HUNGERFORD BRIDGE

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Abstract: This paper looks at Hungerford Bridge and the Millennium footbridges either side of it. The paper looks at the history of the site and then focuses on the design and construction of the two millennium footbridges either side of the Charing Cross railway bridge.

Keywords: Hungerford, bridge, cable stayed, Millenium

1 Introduction

1.1 History of site

Hungerford Bridge was originally a suspension bridge built by Isambard Kingdom Brunel providing link between the South Bank and Hungerford market for pedestrians and horse-drawn vehicles. The bridge had a central span of 200m between brick piers in the river. Hungerford Bridge was first opened on May 1st 1845 (Fig 1). Hungerford market was closed shortly after the bridge had been built and the site was sold for the proposed Charing Cross railway station. This required the pedestrian bridge to be replaced with a railway bridge.

In an early example of recycling the chains from the suspension bridge were sold for £5000 and used to complete the Clifton Suspension Bridge in 1864, another of Brunel’s bridges. Work on the new Charing Cross railway bridge begun in June 1860 to the designs of Sir John Hawkshaw.

The new bridge is made up from wrought iron lattice girders and consists of nine spans. The bridge reused the existing original brick piers from Brunel’s bridge and the on South Bank side the buttress still uses the steps from the original pier Brunel built. The new iron bridge was completed in 1864.

Walkways either side of the railway bridge were added on at a later date making the bridge one of only three in London to combine both rail and pedestrian use. When the railway bridge was widened one was removed to make space for three additional tracks. A temporary Bailey Bridge was added in 1951 and in 1980 another temporary pedestrian bridge was added during refurbishments of the existing bridges.

Figure 1: Brunel’s original suspension bridge
1.2 The new walkways

Hawkshaw’s railway bridge was never liked by Londoners and for many years new schemes were devised to improve or replace the bridge. These ideas varied from simply putting a cover over the railway to constructing an entirely new station on the South side of the river and providing a monorail link over the river. The footbridge was narrow, noisy and dangerous and in 1996 the Cross River Partnership used lottery money to hold a competition. It was decided that new pedestrian bridges on either side of the rail bridge would regenerate the South Bank and aid tourists and commuters. 6 teams were short listed from an initial group of over 40 all with very varied schemes. The six finalist’s schemes were

(a) a multi-span cable stayed bridge
(b) a steel girder bridge
(c) a suspension bridge
(d) a slender steel box girder
(e) a truss that enveloped the railway bridge
(f) a combined cable-stayed and beam bridge

The architects Lifschutz Davidson and the engineers WSP Group won the competition with their multi-span cable stayed bridge. The two four metre wide cable stayed bridges were completed in 2002. They were named the Golden Jubilee bridges.

Figure 2: Site Map surrounding bridge

1.3 Aesthetics

The footbridge offers views along the Thames and the Southbank for pedestrians but also stands out as one of the Thames’s most aesthetically pleasing crossings. The angled supports offer a range of view points to pedestrians walking up to the bridge. Hungerford Bridge has now changed from being an ugly railway bridge into an exciting architectural feature. The architects chose to use more cables then the minimum to keep the diameters small. This gives the effect of a floating deck with only the main pylons visible from afar. At night the bridge is down lit from the top of the angled pylons. The strong lighting on the bridge keeps the ugly railway bridge hidden when viewed from the river bank. The strong lighting also solves one of the original problems of the bridge being very dangerous at night.

Figure 3: The bridge lit at night

Figure 4: View from Charing Cross station side
2 Loading

The loadings on the bridge due to pedestrians were taken from the Department of Transport standard BD37/88\textsuperscript{12} and the British Standards BS 5400\textsuperscript{11}. Load patterns were then applied to the bridge to work out the forces in the elements. The structural analysis program STRAP was used to process the results and find the worst load combination.

The dead loadings on the bridge will be relatively low as the deck is light weight steel and the balustrades and fixings will not add too much additional weight to the structure.

Under BS 5400 the live loading applied to the footbridge is 5KN/m\textsuperscript{2} which can be then reduced by a factor $k$ for normal use. However this value should be kept the same because the bridge could be subject to excess loading, for example on opening day.

There is no need for HB loading as there is no possibility of vehicle loading onto the bridge as the bridge has effective barriers at both ends.

The cable stays are arranged in a single plane with 9 cables spanning out to each side of the bridge. Each cable is a pre-tensioned rod which carries the weight of the deck.

The dead loads from the bridge are assumed to be equal to the combined loading from the balustrades, decking and the steel edge beams. I have assumed this value to be equal to 4 kN/m.

The angle between the cable and the deck is equal to 43°. The downward force carried by the cable can be worked out as a combination between the live and dead loadings. This can then be turned into a tension force along the steel rod and then the stress in the cable can be worked out using Eq. (1).

Length of bridge deck supported by worst case cable = 6m

Area of bridge deck supported by worst case cable = 12m\textsuperscript{2}

Combined Loadings using load case 1:

- Imposed = $5 \times 1.75 \times 12 = 105$kN
- Dead = $4 \times 1.05 \times 6 = 25.2$kN

$$\text{tension factor} = (25.2 + 105) \sin 43 = 88.8$kN

Assuming a diameter of 40mm this creates a stress in the rods of:

$$\sigma = \frac{F/A}{50.27} = 1766 \text{ N/mm}^2 \quad (1)$$

Wind tunnel tests were completed and it was found that although the critical wind speed was much higher than expected, the vibrations should not be noticeable. The Charing Cross rail bridge shields the footbridge when it is upwind and when it is downwind the air flow separates so the wind effects on the footbridge are reduced. A basic wind calculation can be performed to find the forces experienced on the bridge. An extra reduction factor on 0.75 can be used because the bridge is a footbridge. These calculations do not take into account the wind shielding from the railway bridge so in reality the only accurate way if to use the wind tunnel tests. The
rods are circular sections which minimizes the wind drag on them.

\[
\text{maximum wind gust, } v_c = v K_1 S_1 S_2 \\
= 0.75 \times 26 \times 1.23 \times 1 \times 0.89 \\
= 21.3 \text{ m/s}
\]

Horizontal wind load, \( P_t = q A_1 C_D \) (3)

Dynamic Pressure Head, \( q = 0.613 v_c^2 \) (4)

\[
= 0.613 \times 21.3^2 \\
= 278 \text{ N/m}^2
\]

Taking a \( b/d \) ratio of 2.2 gives a drag coefficient value, \( C_D \) of 1.48

The solid horizontal projected area, \( A_1 \) can be taken as 600m².

Referring back to equation (3):

\[
\text{Horizontal wind load, } P_t = 278 \times 600 \times 1.48 \\
= 247\text{kN}
\]

A vertical force can also be found using equation (5):

\[
P_v = q A_3 C_L \\
= 278 \times 1400 \times 0.4 \\
= 156\text{kN}
\]

Again it must be mentioned that these calculations only give a rough guideline as in reality a wind tunnel test must be used.

3 Temperature

Two temperature effects must be considered for the bridge:

1. The overall temperature increases and decreases
2. The change in temperature between the top and bottom surfaces.

For the location the maximum and minimum temperatures are shown in table 1:

<table>
<thead>
<tr>
<th>Table 1: Maximum and minimum temperatures</th>
<th>Temperature °C</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum</td>
<td>35</td>
</tr>
<tr>
<td>Minimum</td>
<td>-10</td>
</tr>
</tbody>
</table>

The temperature difference between the top and bottom of the bridge is going to be the key factor to see how much the bridge wants to bow upwards.

4 Dynamics

The acceleration of the bridge deck is the critical issue of the dynamic behaviour of the bridge. High acceleration of the bridge deck can lead to increased forces in the members and also could be felt by pedestrians. Using the program LUSAS the dynamic response of the bridge could be found, the results can be seen in Table 2.

<table>
<thead>
<tr>
<th>Table 2: Calculated accelerations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Loading Case</td>
</tr>
<tr>
<td>-----------------------------------</td>
</tr>
<tr>
<td>single person defined in BD 37/88</td>
</tr>
<tr>
<td>single person defined in ENV 1995 Part 2</td>
</tr>
<tr>
<td>single person at high natural frequencies</td>
</tr>
<tr>
<td>vandal loading of ten people</td>
</tr>
<tr>
<td>vandal loading with tuned mass dampers</td>
</tr>
<tr>
<td>rush hour loading of 250 people</td>
</tr>
<tr>
<td>opening day of 2000 people on the bridge</td>
</tr>
</tbody>
</table>

0.5% critical and 0.7% critical damping was used for single person loading and many people loading respectively. As well as this the effect of 1.0% damping was looked at.

Vandal loading is when a group is synchronised through jumping or dancing. The frequencies given for groups jumping in a coordinated manner are

(a) 1.5-2.5 Hz for a small group
(b) 1.8-2.3 Hz for a large group

4.1 Acceptable acceleration

There is an acceleration at which pedestrians on the bridge are discomforted. BS 6841:1987 states the following values for human comfort in reaction to the root mean square acceleration

(a) slightly uncomfortable: 0.3-0.6 m/s²
(b) fairly uncomfortable: 0.5-1.0 m/s²
(c) uncomfortable: 0.8-1.6 m/s²

With these values in mind the acceptable limit for the acceleration was given as 0.7 m/s² for normal usage. In the case of vandal use it was assumed that other pedestrians could see the cause of the increased vibrations so the acceptable limit was set to 1.5 m/s².

4.2 Transverse oscillation

Hungerford footbridges were under construction at the same time the Bankside Millenium footbridge were experiencing the oscillations. Excessive sideways movement is thought to only occur at horizontal frequencies of under 1.3 Hz. The lowest horizontal frequency was 1.6 Hz so the bridge was assumed to be safe from excessive oscillations. The Bankside Millenium footbridge had frequencies of 0.49 Hz on its central span.
4.3 Deflections

The maximum deflection on opening day with 2000 people on the bridge was calculated to be 4mm. Deflections due to vandal loading were calculated to be 15mm. The static deflections under loadings were 53mm for the standard span and reached 100mm on the span with the movement joint in place.

4.4 Testing the bridge

4.4.1 Frequencies

Before the upstream bridge was opened, tests were carried out to assess the damping characteristics of the bridge and to verify the calculated natural frequencies of the bridge. An accelerometer was used to measure the actual natural frequency of each span. The tests were carried out at night to avoid any interference from the Charing Cross railway bridge. The results showed the calculated frequencies to be on average 5-9% lower than the measured values. For the horizontal frequency the calculated value was 1.64 Hz, 8% greater than the calculated value.

4.4.2 Accelerations

Three scenarios were set out to measure the maximum accelerations on the bridge. Table 3 shows the maximum accelerations under the different loading conditions.

<table>
<thead>
<tr>
<th>Loading Condition</th>
<th>Acceleration m/s²</th>
</tr>
</thead>
<tbody>
<tr>
<td>157 people at 1 Hz stepping pace</td>
<td>0.09</td>
</tr>
<tr>
<td>157 people at 2 Hz stepping pace</td>
<td>0.15</td>
</tr>
<tr>
<td>4 men running along the bridge</td>
<td>0.12</td>
</tr>
</tbody>
</table>

4.4.3 Conclusions

The calculated values were within 10% of the actual measured values. For a bridge of this complexity these are satisfactory results. For the case of vandals deliberately trying to oscillate the bridge it was concluded that the case is very unlikely and the cost of additional dampers would not be justified.

5 Connections

The cables used were Macalloy 460 rods which were bolted to the side of the deck. The cables were attached using steel plates and then stressed. These steel plates were pinned to 2.4m steel fabrications at the top of the tubular pylons. The connections at the top of the pylons were referred to as ‘Angel’s wing’s’ (Fig 9) because of their shape. The connection detailing allowed for a large amount of cable to join together at the top without the structure looking too crowded.
6 Foundations

The foundations consisted of piles in groups of four used upstream and downstream of the railway bridge. The piles were then connected by pairs of concrete beams 40m long, 2.5m deep and 0.6m thick. These play the role of ship impact beams and were designed to dissipate impact forces among the pile groups. To avoid sheet piling under the railway bridge the impact beams were made from precast concrete and brought to the site on barges.

The piles used varied in length from 9m to 42m but all were 1.5m in diameter and were bored cast in place piles. Tubular casings were driven through the river bed into the London clay beneath.

The foundations would experience downward forces from the pylons and uplift from the backstays in addition to the ship impact loads. It was also important that the foundations did not look too large to be supporting the footbridge. The foundations were curved to follow the flow of the forces and to soften the edges of the concrete. The back stays are connected to the 3m columns from the existing railway bridge. At high tide the two D-shaped pylon foundations are visible and at low tide the curved ship impact beams can also been seen.

The pile caps were 10.5m long, 7m wide and 1.8m deep. These were formed within cofferdams formed of sheet piles.

Figure 11: Cross section showing foundations

6.1 Ship Impact

One of the major challenges with the design of the bridge was the risk of ship impact. It was decided that there was a significant risk of impact due to the meandering nature of the Thames. During the design of the pedestrian footbridges a 1000 ton vessel hit Putney Railway Bridge which was similar in design to Charing Cross Railway Bridge. The impact caused one of the piers to be displaced and the truss girders were left some way from the centre of the support. Immediate impact protection was provided by surrounding the supports with steel caissons filled with gravel. A more aesthetically pleasing solution was sought after which protected the railway bridge supports from impact and had minimal effect on ships navigating the Thames.

The bridge was designed to withstand the forces of a 3000 Mg ship travelling at 6.2 m/s and it was also decided that there was a possibility of the ship hitting at a small angle. The substructure was designed according to the AASHTO Guide Specification which recommended that ‘critical’ bridges be designed for a one in thousand year impact. The American standards were used to determine the maximum ship impact force as the British and Euro codes did not cover ship impacts. Firstly Minorsky’s method can be used:

\[
\text{Kinetic Energy} = 0.5 \times m \times v^2 = 0.5 \times 3 \times 10^6 \times 6.2^2 = 57.7 \text{MJ}
\]

From Minorsky’s graph of energy absorption the volume of steel crushed \( V \) can be found using Eqn 6:

\[
E = 29 + 42V
\]

\[
V = 0.68 m^3
\]

For a 3000 ton ship, a value of 0.52 \( m^3 \) of steel per metre length of the ship can be used and it was assumed that only 2/3 of this will deform plastically on impact.

Stopping distance, \( x = 0.68/(0.67 \times 0.52) = 1.96m \)

Stopping Force, \( F = E/x = 29.5 \text{ MN} \)

This maximum value was suitable for head on collisions. If the vessel hit the pier on one side there would merely be a glancing blow and the vessel would not be brought to a halt. at +/- 15 degrees or 8MN laterally. The bridge was designed to carry a normal lower impact load of 2MN. Crumple zones were not used because of the risk of damage from relatively low energy impacts. The structure was designed to be solid and robust and concrete was removed where it was not carrying load for aesthetical reasons. For the high impact cases the substructure was allowed to deform and transfer some of the loads to the railway bridge supports, where there was spare capacity. The impact was resisted by the weight of the footbridge and railway bridge, by the tension in the new piles and by lateral and compression forces in the railway bridge caissons.
Figure 12: Dissipation of forces in ship impact

Figure 13: Ship impact beam being installed

Figure 14: Connecting the piles to the impact beam
7 Construction

Firstly new piles were constructed upstream and downstream of the bridge (Fig 16). Steel tubular casings were driven into the London clay and bored piles were constructed using barge-mounted plant. The ground under the planned supports contained pockets of permeable material. To stop water inflow casings going up to 40m in length were needed. To complicate matters even further it was known from the very beginning that there was a possibility of unexploded bombs in the river bed. The Northern and Bakerloo tube lines flow near the foundations causing reason to be cautious. A magnetometer survey was completed by divers to a depth of 3m and it showed that there was little chance of a bomb being found. Although the risk was small the consequences were very serious so extra precautions were decided to be taken. The foundations near the tube tunnels were completed over the weekend with the tube lines closed and flood barriers in place. In these locations the 4.8m diameter and 20m deep shafts were hand dug although no unexploded bombs were found.

Concrete was used to join the impact beams above water level.

Steel sheet piled coffer dams were then constructed around the piles and dewatered (Fig 17). Concrete pile caps were constructed after the pile casings were cut off at water level. Ship impact beams made of precast concrete were brought to the site by barge and hung from temporary works using strand jacks (Fig 18). In-situ concrete was used to join the impact beams above water level.

Then the temporary works and coffer dams were removed and the concrete was lowered onto the pile caps (Fig 19). Precast shells filled with in-situ concrete forming the permanent concrete substructure were completed next. Concrete was placed inside steel shutters to form collars around the railway bridge casions.
The 650mm deck was cast in sections and stiffened with temporary steel trusses (Fig 21). The bridge was cast incrementally towards the north bank. Next tie-downs and diagonal struts from the pylon bases to the tops of the tie-downs were erected. Temporary stiffening frames were used in assembling the pylons and backstays. These were lifted over the top of the temporary truss by a floating crane before the deck supporting cables were attached. The temporary works were removed and finishing touches such as fixing the handrails and deck surfacing were completed. Once the upstream bridge has been completed the downstream bridge can be built in the same way. The existing walkway on the railway bridge can be removed to complete construction.
8 Future Changes

The modern design of the bridge is a highly successful and popular among Londoners. The weak point of the bridge is undoubtedly the railway bridge in between the pedestrian bridges. The railway bridge is in constant use and closing the bridge to be redesigned would cause disruption to commuters. The pedestrian bridges were able to be completed without ever stopping the flow of pedestrians across the bridge. This could be possible with the railway if some of the tracks were kept open whilst the other side was redeveloped.

Acknowledgements

The author would like to thank for the help of the ICE library and the help of John Parker from WSP Group.

References

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