A Critical Analysis of the Humber Bridge

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Abstract: This conference paper attempts to examine The Humber Bridge, and the techniques utilised in its construction. In analysing the current bridge this paper will portray the author’s opinions with respect to the aesthetics, refinement and suitability of the bridge.

Keywords: Humber Bridge Suspension Bridge Freedman Fox & Partners

1. Introduction:

The Humber Bridge was opened on 24th June 1981, and marked the completion of centuries of campaigning and nine years of construction. The Bridge includes a main span of 1,410m, making it the longest bridge in the world for 17 years. Designed by Freedman Fox and Partners it has many similarities to their earlier Severn, Bosporus and Forth Bridges. The bridge carries two pedestrian walkways and two lanes of traffic in each direction on the A15 between Hessle on the northern bank and Barton on the southern bank of the River Humber.

Figure 1: Map showing the bridge location

2. History:

The Humber has presented a significant obstacle to North/South movement of goods and persons since Roman times. A fixed crossing was always considered desirable, however, this was not seriously proposed until 1872 when commercial concerns prompted the need for a more direct rail route to Hull. Although the route was never built for political reasons, it proposed a tunnel section under the Humber. In 1928 Sir Ralf Freeman was appointed to investigate the possibility of a bridge over the Humber. He initially proposed a multi-span truss bridge comprising of a main span over the navigation channel with shorter spans either side, however, by 1935 this plan had adapted to that of a suspension bridge on the same site and with the same dimensions as the bridge we see today. Initially, however, his designs weren’t commissioned, firstly due to the 1930 economic crisis and then the Second World War. Following numerous feasibility reports the government finally agreed to lend 75% of the finance required for the project in May 1971, with the remaining 25% being raised from private sector investments. Construction began in 1972.

3. Basic Design

The eventual site for the bridge was selected due to a narrowing of the river channel, which whilst reducing the span required did mean that foundering conditions weren’t ideal, resulting in large expensive foundations. I would have considered choosing an alternative site with more favourable foundering conditions, even if it meant a longer span. However, Freedman Fox’s reasoning for choosing the narrowest point of the river is understandable as even this meant building a longer span than had ever been built before.

A major design requirement of the bridge was that it would not impede shipping on the Humber, which is economically vital to the region. In addition, the river has a soft alluvial base which is constantly shifting such that the shipping channels frequently move. Initially a truss bridge was proposed, but it was felt that the piers required would impede shipping. In 1935 the only real alternative over such a long span was seen as a suspension bridge.

Figure 2: Artists impression of Proposed Truss Bridge

As construction did not commence until 1972, I would have considered the use of a cable stayed bridge on this site, a number of advantages of which are outlined in this paper. Suspension bridges are the preferred for creating a large central span, however, were a cable stayed bridge to be used the approach spans could be increased, reducing the central span.
4. Aesthetics

From the outset freedman Fox were conscious that a bridge of this scale would have a visual impact on the locality, consideration of which would be vital to constructing a successful structure.

Figure 3: The Humber Bridge viewed from the Northern Bank

Suspension bridges are inherently beautiful due their slender proportions and coherent structural form. In the case of the Humber Bridge it’s instantly obvious that the main structural rigidity is provided through the catenary main cables. The load path from and point on the deck can be very easily followed through the deck, into hanger cables, into the main cables and finally to the foundations through the main towers. This ability to understand the structure is very pleasing aesthetically, and also imparts confidence in the design.

The main towers on the bridge are 6x6m square at their base, but the three sides away from the second tower taper such that the top is only 4.5x4m. This is vitally important to the aesthetics as the towers could otherwise appear thicker at the top, which could be detrimental to the simplistic structural beauty on which the structure relies.

In the design of the Humber Bridge the Deck only acts to spread localised loads over the few nearest hangers and is relatively slender as a result. The depth to length ratio of the main span is 1:310 which is very shallow compared to 1:170 in the Golden Gate Bridge, which is commonly regarded as the most beautiful bridge in the world. This is also largely due to the use of an aerodynamic section, reducing lateral wind loads, and therefore the stiffness required. This could look out of place but the apparent strength of the main cables compensate for the slender nature of the deck, keeping the bridge in proportion.

The bridge is situated on the edge of the industrial port of hull and was constructed to allow the congested northern bank to expand onto the southern bank and create a new industrial area. Consequently, the bridge was designed with this environment in mind and has large unpainted expanses of concrete and similarly coloured painted steel. However, since opening local industry has declined such that the industrialisation of the south bank never occurred. The result is that the vast concrete towers and abutments do look out of place now they are surrounded by open countryside. Had this been anticipated at the time of construction I would suggest the design would have been adapted to embrace the local environment, for examples the cables and towers could be painted to blend into the sky, as seen on the second Seven Crossing.

In general any problems with order in the design of suspension bridges are related to a truss under the deck. The Humber Bridge’s deck is aerodynamically constructed from steel box sections which give it a constant section, allow the eye to flow over the deck.

However, the Humber Bridge has inclined hanger cables which can appear to cross each other when the bride is viewed at oblique angles. This is not normally a problem encountered in suspension bridge design, and is usually only seen in cable stayed bridges, which must by definition have inclined cables. However, the cables don’t appear to overlap from on the bridge and the hanger cables are sufficiently slender that they are not significantly visible from the riverbanks. Although not a major aesthetic problem this problem could be completely overcome by the use of vertical cables as are used on modern suspension bridges.

Figure 4: Oblique view of Hanger Cables

One major aesthetic flaw in the Humber Bridge is that due to founding conditions it is not sited centrally on the river, such that one tower is on a bank and the other is a third of the way across the river. In addition to this the side spans aren’t equal with the northern span being 250m longer. The result is that the bridge appears unbalanced.

Currently lighting on the bridge is as it was when first designed. Aside from the regulation lighting on the towers and under the deck for Planes and boats respectively the only lights on the structures are the street lamps as shown in Figure. Recently a new lighting system was trialled on the bridge involving the use of spot lights which project columns of light into the sky. They were met with mixed reviews by the public and have since been removed. It’s my feeling that the more simplistic lighting scheme was more successful it complements the simplistic silhouette of the bridges structural form, upon which the aesthetics rely.

Figure 5: The Humber Bridge at Night
5. Construction and Design Specifics

Although the final construction of the bridge is considered an engineering success its construction was not. The project was initially anticipated to last five years, at a cost of £28m, however technical difficulties, labour relation problems and unusually poor weather the bridge took 9 years to complete at a final cost of £98m, although much of this cost overrun was due to unprecedentedly high inflation rates.

The construction of the bridge was undertaken in five main phases, using two main contractors. The Substructure (foundations and main towers) was constructed by John Howard & Company. The contract for the superstructure was won by British Bridge Builders Ltd, a partnership combining, Sir William Arrol & Co Ltd, Cleveland Bridge and Engineering Co Ltd and Redpath Dorman Long.

Using four different main contractors, could be problematic should any unforeseen delays occur. This is particularly relevant given the very progressive nature of the construction, meaning the next stage can’t start until the previous has been completed. For example delays in construction of one phase could result in another contractor being onsite but unable to start the next phase of construction, meaning costs can quickly escalate. Furthermore, any subsequent proceedings for compensation are often complex as no single party is responsible for delivering the project on time.

5.1 Foundations

There are four main foundations inherent in the design of the structure, which are the two main towers and an anchorage point at each end, as below:

![Figure 6: Diagram of main Foundation locations](image)

As shown above suspension bridges require large anchorages at their ends which are expensive, especially given the poor founding conditions on the south bank. A symmetrical cable stayed bridge would require no such anchorage points as the horizontal forces are resolved within the structure as shown below:

![Figure 7: Structural Diagram of Cable stayed Bridge](image)

The foundations on the Hessle (north) bank were relatively simple as a bed of chalk lies close to the surface covered by a relatively thin layer of tough boulder clay. The advantage is that although the foundations must still be large they can be relatively shallow, which reduces the excavation and construction required.

The Hessle anchorage is built of reinforced concrete sections constructed in a 20m excavation, forming two chambers into which the main cables spray out. In total the anchorage is 39m wide, 65.5m long and 36m high.

![Figure 8: Diagram of the Barton (Left) and Hessle Anchorages](image)

The Hessle tower is founded on a 44m x 16m reinforced concrete slab 11.5 meters high sunk 8m into the ground. Although the foundation was set into the river bank the close proximity to water required that sheet piles were driven between the excavation and the river and backfilled with gravel to act as a cofferdam. However water was still seen to enter the excavation through fissures in the chalk and required constant pumping. The fissures were sealed using cement grout as soon as the pier was completed to prevent settlement.

Foundations on the Barton (south) bank were considerably more complex, and expensive to construct. The Chalk used for founding on the north back had been eroded away and replaced with soft alluvial bolder clay to a depth of 30m.

The Barton anchorage is designed to look to similar to that in Hessle, however, it’s of a cellular construction of diaphragm walls filled with sand and water. In total its 72m long, 42m wide and penetrates 35m below ground to found in the same chalk used on the Hessle bank.

The Barton tower is inherently different to that in Hessle due to its location 500m from the south bank, and proved to be the first major challenge of the project. The construction...
comprises a reinforced concrete platform supported on twin hollow circular caissons, each 24m in diameter. The piers were constructed by first building a ‘T-shaped’ access jetty.

The variability of river bed and its shifting nature specified the construction of a sand island enclosed by sheet piles, as the more normal watertight cofferdam approach was unsuitable. However, this approach was also seen to be inadequate with significant scouring disturbing the sheet piles. This was rectified through the addition of 12,000 tonnes of sand bags and chalk around the base of the sheet piles.

Steel caisson shoes, complete which cutting edges were constructed in situ and subsequent concrete sections were formed above in 3m sections. The caissons were allowed to sink under their own weight while material was excavated from flooded inner cells. Friction was reduced by pumping bentonite through pipes cast in the concrete walls into a void which created a skin between the caisson and riverbed.

During construction of the west caisson it penetrated an unanticipated layer of clay, with high pore water pressures, which washed away the bentonite skin. This proved to be detrimental during the final stages of sinking, with an extra 4,000 tonnes of concrete having to be added to each caisson to help them sink. However, even this was inadequate and a further 6,000 tonnes of steel pellets were added to sink the caissons to their required depth.

The two anchorages were carefully constructed such that the load under the base of the anchorage would be constant to prevent large initial settlements. In essence this meant constructing the majority of the anchorage, then tensioning the main cables, and then adding the final concrete ballast as the deck was constructed.

5.2 Main Towers

The two main towers are hollow and constructed from slip formed steel reinforced concrete. Each pier supported two towers which would be linked by portal beams to aid stability. Each tower is 155m high, with the bases of 6m x 6m square tapering to 4.5m x 4.75m at the top. The formwork was mounted on an operating platform supported on 48 hydraulic jacks running on tubes cast into the concrete. In addition it was designed such that the sides could be moved in as the tower was climbed to create the required taper. This reduces the amount of expensive formwork required but also speeds up construction as forms don’t have to be changed and lifted into place. Each tower was heavily reinforced such that the main factor influencing the speed of construction was the rate at which the reinforcing steel could be completed.

![Figure 11: The Barton Pier Caisson under construction](image1)

Each caisson was divided into seven cells, with the soft alluvial river bed being removed from each and replaced with a concrete plug. A capping slab was then cast, onto which sat a concrete block which would distribute load from the towers onto the caissons. An important aspect of the construction was that the two caissons weren’t connected until the tower had been constructed to allow and differential settlements to occur.

Although many aspects of the caisson design were carefully planned the problem of an area of high water pressure would appear to be a foreseeable one. Although a suitable solution was found the delay did impact strongly on the remainder of the project, and so I would have thought that some provision could have been made for such a situation, even if this was only to build an extra period of time into the programme to allow for problems.

![Figure 12: Diagram of Barton Pier with Steel Reinforcement for Tower](image2)

Construction of the four horizontal portal beams immediately followed completion of the tower legs. Shuttering was mounted on a large steel truss spanning between the two towers and casting was carried out in
1m stages. Obviously the load from each layer of concrete would cause increased deflections to occur, so the entire shuttering was mounted on hydraulic jacks to compensate for this. The top three beams are each four meters deep, whereas the forth, which connects the towers below the deck is eight meters deep, accounting for the extra load at this point.

To attempt to overcome the Barton Pier delays each tower was constructed continuously by two teams each working in 12 hour shifts. As a result climb rates were higher than anticipated and averaged 100mm/hour for the Barton Tower.

The only difference between the two towers is that the Barton Tower contains much more steel reinforcement owing to the larger approach span.

5.3 Suspension Cables

The main suspension cables contain 14,948 parallel galvanised 5mm wire and total 700mm in diameter. The Hessle Approach Span contains an extra 800 wires to account for the extra load due to the steep angle of the cable to the anchorage.

The first stage in construction was to mount a crane on top of each tower. This was achieved by assembling the cranes at the base and then winching them up to the required location. The cranes were then used to lift the 45 tonne steel saddles required to hold the main cable into position. Unlike some suspension bridges the main cable is not anchored in position at the top of the tower. The reason for this is that any build up in load on the main span would cause a moment in the towers. This results in the need for much thicker towers, which can ruin the aesthetics of the bridge and be very costly. Instead any variations in load are carried by the large anchorages at the end of the bridge.

A wire footbridge was built on 8 steel cables which followed the path of the main cable but hung a meter below it. The locating of the first cable on each side was the only time during its construction that the river was closed to shipping, as a rope had to be laid along the river bed before being winched into position.

Above each walkway a tramway system was constructed and two spinning wheels were used in tandem to spin each cable. The wire arrived on site in 500kg coils, which were subsequently joined using specially designed unions formed in a hydraulic press.

Each wire was adjusted by remote winches until it hang at the same angle as the sag as a guide wire which was surveyed to ensure its correct location. The wires were then bound together into 37 strands.

As each strand was spun they were bound at frequent intervals along their length. The strands were positioned in a hexagonal formation which would eventually form the circular main cable. Heavy strapping was used to maintain the cable until the final wrapping could be applied.

Cable bands were then fixed to the cable at predetermined points ready to carry the hanger cables, which were mainly 62mm steel wire, pre-stretched to reduce deflections when adding the deck.

Figure 14: Main Cable and Walkway

Once the loads in the main cable increased to a predetermined level they were painted in a red lead paste and then wrapped with 3.5mm wire. The whole cable was then given a five coat paint treatment to ensure weather protection.

5.4 Deck

The deck of the bridge was constructed of 124, 18.1 m long, 140 tonne pre-assembled trapezoidal steel box sections. These were chosen over the classic stiffened truss as much less steel is required, reducing the dead weight of the structure, which allows a reduction in the size of cables and towers. In addition to this the steel box sections can be easily designed as the neutral aerofoil required to reduce wind loading.

The box sections were floated into position on barges beneath the bridge and then lifted up into position using winches supported on transverse girders mounted on the main support cables. Once in position the section was bolted onto its permanent hanger cables.

Erection began at the midpoint of the main span working towards both towers, and concurrently from both anchorages towards the towers to ensure that excessive forces did not build up in the anchorages.

Due to the distortion of the main cable during deck construction when the first two box sections were lifted into position the top edges were connected, however, the lower edges were lying far apart and could not be connected until a further 28 box sections had been put in place.

Figure 15: Deck under construction
All the transverse joints were manually welded following completion of the structure and then check using radiography. Any substandard joints were removed and re-welded.

In order to try prevent corrosion all surfaces of the steel deck panels were given three thorough weatherproof coat of paint before construction and a further two coats to the outside once in-situ.

The top surface of the deck was then surfaced with 38m of mastic asphalt, chosen because it’s very hard wearing and dense enough to prevent water penetrating to the steel deck.

The deck has a 2.8m expansion joint located at each tower to accommodate movement in the suspended structure due to changes in traffic, wind loading and temperature.

6. Loading Considerations

Although the main span of the bridge is 1,410m the deck can be considered to span between the hanger cables, the distance of which is approximately 18m. In addition to this although the deck is constructed in section its welded joints are sufficient for it to be considered continuous, the benefit being that abnormally large loads are distributed over a number of hanger cables and not just the nearest one. It follows that the worse case loading scenario is for every other span to be loaded as large deflections and bending moments will be experiences as in Figures 17-19.

6.1 Dead Loading

Table 1: Dead Loads for Bridge Components

<table>
<thead>
<tr>
<th>Description</th>
<th>Weight (kn)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Asphalt</td>
<td>34,335</td>
</tr>
<tr>
<td>Steel in Deck</td>
<td>166,770</td>
</tr>
<tr>
<td>Steel in Cables</td>
<td>107,910</td>
</tr>
</tbody>
</table>

Table 2: Safety Factors used in the Design

<table>
<thead>
<tr>
<th>Load</th>
<th>Limit State</th>
<th>( \gamma ) (safety Factor)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dead Steel</td>
<td>ULS</td>
<td>1.05</td>
</tr>
<tr>
<td></td>
<td>SLS</td>
<td>1.00</td>
</tr>
<tr>
<td>Dead – Super</td>
<td>ULS</td>
<td>1.75</td>
</tr>
<tr>
<td>Imposed</td>
<td>SLS</td>
<td>2.00</td>
</tr>
</tbody>
</table>

Table 3: Factored Dead Loads for Components

<table>
<thead>
<tr>
<th>Description</th>
<th>ULS Weight (kn)</th>
<th>SLS Weight (kn)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Asphalt</td>
<td>60,086</td>
<td>68,670</td>
</tr>
<tr>
<td>Steel in Deck</td>
<td>175,109</td>
<td>166,770</td>
</tr>
<tr>
<td>Steel in Cables</td>
<td>113,306</td>
<td>107,910</td>
</tr>
</tbody>
</table>

6.2 Live Loading

In order to calculate the traffic loading on a bridge the number of notional lanes must first be calculated. Although there are two lanes of traffic in each direction on the Humber Bridge the carriage way is approximately 19m wide which is equivalent to 6 notional lanes. There are three main loading conditions to consider:

6.2.1 HA Loading

HA loading conditions state that two lanes are fully loaded with a UDL, which together with a knife-edge load (KEL) represents heavy fast moving traffic. All other lanes on the bridge are then loaded with a third of the same UDL.

In the case of the Humber Bridge the worst two lanes to load heavily are the two central lanes as the deck is supported at the outside edges meaning this induces the greatest moment in the deck.

Since the deck spans less than 30m long the HA UDL can be assumed as 30kn/m. In addition to this a KEL of 120kn/m can be assumed in the centre of the deck, at the furthest point from the supports.

Total load on each 18.1m box section is:

\[
18.1((2 \times 30) + 2(\frac{2}{3} \times 2 \times 30)) + 120kn = 1,930KN
\]
6.2.2 HB Loading

HB loading is particularly relevant in such an industrial location as it take account for the likelihood of trucks transporting abnormal loads. Again the worst case scenario is that the HB loading and HA loading are applied in the two central lanes, creating the maximum moment in the deck.

It’s important to note the 25m unloaded region in front of and behind the HB load, which could cause an increase in longitudinal moment as shown in fig 21.

![Figure 21: Cross sectional diagram of worst case loading under HB conditions](image)

Total load on each 18.1m box section is:

\[
18.1(30 + 2(152 × 2 × 30)) + 1,800 = 3,067 \text{ KN}
\]

6.3 Wind Loading

In essence the longer a suspension bridge the more significant wind loading effects become. In addition, the nature of the site and the height of the roadway it’s also likely to be subject to high winds. Freedman fox chose the same approach to this problem as in their earlier Severn and Bosporus Bridges which is to design the deck as a neutral aerofoil such that winds are accelerated and pass over and under the deck, reducing the lateral force. In reality the wind loading on the deck was assessed using expansive wind tunnel testing, however, for the purpose of basic load consideration I will neglect the aerofoil effects and calculate the wind load using the principles outlined in BS 5400 for the usual 120 year return period.

\[
v_c = V \times K_1 \times S_1 \times S_2
\]

\[
V = \text{the mean wind speed (30m/s)}
\]

\[
K_1 = \text{a wind coefficient specific to the structure related to height above ground level (30m) and the length of the structure (2220m) and is given as 1.35 in BS 5400.}
\]

\[
S_1 = \text{a funnelling factor which is can be taken as 1 as the lack of other nearby structures mean funnelling effects can be discounted.}
\]

\[
S_2 = \text{a gust factor which is related to the height above ground level (30m) and is given as 1.21 in BS 5400.}
\]

\[
v_c = 30 \times 1.35 \times 1 \times 1.21 = 49.01 \text{ms}^{-1}
\]

In reality the design wind load used at the deck level was 48ms\(^{-1}\). It’s possible to use this wind speed to calculate the resultant horizontal load at the centre of the deck.

\[
P_t = q \times A_1 \times C_D
\]

Where \(q\) is the Dynamic pressure head, equal to:

\[
q = 0.613v_c^2
\]

\(A_1\) is the decks projected cross sectional area, however, the depth of the deck must be increased in include the area of vehicles on top of the roadway.

\[
A_1 = 2220 \times 7.5 = 16650 \text{m}^2
\]

\(C_D\) is calculated as a function of the b/d ratio, however, hollow box decks can’t be simplified to the standard cases and require wind tunnel testing to establish \(C_D\) in line with BS 5400. However, a similar deck construction to that of the Humber Bridge with identical b/d ratio returned a CD value of 1.1, which I will assume for the purpose of this analysis.

\[
P_t = (0.613 \times 49.01^2) \times 16650 \times 1.1 = 26.97 \text{MN}
\]

This is obviously a very significant load, which requires careful consideration to prevent bending, and possible failure of the deck.

Designing the deck as an aerofoil attempts to reduce the load, however, the major disadvantage is that higher wind speeds are experienced across the bridge deck than in the surrounding area. Furthermore, the industrial nature of the area means that much of the traffic will include high sided heavy goods vehicles which are particularly susceptible to crosswinds. This means the bridge will almost certainly have to be closed to some or all traffic should strong winds occur.

In addition to the aerodynamic deck the hanger cables are inclined to reduce longitudinal movements of the deck by creating a simple truss system. This was of particular importance to designers as the Tacoma Narrows had been seen to fail unexpectedly through such movements in 1940. However, inclined hanger cables result in a greater range of stress fluctuations as a result of live loads, and so are no longer considered in such structures.

One alternative would be not to design the deck as an aerofoil, reducing the wind speed over it. However, as this causes the lateral wind loading to increase the deck has to be heavily braced. In addition wind breaks could also be provided where required. However this is expensive and creates an overly deep deck which can affect the aesthetics of the bridge.

In my opinion the best option would have been to construct a cable stay bridge which would be considerably stiffer than an equivalent suspension bridge. This is because the cable stay bridges have a more direct load path from the deck to the pylons, through the cables. This increased stiffness could be used to resist the wind loading, neglecting the need for an aerofoil deck.

In addition to this there will also be wind loading on the towers which can cause substantial bending moments. For this reason the two towers are constructed of heavily reinforced concrete. Obviously as the towers are much higher than the deck a higher wind speed must be considered, for example the design wind speed at the tower top was taken as 66ms\(^{-1}\).
6.4 Loading due to Temperature Effects

When the temperature increases materials in the bridge structure will attempt to expand. If the members are restrained such that this expansion can’t occur, large forces can build up which could possibly lead to failure of a component.

6.4.1 Expansion from 25°C Rise Across the Section

\[ \Delta L = L \times \Delta T \times \alpha \]

Where \( \alpha \) is the coefficient of thermal expansion for Steel which is equal to \( 12 \times 10^{-6} \).

\[ \Delta L = 1410 \times 25 \times 12 \times 10^{-6} \]

\[ \Delta L = 0.423m \]

This is well within the movement allowed by the expansion joints and so does not pose a problem.

6.4.2 Force due to Blocked Expansion Joints

Under certain conditions the expansion joints can become blocked so it’s necessary to consider the forces which can build up should this occur. For the situation above:

\[ \sigma_y = \frac{E \times \varepsilon}{L} \]

\[ \varepsilon = \frac{\Delta L}{1410} = 3.0 \times 10^{-4} \]

\[ \sigma_y = 200 \times 10^4 \times 3.0 \times 10^{-4} \]

\[ \sigma_y = 60N/mm^2 \]

We know each section weighs 1,177KN and is 18.1m long. So if we assume the density of steel is 76.9KN/m³ it’s possible to calculate the area of cross section of deck to be 1.15m². The force in the deck can now be calculated to be:

\[ 0.85 \times 60,000 = 50.7MN \]

This is obviously a very large force which could result failure of the bridge. Even were this not the case it would significantly affect the ability of the bridge to carry other loads, which underlies the importance of regularly inspecting the expansion joints.

6.4.1 Uneven temperature rises across the section

Usually a bridge will not expand uniformly across the section, and will, instead expand more at the upper face than the lower face, as shown in figure 22.

There are a number of other loading conditions which would be considered for the full structural analysis of the bridge, but for this simplified analysis I shall consider a few of the more important in bridge design.

The stress relaxation of steel cables in the bridge could become a considerable problem. All steel cables under a sustained tensile load will lengthen with time which could cause increased deflections in the deck, which could present a serviceability issue. One approach to combat this is that the deck is designed to curve up to a highest point in the centre of the span such that any additional deflections only act to reduce the curvature.

Earth water pressures require close consideration on such a site, as the river is renowned for its rapidly changing water levels, and consequently pore water pressures. Such pressures could create a cyclic loading pattern on the foundations which could result in damage over time.

The natural flow of water in the river channel will also exert a horizontal force on the Barton Pier. To attempt to reduce this force the piers are shaped to be as streamlined as possible, although this force is still likely to be significant. There will be an additional snow loading applied to the entire bridge in the region of 0.5kn/m². Vehicles can also cause additional loading in excess of their normal HA or HB loading. This can be due to a collision with a parapet or part of the superstructure or a vehicle skidding or breaking heavily.

7 Basic Structural Calculations

In order for the bridge design to be successful it must pass the design checks for both Ultimate Limit State (ULS) and Serviceability Limit State (SLS). ULS is a test that the members in the bridge are strong enough to prevent collapse, whereas SLS checks that the structures usage will not be impaired by excessive deformation.

7.1 ULS Capacity of hanging cables

It’s possible to assess the adequacy of the hanger cables by calculating the total load of the deck and then comparing this to the failure load of the hangers.

Total Deck Weight = Asphalt Weight + Steel Weight

\[ 60,086 + 175,109 = 235,195 \text{ KN} \]

Dead Load/Section = 235,195 ÷ 124 = 1,897KN

Total Live Loading/Section (HA) = 3,067KN

Total Load/Section = 4,964KN

Each precast deck section is supported by four hangers, such that the maximum load supported by each hanger is 1,234KN.

The capacity of the cable can be calculated using the diameter, and assumed yield strength of 275N/mm².

Diameter of hanger cable = 62mm
Cross section area of hanger = \( \pi r^2 \)
\[ \pi \times 0.062^2 = 0.012m^2 \]
Maximum load capacity of Hanger = \( \sigma_y \times A \)
Capacity = 275,000 \times 0.012 = 3,300KN

The Factor of safety can now be calculated to be:
\[ \frac{3,300}{1,234} = 2.67 \]

**7.2 ULS Capacity of Main Suspension Cables**

Similarly it’s possible to assess the adequacy of the main suspension cables.

![Diagram of Main Suspension Cable](image)

**Figure 23: Bridge Dimensions**

Consider the cable supporting the central span.

![Diagram of Main Suspension Cable](image)

**Figure 24: Diagram of Main Suspension Cable**

The maximum tension occurs in the table at the supports where;

\[ T_y = \sqrt{H^2 + V^2} \]

It can also be shown that:

\[ H = \frac{\omega L^2}{8f} = 1883\omega \]
\[ V = \frac{\omega L}{2} = 705\omega \]

Although initially it appears that HB loading should be considered at the above value this doesn’t take into account the 25m unloaded span either side of the truck. Assuming the truck to be 10m long the total region occupied by the HB loading is 60m and the region carries the same total load of 1800KN as under HA conditions. For this reason when considering the entire span HA and HB conditions will be equivalent in this situation.

Total Live Loading/Section (HA) = 1,930KN

Total Load/Section = 3,827KN

Each deck section is 18.1m long such that the load on the cables is:

\[ (3,827 + 2) + 18.1 = 106KN/m \]

The self weight of the cable must also be included giving the total load on the cable as:

\[ 113,300 \div 4440 = 26KN/m \]

Thus, 
\[ \omega = 132KN/m \]

Therefore, the Tension induced in the cable can be calculated as:

\[ T_y = 265,400KN \]

Although no data could be located for the yield strength of the wire in the cables it’s possible to calculate the yield strength required to carry this maximum loading condition and then check it appears reasonable.

Diameter of main suspension cable = 700mm

Cross section area of cable = \( \pi r^2 \)
\[ \pi \times 0.35^2 = 0.38m^2 \]

Maximum load capacity of Hanger = \( \sigma_y \times A \)
\[ \sigma_y \times 0.38 = 265,400KN \]
\[ \sigma_y = 700N/mm^2 \]

This may initially appear high, however, suspension cables are high strength by requirement. A good example of this is the recent Akashi Bridge in which the suspension cables have a tensile strength of 1,800N/mm². Its therefore possible to conclude that the main suspension cables are of an adequate strength.

**7.3 SLS Assessment of Main Cables**

The main serviceability requirement of a suspension bridge is that the main cables don’t deflect to the extent that the deck deforms so as to inhibit its use. The cables were constructed higher than their final position; such that one the deck was completed its dead load would cause the cables to deflect such that the deck was in its desired location. For this reason only the live load need be considered when assessing the extensions in the main cable.

Obviously the load in the cables is not constant and is at a minimum in the centre and a maximum at the supports, however for this simplified calculation I will overestimate the extension by assuming the maximum tension at the support is constant across the length of the cable. The Live loading, excluding safety factors, can be calculated to be:

\[ \omega_{Live} = 53KN/m \]

Using the same approach as in section 7.2:

\[ T_{Live} = 106,500KN \]

The extension of each cable can be calculated using:

\[ \Delta L = \frac{T_{Live}L}{EA} \]
\[
\Delta L = \frac{106,500 \times 1410}{200 \times 10^6 \times 0.38}
\]
\[
\Delta L = 1.98 \text{m}
\]
Although initially a 2m extension in the cable seems quite large, this is equivalent to a 0.2m drop at the centre point which is perfectly acceptable over such a large span.

8 Natural Frequency

Natural frequencies were of particular concern at the time of conception of the Humber Bridge due to the Tacoma Narrows collapse. Furthermore the longer the bridge the more significant the loads associated with natural frequency vibrations become. It’s extremely difficult to assess the natural frequency of a bridge and would require complex calculations which can’t be reproduced in this paper.

In order for natural frequencies to become problematic vibrations must firstly be induced. British Standards state in the design of road bridges vibrations due to traffic such neglected such that only wind need be considered. Although the maximum wind load has been assessed using a simplified approach in section 6.3 it’s impossible to assess how this will affect the bridge in terms of vibrations without the use of complex computer models, or wind tunnel testing, as was used in design of the Humber Bridge.

9 Maintenance and Deterioration

The Humber Bridge had been designed with a number of permanent features to aid future maintenance.

Recent intrusive testing on the similarly designed Forth and Severn Bridges revealed significant unexpected deterioration of the main suspension cable. This corrosion is thought to be a result of water penetrating the external skin of the cables.

As yet Intrusive testing has not yet been performed on the Humber Bridge, mainly as it’s a younger structure, so significant corrosion of the main cables is less likely, however, it is expected that significant deterioration in line with the Severn or Forth bridges will be found in due course.

Although hanger cables can easily be renewed, replacing the main cable in suspension bridges is extremely difficult. The maintenance would have been much simpler had a cable stayed structure been constructed. This is because cable stayed bridges have more, shorter, lighter cables, each of which is easier to remove and replace, greatly extending the life of the bridge.

In addition, the box sections of the deck are also made of steel and are consequently also prone to corrosion. To prevent this, the steel in the bridge deck requires regular applications of weatherproof paint. This requirement could have been removed had the deck been constructed of reinforced concrete like the towers, however, this would have increased the weight of the deck which would increase the material utilised in cables and the towers.

The only other major maintenance required by the bridge since construction was the resurfacing of the roadway, which was carried out by Colas Civil Engineering. The original surface was removed using special equipment before the deck was cleaned which high pressure water, and shot blasting. The Deck was then surfaced with a combination of epoxy water proof layer and spray grip bitumen.

10 Susceptibility to Intentional Damage

Susceptibility to intentional damage is becoming increasingly important, especially in the current climate of terrorist activity. The Humber Bridge was designed with inherent redundancies, such that in most cases failure of a certain member would not result in a catastrophic collapse.

It requires complex structural calculations to asses exactly which parts of the bridge would cause a catastrophic collapse if removed. However, it can be seen from calculations in section 7.1 that each hanger can carry more than twice the maximum load required, however, if every other hanger were removed the deck may collapse as it would have to span further, creating higher bending moments.

Obviously there are certain aspects of the bridge which would if removed cause a catastrophic collapse. These include the anchorage, piers, towers and main suspension cables. However, these members are over engineered through the use of safety factors such that partial damage will not necessarily cause a catastrophic failure.

11 Future Alterations

The Humber Bridge was constructed to enable the growth of an industrial town on the south bank. However the predicted growth never occurred, and the large tolls mean that the bridge is not yet a significant part of the national road network. As a result the bridge serves no major purpose, although an eastern motorway utilising the bridge is planned. The result is that the bridge is currently under used and so no expansion is planned or required. Furthermore significant fractions question whether the bridge ever should have been built and if it will ever manage to pay back its construction costs.

The government was recently forced to freeze the interest payable on the loan in order for the Humber Bridge Board to remain solvent. Since then the government has come under increasing local pressure to write off the debt and incorporate the bridge into the national road network, scrapping the tolls in the process. It’s predicted that only when this occurs will the bridge recognise its full potential.

12 References

1 Bridge Engineering Notes By Tim Ibell, University of Bath
2 Bridging The Humber, The Humber Bridge Board