of 120 years, and so design temperatures should reflect this return period.

4.7.1 Effective temperature change

The deck, due to its classification owing to material properties and location, is expected to operate between -12°C and +38°C; or -22.5°C below mean and +27.5°C above mean. Should the movement joints become immobilised, then the bridge should be designed against apparent stress.

Taking the more onerous temperature change, using mean temperature as datum, and \(12 x 10^{-6}\) as the coefficient of thermal expansion for steel and concrete Strain, deflection and apparent stress can be calculated as:

\[ \varepsilon = 330 x 10^{-6} \]  

(5)

\[ \delta = 280mm \]  

(6)

\[ \sigma_{apparent} = 63.9\ N/mm^2 \]  

(7)

The deflection calculated in equation (6) lies within the +/- 250mm movement limit imposed by the
joint, as the value must be halved to represent movement at one end.

4.7.2 Temperature difference

Differential heating, for example at dawn or dusk, can generate bending moments from the heating profiles shown in Figure 7: Temperature profile across deck.

In deep sections such as the Shoots Bridge, the lever arm is large, and the greater the temperature difference, the greater the force. The bridge should be designed with enough stiffness and longitudinal shear capacity to withstand these forces.

4.7.3 Closure due to cold weather

On 6th and 7th February 2009, the SSC was closed because ice sheets formed on the cables and fell onto moving traffic. The air temperature was well above minimum design parameters. It is not known whether the engineers had acknowledged such an event, as it is very rare in the UK [13].

Protection from falling ice would require total coverage of the deck, as the ice could strike any point. As this is not already installed, a retro-fit would be required. This is uneconomical for two reasons: Firstly, closure of the bridge would be necessary for final installation and there would be loss of intake, and cost towards closure of the motorway. Secondly, assuming this was not considered by engineers, the infrequency of such an event may not merit installation at all, if the solution is to close the bridge whenever it occurs.

In engineering design terms, further modelling will be required and stiffening where necessary, as the area for wind to act upon will be greatly increased, and any coverage will require fire testing. With cars travelling through at speeds, pistoning effects will occur and thus spreading smoke.

5 Strength

5.1 Deck

The 36.6m wide deck is a composite section of 2.15m deep steel longitudinal girders supporting a reinforced concrete slab varying between 200mm and 470mm. There are transverse members arranged in truss formation at 3.6m spacing and stay cable anchorages are spaced at approximately 7.3m.

The composite section construction provides a good strength/weight ratio. Assuming a perfect section transformation, an entirely concrete deck would weigh 373kN/m, about 43% heavier. There would be further fabrication issues as large volumes of concrete require excessive amounts of time to cure. Combined with the amount of reinforcement needed (owing to concrete’s poor tensile strength), the use of concrete sections is unfeasible. Instead, steel sections can be rolled and bolted together quickly.

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Figure 7: Temperature profile across deck

Truss action of the transverse members gives stiffness across the deck’s width whilst maintaining the lightweight shape. The diagonal members were arranged to provide a wide, central rectangular space. Here, a suspended service train running along the entire length of the SSC could access the soffit. This is discussed in section 8.

5.1.1 Flexure of deck

Although crude, flexure can be performed by assuming: the composite section to be transformed into one material - in this case into steel; uniform along the length of the span; built-in beam condition with the continuity of the deck either side of the stay cables providing infinite rigidity.

Figure 9: Bending moments in deck

The condition in Figure 9: Bending moments in deck represents a simplified ‘worst-case’ scenario using BD 37/88. Here, HA and HB loading are combined with the windiest conditions. Moments are calculated as M_A = -3,770 kNm, M_B = -3,020 kN, M_C =
+2950kN.m. In terms of stress, the maximum hogging stress is -1.39N/mm² and the maximum sagging stress is +5.74N/mm². Using similar calculations, stresses under BD 37/01 are -1.57N/mm² and +6.98N/mm². Of course, the flaw with this calculation is that stay cables are assumed inextensible. This extension would increase sagging moment values, and may cause some areas of hogging to go into sagging.

At such low stresses, it would seem that the girders are deep to resist redundancy and other types of stresses; e.g. shear, and to provide stiffness to the structure.

5.1.2 Compression in deck

The stay cables are arranged in semi-harp formation, therefore the applied compressive force will not be uniform in the longitudinal direction. The shallowest cables were at 24° to horizontal and the steepest were 76°.

![Figure 10: Compression in deck from cables.](image)

The cables were prestressed to support dead and superimposed dead loads. Using loadings from table 1 and the known geometries, the compressive force is estimated to be:

\[
F_{\text{compressive}} = \sum \frac{W_{\text{DL}} + W_{\text{SDL}}}{\tan \theta} (8)
\]

\[
F_{\text{compressive}} = 7.3(260 + 83.8 + 39.9) / \tan \theta
\]

\[
F_{\text{compressive}} = 133\text{MN}
\]

Where \( \theta \) = Horizontal angle between cable and deck.

The theoretical buckling load is calculated to be 60,000MN - much greater than (8). The most significant factor is the short effective length, as provide restraint against buckling.

5.2 Pylons

The pylons are built as a portal frame with rectangular legs providing resistance to flexure along the two principal axes, as well vertical normal stresses. The reinforced concrete legs are hollow inside and range in size from 10.2m x 4.0m to 5.4m x 4.0m, which provide a large lever arm to resist bending.

The foundations of the pylon rest on bedrock with no substructural link. Any overtopping moments are resisted by the weight of the footing.

The areas of cable anchorage are the highest stressed and susceptible to failure given the cyclic nature of loading. Plug failure through shear would dictate the thickness of the wall, as well as the amount of axial prestress/post-tensioning to limit flexural failure.

5.3 Backspan piers

The piers on the backspans use their self-weight to provide tension forces to balance forces from the central span. Indeed, they are also cable to carry compression forces under different load regimes too.

5.4 Redundancy

Part of the design criteria was to have enough redundancy to withstand removal of up to two stay cables. Under these conditions and using a similar method from section 5.1.1, the maximum moment is estimated to be +23.0N/mm². This is well within the typical design stress of steel. Further checks against other failure modes are required, but for such a complex system, computer analysis will provided the most accurate analyses.

6 Foundations and Geotechnics

The ground beneath the Shoots Bridge is mainly sandstone of effective friction angle \( \Phi' = 30^\circ \) and mudrock \( \Phi' = 26^\circ \) [14] See Figure 12: Geological section through site.

There are three inactive faults across the span of the SSC. These pose no threat of earthquake, transverse or longitudinal movement, but would have meant some piers in the approach viaducts were piled owing to weaker soil strata.

The exposed surface bedrock across most of the Severn Estuary kept foundation design relatively simple. The pylon uses spread footings of length 53m, width 13m and height 11m which simply rest on the surface. (see Figure 13: Lifting a caisson into place). The backspans uses the same design, but with added 2m deep hollow tube shear keys at the base to account for less down force due to uplift from the central span.

With no permanent fixture to the soil it rests on, the footings need to be checked against transverse movements. The most critical movements will arise through ship impact.
Resistance against ship impact is assessed using standard laboratory techniques for shear box testing. Given its dimensions, a volume of 7,180m³ and a density of 24kN/m³ is assumed, giving the footing a self weight of 172,300kN.

\[
\tan \theta = \frac{\tau_{\text{max}}}{\sigma'_{N}}
\]

\[
\tau_{\text{max}} \Rightarrow \frac{F_{\text{max}}}{A} = \frac{\sigma'_{N}}{\tan \theta} = \frac{172,300kN}{A} \tan 30^\circ
\]

\[
F_{\text{max}} = 99.5MN
\]

\[
F_{\text{max}} < F_{\text{ship}}
\]

The above calculation shows that the footing’s self-weight alone is able to withstand impact of a ship. Its enormous dimensions are therefore designed for bearing pressure from the pylon’s large vertical loads.

7 Construction

By far the greatest challenge in the Shoots Bridge’s construction is the large diurnal tidal range. At 14m, this range meant that any construction causeways were accessible for only a few hours each day. To optimally utilise any high water times, transportation by barge would be required.

An on-shore construction site with a roll-on/roll-off (ro-ro) ramp was built. This allowed crawler-transporters, similar to those used for manoeuvring space shuttles, to move components to a flat-bottomed barge and float them out to their resting place.

Current speeds in the Severn Estuary can be as high as 5m/s. The barges were fitted with laser-operated dynamic positioning system (DPS) which, when connected to the propulsion units, allow the barge to maintain a position accurate to 500mm against the current – an impressive 1% of its overall length.

7.1 Foundations

The high tidal range called for jack-up barges to be used for footing construction. Each precast, hollow 2,000t caisson was floated into place at high tide and then the legs of the jack-up barge were lowered. The caissons were then suspended until low tide exposed the surface bedrock thus allowing caissons to be lowered into place. After lowering, caissons were then infilled with mass concrete.

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This technique relies heavily on spring high tides, as opposed to neap high tides, as bedrock would be exposed in many areas thus grounding the barge. Operations would, therefore, only have taken place not only over a few hours per day, but few days per month too.

The lifting crane supported the caissons at four points at its base. Combined with its self-weight, there would have been areas of high stress concentration around these points thus requiring strong reinforcement. It is expected that some computer analysis was performed to ensure safe bearing here.

Figure 12: Geological section through site

Figure 13: Lifting a caisson into place
7.2 Pylons

The reinforcement cage of the hollow legs of the 137m tall pylon and both cross beams were prefabricated on-shore. Concrete for the prestressed legs was cast in-situ using climbing formwork.

Prefabricating one component and casting the other in-situ both allows for full moment connection whilst easing constructability, compared to casting both in-situ. Full moment resistance is needed here as portal frame action is necessary to resist transverse loading.

During erection, the upper beam had to temporarily rest on the lower beam as the pylons had not been built (see Figure 14: Lifting of the upper cross beam into position). Explicit evidence of whether this was intended is not available, but facts points towards a planned execution.

Justification for its intended execution includes:

- The crane from the jack-up barge was not tall enough and a different one was required for final lifting. This is easily foreseeable.
- The upper beam was ready to be erected four months after the lower beam. During this time over 60m length of pylon leg length would need to have been built. This timeframe is unreasonably short.
- Swift turnover of the upper beam at the on-shore site provides space to store other components of the bridge.

Weighing at 9,000kN, the moments and shears induced by the upper beam resting on the lower beam would have been small compared to operational loads.

7.3 Deck

The composite deck was prefabricated in Italy in 7m units. After their construction, they were shipped to Avonmouth Docks and later transferred to the on-shore construction site by road. As there was only one ro-ro ramp, delivering goods directly to the offshore would have caused congestion here, as the ramp was in constant use to lower other bridge components.

To ensure perfect fit at interfaces, adjacent deck units were match-cast against each other. Such a method minimises 'lack-of-fit' stresses and eliminates any time wasted from rectifying poor connections between deck units.

The deck was erected using balanced cantilever construction. This is the standard method used for cable-stayed bridges, as deck units can be easily made to match cable spacing, and each cable can be anchored individually.

The reinforced concrete of the composite deck was cast on land. After hoisting into place, splice plates were bolted onto longitudinal girders and the concrete deck was stitched with concrete cast in-situ. Without need to wait for large areas of concrete to cure, this technique allowed each completed deck unit to serve as an ever-growing platform for construction crew.

7.4 Cables

Cranes attached to the legs of the pylons were able to lift one end of each stay cable to its anchor point. This end was fixed in before stressing was performed at the deck end.

To achieve uniform stress in cables, and thus eliminate out-of-plane bending, strands within the cables had to be jacked individually. First, a strand was loaded with a hydraulic jack to a reference stress – approximately 60% of its final stress. The other strands within this cable and the paired cable on the side were then jacked to this same stress. This continued until all cables along the bridge were of 60% prestress. (See Figure 15: Jacking the stay cables.)

After monitoring movements due to steel relaxation, the measured alignment of the deck would be compared to theoretical values. This stage would have been important as jacking would have taken place as each deck unit was lifted. Without the stiffness from a fully-installed deck, transverse deflections from wind would be greater and more critical.

8 Durability and serviceability

Serviceability is provided by the suspended maintenance gantry. Using mobile platforms, the entire
length’s soffit can be accessed and inspected. Without this, engineers would need to travel to the base of the piers by boat and climb up. This method is hazardous and clearly unfavourable.

All exposed steel girders had primer coats applied in the prefabrication factory and a final polyurethane coat applied at the on-shore construction yard. With good cohesion, elastic and hardiness properties, the outermost coat will provide good durability whilst being constantly deformed under all types of load. This is especially important during winter months when the UK road network is covered with de-icing salts. When dissolved, the solution will infiltrate any crack and cause potential harm.

All stay cables are coated in numerous HDPE sheaths with wax grouted along its length. Over the 120 year design life, this level of protection is more than sufficient.

9 Future changes

At the time of writing, there are no planned changes. It is possible that the bridge will require further strengthening or retro-fitting during its lifetime, as traffic levels today are already above 2010 design levels and recent unforeseen weather halted traffic.

Further, the update of BD 37 reflects on large changes. It is often the case that design loads are lower in later versions as more knowledge and confidence is acquired from precedents. However, the most significant change is the increase of HA loading to reflect the growing size of modern-day lorries. Increases changes to the lanes factors also represent their large presence, which will only increase with time.

10 Conclusion

This paper has analysed various aspects of the Shoots Bridge against differing design codes. Hand calculations have shown the bridge to be adequate against worst-case loading even to today’s standards, although further computer simulation is recommended for stronger evidence of adequacy.

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References