A CRITICAL ANALYSIS OF THE HULME ARCH BRIDGE, MANCHESTER

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Abstract: This paper evaluates the aesthetics, structural design and construction of the Hulme Arch Bridge in Manchester. It will explore the nature in which the bridge was initially conceived involving relevant precedents and design concepts, eventually leading on to a critical breakdown of the aesthetics. A detailed analysis of the structural concepts, loading conditions and serviceability of the structure will follow. There is also considerable insight into the construction process and the general functionality of the structure. The paper pays particular focus to the orientation of the steel arch and asymmetric cable arrangement that makes the structure so unique.

Keywords: Hulme, diagonal arch, cable-stayed, gateway, asymmetric;

1 Introduction

The Hulme Arch Bridge was commissioned as Phase 4 of the Hulme City Challenge Initiative (1992), a £37.5 million government regeneration package over 5 years to create a high quality urban environment for the residents. This scheme was devised to mitigate a number of issues associated with the Hulme area.

Hulme is located in a south-eastern area of Manchester. The area was commonly linked with poor housing, high unemployment rates, a slowing micro-economy and limited opportunities. Local council urban planners had been criticised for their awful zone allocation and segregation of the area, which were considered to be contributory to the degradation of the area [1]. Previously Stretford Road had provided a key east-west access link to the residents but the construction of a major carriageway in 1962, Princess Road, bisected the route. As part of the regeneration scheme a unique bridge was commissioned to reinstate Stretford Road and unite the separate areas.

As part of a design competition, Ove Arup & Partners and Chris Wilkinson Architects entered a joint project which incorporated a ‘gateway’ arch structure spanning 52m over Princess Road. The diagonal parabolic arch rises 25m above the bridge deck and was fabricated as a trapezoidal plated steel box section. The composite concrete deck is supported by an arrangement of 22 asymmetrically orientated steel cables hung from the arch. The deck is 18m wide and carries 2 lanes of traffic and also has provision for pedestrians.

The bridge highlights the effectiveness of engineering design simplicity and how it can be manipulated to create visual intricacy. The design has won critical acclaim from numerous institutes, including the RIBA Award for Architecture and the ICE Merit Award, stressing the underlying importance of any successful structure; the careful integration of architecture and engineering.

2 Aesthetics

To the majority of the public, one of the most important aspects of any successful bridge is aesthetics and in order to analyse this particular bridge, a set of principles devised by Fritz Leonhardt will be used to objectively assess the aesthetical impact of the bridge. His work suggests ten areas that each address a different aspect of aesthetics, and it is a simple yet effective way of determining the aesthetical appeal of a bridge.

One of the most critical aesthetical considerations is related to how the bridge functions, in other words how it discloses its structure to the viewer and also how structural stability is communicated to the user. The engineers and architects worked closely to ensure that as the position of the viewer changes, the complexity and character changes and the balance between symmetry and asymmetry is the main contribution to this. The two sets of cables are arranged to interlock and to symbolise the renewed unity of the two halves of Hulme.

Fig 1: Skyline at night
The design team have achieved something very special with this bridge, through the use of simple geometry and clear form, the end result is a piece of visual complexity that not only provokes discussion amongst users but visually acts as a ‘gateway’ for Hulme.

Most cable-stayed bridges promote a high degree of stability, yet it seems fine to question the structural stability of this particular structure, something that sets this design apart from other bespoke bridges in the UK. Observing the structural hierarchy of the bridge it seems obvious that the steel arch carries the load from the cables, which in turn carries the lightweight deck; however from certain viewing angles there is bound to be a degree of uncertainty generated by the overlapping cable pattern, Fig 3. This also happens to be synonymous with the most frequent view of the bridge (elevation) and this helps develop intrigue in how the bridge functions, as well as further emphasising the effective balance of symmetry and asymmetry for aesthetics.

Proportionally, the bridge seems to have the right balance between each particular element, attention is immediately drawn to the arch and cable system, due to the slenderness of the deck, following the flowing curve from one end to another. Relative to the small span, the steel trapezoidal arch seems in good proportions to the length and thickness of the deck; it seems sensible that the deck is thin enough to suggest its reliance on the arch and cables for stability. The cables also seem suited, in terms of size when considering the connection between them and the arch, tying the whole structure together as a carefully balanced and well proportioned bridge. In terms of masses and voids the asymmetric cables effectively divide the space created within the arch, hence the impressive transverse views, especially at night with careful artificial lighting, Fig 4.

Simple geometry used effectively give this particular bridge a distinct order; many cable-stayed bridges have aesthetical issues associated with views from slanted angles, due to their linear simplicity, however in this case the Hulme Arch Bridge excels from the more angled viewpoints. Visual intricacy between the asymmetric cables and the diagonal plan shift give this bridge considerable character, emphasising the ‘gateway’ inspiration that drives the design.

As a focal point for the regeneration of Hulme, it is imperative that any succinct design refinements enhance the aesthetics and first-hand experience for travellers. Low-level lighting integrated into the lower structure of the bridge illuminate the underside of the arch and accentuates the primary elements of the bridge; the lighting scheme also ensures unobstructed views, where lampposts would detract from the visual impact of the arch and the cables. A constantly changing tapered arch, which is wide and shallow at the crown and deeper and narrower at the springings, creates visual continuity and also benefits from the play of light and shade on the structure [2]. Tubular steel nosing supported outside the cable brackets further promotes the smooth continuity of the structure.

The surface textures applied to the structure wanted to encompass the ideology behind the design; therefore the choice of finish should reflect some of the connotations associated with the new start for Hulme. The smooth surfaces of the steel nosing and the arch itself give the structure a feeling of elegance and simplicity; finished with a shiny aluminium coating, it is illuminated at night to add to the visual impact of the structure. The simplicity of the colour scheme helps integrate the bridge into the surroundings; on the contrast, in the backdrop of the skyline, the darkness of the palette helps it stand out, making it visible from the far extents of the area.
The principal design precedent for the bridge has been cited to be Eero Saarinen’s Gateway Arch, St. Louis built in 1964 (fig 5.) [3]; a landmark ‘gateway’ structure for St. Louis, the design team wanted to encapsulate the same kind of theology for this structure and the region of Hulme. As the principal symbol of the regeneration of Hulme, the bridge has won critical acclaim from the public, as well as the professionals; it’s difficult to define character but this bridge undoubtedly has character aplenty. A totally bespoke form, with a selective balance between simplicity and complexity, the Hulme Arch Bridge could have easily fallen into the ‘generic’ cable-stayed bracket, yet the design team have engineered a total solution that not only acts as a ‘gateway’ structure for Hulme but as a global engineering achievement.

3 Structural Design

The Hulme Arch Bridge has three primary structural elements that function collectively to distribute the range of loads and stresses that the structure experiences; they are the arch, the deck and the asymmetric cable arrangement, Fig.6. Through critical analysis of each of these elements and how they interact with the other key structural elements, a broader idea of how the bridge functions structurally can be compiled.

3.1 Arch Design

The steel trapezoidal box-section arch was specified to be parabolic; the advantages being that when the bridge is in equilibrium the arch should be in pure compression and that it also ensures that the thrust line generated by the in-plane effects of dead loads follow the centerline of the arch section, reducing moments generated by the eccentricities.

However, as a result of the asymmetric cable arrangement, large out-of-plane bending moments are generated within the arch; they are so significant structurally that they become the most onerous design load for the arch. Consequently, the arch acts more like a laterally loaded bending member as opposed to a conventional arch [4].

There is a net rotational force in the arch, generated by the asymmetric cable arrangement, this is carried by the large concrete foundations and the crown of the arch is also filled with concrete to resist the turning force and also increase the stiffness of the member, Fig 7.

Fig 7: Out-of-plane bending moment diagram - arch

Significant axial stresses are generated by the axial and bending effects experienced within the arch and the stresses are carried by the arch top and soffit plates. Due to the unordinary nature of the force distribution, stiffening plates and diaphragms have to be provided to stop the plates moving out of plane. Where the cables are fixed to the arch through lugs attached to the soffit, there would be high stress concentrations where the point loads act, and therefore the diaphragm distributes the load across the section.

3.2 Cable Design

The composite concrete deck is supported from the arch by 22 diagonal spiral-strand cables and each cable is 51mm dia. with a minimum breaking load of 216 t [4]. The cables are asymmetrically arranged and orientated to fan out in opposing directions, such that each side of the deck is fixed to a separate half of the arch, such that 11 cables support each side of the deck. A degree of redundancy has been designed into the structure such that the bridge will handle a removal of a single cable from either side, whether it’s accidental or for maintenance purposes.

To ensure safety during use, the cables were required to have vertical and horizontal clearance and the dimensions were calculated for both the carriageway and the footpath. Concrete bollards were installed at regular intervals along the deck to prevent high vehicles sailing up over the footpath and snagging the cables, causing structural instability.

3.3 Deck Design

The bridge deck is designed as a composite concrete slab cast on permanent formwork, which in turn is arranged on a series of 17 transverse girders spanning to 2 steel edge beams. The steel cables are connected to the deck through outrigger brackets. The deck is designed for vehicular and pedestrian use, with two 5m single carriageways and two 2.5m footpaths. [4]

4 Articulation
To ensure sufficient articulation of the bridge deck and the structure itself, the installation of a series of bearings allow for vertical and lateral movements, which are associated with temperature effects and settlements. There are four vertical support bearings installed at each corner of the deck, the two bearings at the east end are effectively restrained, thus expansion is allowed at the west end. To locate the deck laterally, there is a transverse guide bearing situated in the middle of the two end girders; these bearings also resist the turning force generated from the asymmetric cable arrangement.

In order to limit the vertical deflection of the expansion joint to 3mm [5], two free-sliding vertical support bearings were installed after the permanent deck loads and the cable pre-stress were in place. Frequent maintenance and cleaning is required to ensure any expansion joints do not become cluttered with natural material (i.e. leaves, soil etc.) as this would generate unwanted stresses within the deck.

5 Loading

Every bridge has a variety of load combinations that the main structure has to resist safely, the main loads that make up these combinations include: dead load, superimposed dead and live loads, along with creep, temperature effects and wind loading. This paper considers the design of the bridge under BS 5400, however for Highway Bridges; BD 37/88² can also be used.

For each critical load combination, the characteristic values of their effects are multiplied by partial load factors, they are \( \gamma_{fL} \) and \( \gamma_{f3} \). The values are taken from BS 5400, and \( \gamma_{fL} = 1,10 \) at the ULS for all analysis techniques, and 1,00 for SLS. Values for \( \gamma_{f3} \) varies depending on load combinations.

5.1 Load Combinations

To consider the impact of all the loading schemes, there are five different load combinations that are tested at ULS. These combine loading aspects such as wind and temperature effects, but for the purpose of structural analysis, this paper will only consider load combination 1 which deals with all permanent loads as well as the critical live traffic loads (HA and HB loading).

5.1 Dead and Superimposed Loading

To consider the impact of loading on the structure and the effect at the ULS, the permanent loads must be calculated. Dead loads refer to the principal structure of the bridge whilst the super-imposed dead loads refer to the surface finishes and services that are installed once the structure has been erected. In order to calculate the value of the permanent load, a logical cross section and material specification was used to estimate the value for the permanent load. The cross section of the deck used for the calculation is shown in Fig. 8.

The total factored permanent load of the deck was found to be 9,70MN, which has to be supported by the 22 steel cables. To help rationalise the loading scheme, I have assumed the even load distribution across the deck; therefore each cable support will take about 0.44MN vertically. The large steel arch itself weighs 1.6MN [4].

The cables should be post tensioned so they carry the dead weight of the deck, each cable is at a slightly different angle; therefore the resultant forces required for the pre-stress depends on which cable is analysed. Consequently, the worst case post-tension force is given by the 0.44 MN/sin54,2 = 0,543MN. For dead loads \( \gamma_{fl} = 1,05 \) at ULS, and \( \gamma_{fl} = 1,00 \) at SLS. Superimposed dead loads (negligible in comparison) are factored by \( \gamma_{fl} = 1,75 \) at ULS and \( \gamma_{fl} = 1,20 \) at SLS.

5.2 Vehicular Live Loading

Traffic loading is one of the principal load types that are incident on bridge structures and in order to calculate the load distribution from the traffic, according to ref [6], the number of notional lanes need to be defined. The number of notional lanes is dependent on the carriageway width and for this particular bridge, the carriageway width is 10,2m; this translates to three notional lanes which are each 3,4m wide.

The two types of traffic loading, HA and HB loading, are calculated dependent on the length of the bridge, number of notional lanes and other important design criteria. HA loading is a uniformly distributed load with either a knife edge load (KEL) or a single wheel load and HB loading constitutes a loading based on an assumed truck weight and wheel distribution.

5.2.1 HA Loading

Using the design criteria set out in Ref. [6], for a bridge with a loaded length of 52m, an equation can be used to find the nominal load over the notional lanes.

\[
W = 151 \left( \frac{1}{l} \right)^{0.475}
\]

\[
W = 151 \left( \frac{1}{52} \right)^{0.475} = 23,1kN/m
\]
Therefore per notional lane, the unfactored load is 23,1/3,4 = 6,79kN/m². The KEL will be taken as 120kN per notional lane.

5.2.2 HB Loading

According to Ref. [4], the Hulme Arch Bridge was designed for 35 units of HB loading, this equates to 87,5kN per wheel and there are sixteen wheels with four axles; this loading is applied to the bridge at a position where it creates the most onerous loading effect.

5.2.3 Traffic Loading Scheme

![Traffic Loading Scheme](image)

Table 2: Values for Live Nominal Traffic Loads

<table>
<thead>
<tr>
<th>HA UDL = 6,79kN/m²</th>
<th>HB LOADING = 35units</th>
<th>KEL = 120kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>γl = 1,10</td>
<td>γl = 1,25 for Loadcase 1.</td>
<td></td>
</tr>
</tbody>
</table>

5.3 Pedestrian Loading

The bridge has two pedestrian walkways on either side of the bridge, each walkway has an approximate width of 2,2m and using Eq. (2,3), a value for the nominal pedestrian live load can be calculated.

\[ k = \frac{HA \ UDL \times 10}{L + 270} \]

\[ k = \frac{23,1 \times 10}{52 + 270} = 0,72 \]

\[ W_p = k \times 5,0kN/m^2 \]

\[ W_p = 0,72 \times 5.0 = 3,6 \ kN/m^2 \]

5.4 Braking and Acceleration Forces

Braking and acceleration generates longitudinal forces along the bridge deck, and for HA loading, the horizontal force is taken as 8kN/m along a single notional lane along with a single 250kN force [6]. This generates a nominal longitudinal force of (8x52) + 250 = 666kN.

For HB loading, 25 % of the total nominal HB load is applied and it is equally distributed between the eight wheels of two axles of the vehicle, 1.8 m apart [6]. This generates a nominal longitudinal force of 43,8kN per wheel.

5.5 Compression in the Deck

The compression force in the deck is obtained by considering the horizontal component of the worst case cable force, acting at 54,2º to the deck. The total post tension force was 0,543MN, and so the compression force found using Eq. (4) can be used to analyse buckling effects.

\[ C = T \cos 54,2º \]
\[ C = 0,543 \times 10 \times \cos 54,2 = 0,32MN \]

5.6 Vehicle Collisions

Collision with the parapets is based on 25 units of HB loading, this roughly equates to a large articulated vehicle colliding with the parapet. There are large concrete bollards which prevent taller vehicles colliding with the cables and would therefore reduce the momentum of any collision with the parapet itself. A horizontal load of 150kN will act at a vertical height of 0,75m from the deck level and a longitudinal force of 50kN along the parapet.

The parapet design relies on plastic deformation of the barrier to take much of the force, and it results in a slender barrier which would have to be replaced after a collision.

6 Strength

From the loads calculated in section 5, a set of most onerous design criteria can be found for bending, torsion and shear effects, Fig. 10.

6.1 Bending

To calculate the maximum moment in the main span of the bridge, the deck is treated as a continuous beam with supports at either end; one end being fixed and the other being effectively pinned. To simulate the post-tension force in the cables, elastic directional supports are assumed along its length. The permanent load from the deck will induce small moments within the deck, but when compared with the magnitude of the moments from the live loads, they are fairly negligible; therefore only live loads are considered.

The deck is treated as a continuous beam of 52m clear span and due to the stiffness of the deck and its relatively small span; the effective length used for the calculation can be reduced. By taking moments from the pinned end of the bridge, a conservative value for the maximum bending moment can be found using Eq. (5).

The HB loading is situated in the location which gives it the most onerous loading effects, as to maximise the
hogging moment over the cable support. The maximum sagging moment isn’t as critical due to the short effective spans (4m centres).

\[
M = \frac{w (l_{eff})^2}{8} + \frac{P a^2 b (2l + b)}{2l^3}
\]

\[M = 5.49 + 1.51 \text{MNm}\]

\[M = 7 \text{MNm}\]

\[
\sigma_1 = \frac{M y}{I} = \frac{7000(900)}{9.14 \times 10^6} = 68.9 \text{N/mm}^2
\]

Fig 11: Loading and Moment Diagram

This is clearly not a true representation of the loading across the deck, the cable supports act as supports themselves and they can either be modeled as elastic or rigid supports. In reality, the bridge will indicate a series of curved bending moment diagrams, with the cable stays taking the majority of the dead load, and live load causing localised bending in the deck. To effectively model the elastic loading condition, computational analysis would be required with elastic directional supports located at the points where the cables meet the deck, Fig. 12.

Fig 12: Bending moment diagram for elastic supports

A conservative approach is to consider rigid supports at each cable location and for this particular arrangement the maximum moment can be found in the end span, Fig. 13. Consequently, it is fair to say that the true stress will be found in the region of 4.88N/mm² < \(\sigma_1\) < 68.9N/mm², which seems feasible for a project of this magnitude.

\[
M = \frac{w l^2}{10} + \frac{P l}{4}
\]

\[M = 28.9(4^2)/10 + (450)(4)/4\]

\[M = 496 \text{kNm} << 7 \text{MNm}\]

\[
\sigma_1 = \frac{M y}{I} = \frac{496(900)}{9.14 \times 10^6} = 4.88 \text{ N/mm}^2
\]

Influence lines have been drawn to check the worst case loading for the rigid support bending analysis.

Fig 13: Bending moment diagram for rigid supports

6.2 Torsion

The most onerous loadcase for torsional analysis would occur when the deck is asymmetrically loaded, such that two notional lanes are in full use whilst the other has no live load or limited live load (1/3 HA loading).

Fig 14: Most-onerous torsion loadcases

6.4 Buckling

Buckling is not that critical within the deck, due to the short effective spans between the cables and the relatively small overall span of the bridge. Any unwanted axial forces generated within the deck will be taken by the relatively strong steel sub-structure and due to its continuity, the forces would be transferred into the foundation and through the friction slab into the soil.

The compression value found in the deck from the horizontal component of the cable force equates to a stress of less than 1N/mm² over the cross-section of the deck; which wouldn’t cause any problems. A full computer analysis would have to be undertaken to ensure all loading effects are taken account of, for buckling checks.

6.5 Creep

Creep is a critical issue with the arch-to-foundation connection, as any relaxation at the bond interface will result in a reduction of the pre-stress within the cables this A reduction in pre-stress will compromise the structural integrity of the bridge as the deck will be force to act as a beam; to mitigate this problem all the bars are debonded from the concrete except from the lowest 300mm [4].

Creep within the deck could also pose a potential problem, as due to setting of the concrete, which continues for a long period of time and due to the
constant loading applied to the bridge in the form of the dead and super imposed dead loads, the bridge can ‘creep’ as time elapses.

7 Serviceability

The bridge would be typically monitored by a series of electronic devices that measure the critical aspects of the bridge; a good example of this is the presence of strain gauges within the cables. Measuring the pre-stress in the cables and ensuring that the force stays relatively constant, is imperative to the integrity of the structure.

Devices are also installed to measure the displacements, moments and stresses within the deck, foundation settlement and rotations, allowing the serviceability of the bridge to be closely monitored, ensuring any problems are mitigated as soon as possible.

8 Foundations

As the dead and live loads are distributed throughout the structure, it becomes apparent that the arch carries the majority of the permanent loading from the bridge; therefore it was the most onerous in terms of lateral and vertical loads experienced at the foundations. There are also abutment foundations at each end of the deck, resisting longitudinal loads generated by braking forces and lateral pressure exerted by the fill.

8.1 Geology

The site geology from the ground level downwards at the top of the cutting slopes goes as follows; the first 3-4m is made ground followed by 9-11m of boulder clay, which in turn sits on top of the strong bedrock, Bunter Sandstone. Borehole studies also indicated a shallow layer of glacial sand and gravel between the clay and the sandstone [4].

8.2 Arch Foundations

The arch is supported on a ground-bearing concrete foundation block at each springing; the dimensions of the blocks are 8.5m x 6.5m x 3.5m [4]. Ground-bearing foundation systems have greater resistance to arch splay and a fairly simple construction process.

The key loads from the arch are the horizontal thrusts and vertical reactions [8], generated from the steel cables supporting the permanent deck load. The horizontal forces are transferred through the concrete block and then through friction between the foundations soffit and the clay, Fig 15. The vertical loads are taken by the bearing capacity of the geology below the foundation, and there has also been considerable design consideration for differential settlement [4].

In terms of construction and the connection to the arch itself, the arch is anchored into the concrete foundation using 32 high-tensile stainless steel bars with a diameter of 40mm [4]. It is imperative that there is no tension at the interface of the concrete and the steel for any service limits state, due to the unfavourable properties of concrete under tension; therefore they are anchored deep within the foundations and designed to resist the excessive design stress.

8.3 Abutment Foundations

In order to resist the longitudinal loads generated by vehicles breaking and the lateral pressure of the fill, friction slabs were installed behind each abutment. The 200mm thick reinforced concrete slabs were located 2m below the ground level to allow for trenching and for future installation and maintenance of services [4]. The friction slabs are one of the key elements to the structural integrity of the bridge therefore duplex stainless steel bars were used for connection to the main foundation, as they exhibit good resistance to corrosion from salt ingress.

9 Construction

The construction of the Hulme Arch Bridge required efficient planning and management throughout, due to the logistical constraints of the existing transport network. The highway underneath the proposed bridge, Princess
Road is a commonly used link from Manchester Airport to the city centre and due to its popularity as a travel route a limited number of possessions were agreed within the contract [4]. Effectively, this means that any critical work that needed to be done using the access to Princess Road had to be well planned and executed perfectly as any delay would have cost the developer time and money.

Princess Road has an unusually wide central reservation which gave the option of fabrication of structural elements on site, but the most sensible option was off-site prefabrication. This mode of construction is better suited to this kind of project where there are logistical constraints and a limited timeframe, such that parts of the structure can be constructed off-site and simply delivered and craned into position.

The foundations are fairly central to the integrity of the structure; therefore it was important that during installation and initial operation the temperature of the material was carefully monitored to limit cracking. With over 500m$^3$ of concrete in both of the arch foundations, the temperature of the pour was limited to 70°C with a max differential temperature range of ±20°C [4]. Thermocouples were placed at key locations to take temperature readings over 10 weeks.

The foundations would have been installed during a single possession of Princess Road as access would have been required for pouring the concrete, excavating and transporting the fill. The high-strength steel bars that locate the arch within the foundation need to be fixed to prevent excessive moment within the concrete to prevent cracking and spalling, therefore a steel triangular frame was designed to be cast into the foundations and house the steel bars.

The composite concrete deck was installed during a single-possession of Princess Road, the elements of the deck were prefabricated as much as possible to reduce the work that had to be done on site and limit transportation costs. Once the elements of the deck had been delivered to the site, the deck was constructed into three 17m x 17m sections within the large central reservation, Fig 17. Temporary trestles to support the deck sections were installed in the reservation then each section of the deck was craned into position.

The first section to be installed would have been the west end as this part of the deck would be fixed, then the central section would have been installed on top of the temporary trestles, then finally the last section would have been installed, with accurate checks to ensure the tolerances for thermal expansion were acceptable. Once the deck had been made continuous the permanent formwork for the installation of the slab would be constructed.

The arch was fabricated in six equal sections about 15m long and then delivered on-site and welded together into two 80tonne halves [4]. The problem with installing large, heavy elements such as these, are that they have to be installed simultaneously to reduce the moments at the base, as the arch halves would act as cantilevers if installed separately. Consequently the arch halves were installed using a tandem lift, with two high capacity cranes holding the elements in position until a temporary connection had been made at the crown and connections had been fastened at both of the bases. Structural restraints that acted as bracing were installed on the arch to reduce the effects of wind loading whilst waiting for the next possession of Princess Road and the installation of the cables.

![Fig 18: Arch halves being positioned](image)

The steel cables were initially connected to the arch and to the outrigger brackets on the perimeter of the deck, then using the adjustable anchorages and a hydraulic jack system, the cables were tensioned simultaneously in 11 asymmetric pairs [4]. Computer analysis of all the critical loadcases had calculated stresses for all the cables, this would ensure that the structure could distribute more load than was required. Once the cables had been tensioned, accurate monitoring devices were installed to firstly check the values were correct and to monitor the stresses during operation.

Once the main structural elements had been installed, the surface finishing, paving, electrical and drainage services could be installed, these parts of the construction process didn’t rely on the possessions as they could all be done at deck level, leaving minimal impact on the transport route below.

The bridge is a single carriageway with 2 lanes, therefore the specifications for the road surface were fairly simple and the same applies to the pedestrian routes.

The bridge was completed in April 1997 and was officially opened on the 10 May 1997 [4]. The whole construction process took 11 months, and this is relatively quick considering the limited possessions of Princess Road, the general complexity associated with the design and consequently the high tolerances of the key structural elements. An enormous amount of credit must go to the contractors and the planners who had numerous constraints to allow for in their construction process, yet they managed to produce a visually stunning structure in a relatively short timescale.

**10 Temperature**

Diurnal temperature fluctuations are critical in bridge design; temperature increases within the whole structure and variations between the top and bottom surfaces induce stresses within in the structure. Temperature changes in the cables and the arch are likely to induce significant bending moments in both the deck and foundations.
For the purpose of design data, a 1:120 year temperature return period is used and it is assumed that the deck is entirely restrained; such that the movement joints would be faulty or jammed.

The coefficient of thermal expansion of steel and concrete is taken as 12×10^-6ºC. The design values for temperature change [4] were ±20ºC in the vertical direction. Eq. (10) gives the apparent compressive strain along the deck caused by these temperature changes, while the associated deflections are found in Eq. (11). The bridge will experience this movement as an apparent compressive stress in the section, Eq. (12).

\[ \varepsilon = (12 \times 10^{-6}) \times 20 \]  
\[ \varepsilon = 240 \mu \varepsilon \]  
\[ \delta = \varepsilon_c l \]  
\[ \delta = 240 \times 10^{-6} \times 52000 \]  
\[ \delta = 12.48 mm \]  
\[ \sigma_{c, \text{apparent}} = E \varepsilon_c \]  
\[ \sigma_{c, \text{apparent}} = 210000 \times 240 \times 10^{-6} \]  
\[ \sigma_{c, \text{apparent}} = 50.4 N/mm^2 \]

The final temperature effect to consider is the variation between the top and bottom surfaces, which is specified as the temperature difference. For calculation purposes, \( h = 250 mm \) with 100mm surfacing, the temperature profile is shown in Fig. 21, from Ref [6]. A temperature gradient across the deck will induce a moment in the deck.

The effect of temperature change within the steel cables is very important too; a change in temperature will reduce their load-carrying capacity. Each cable has a cross-sectional area of 2.042 mm^2, and taking the change in temperature again as 20ºC, the stress induced in the cable can be approximated, Eq. (13).

\[ F_{\text{cable}} = 12 \times 10^{-6} \times 20 \times 210000 \times A \]  
\[ F_{\text{cable}} = 50.4N/mm^2 \times 2.042 mm^2 = 103kN \]

However, due to the relatively small span and when coupled with a fairly high stiffness and the general continuity of the deck sub-structure, the effects of the cable slackening is reduced such that the deck should be able to cope with the forces generated.

11 Wind Effects

Wind loads on the bridge are analysed according to BS 5400 [6]. The pressure on the bridge deck is given by Eqs. (14,15,16).

\[ P_{L} = q A_{d} C_{d} \]  
\[ q = 0.613 \frac{W_{d}}{l} \]  
\[ V_{d} = (S_{b} T_{s} S'_{h}')(V_{b} S_{p} S_{a}) \]

The value \( V_{d} \) relates to the maximum hourly wind speed \( (V_{b}) \) experienced at the site which is then factored by a series of design criteria that adjust values according to local geography. The values of \( V_{b} \) taken from Ref [6] are hourly mean wind speeds with an annual probability of being exceeded of 0.02 (equivalent to a return period of 50 years) in flat open country at an altitude of 10 m above sea level.

### Table 3: Values for Eq. (14,15,16)

<table>
<thead>
<tr>
<th>( V_{b} )</th>
<th>( S_{p} )</th>
<th>( S_{d} )</th>
<th>( S_{a} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>22.2 m/s</td>
<td>1.05</td>
<td>1.01</td>
<td>1.00</td>
</tr>
<tr>
<td>1.21</td>
<td>1.00</td>
<td>1.00</td>
<td>28.5 m/s</td>
</tr>
<tr>
<td>497 N/m^2</td>
<td>93.6 m^2</td>
<td>1.07</td>
<td>50 kN</td>
</tr>
<tr>
<td>0.957 kN/m</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

11.1 Longitudinal Wind Loading

Longitudinal wind loading must also be considered and Ref. [6] states that longitudinal loading from wind loads \( (P_{LS}) \) and the effects of traffic on the bridge \( (P_{LL}) \) must be considered, Eqs. (17,18). The depth for the calculation of the coefficient of drag, \( d \), is taken as \( d = d1 + dL \), where \( dL = 2.5 m \). Thus \( d = 4.3 m \), Fig. 23.

\[ P_{LS} = 0.25 q A_{d} C_{d1} \]  
\[ P_{LL} = 0.5 q A_{d} C_{d2} \]

### Table 4: Values for Equation

<table>
<thead>
<tr>
<th>( q )</th>
<th>( A_{f} )</th>
<th>( C_{D1} )</th>
<th>( C_{D2} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>479 N/m^2</td>
<td>18.8 m^2</td>
<td>1.07</td>
<td>1.22</td>
</tr>
<tr>
<td>4.36 kN</td>
<td>0.23 kN/m</td>
<td>9.93 kN</td>
<td>0.53 kN/m</td>
</tr>
</tbody>
</table>

11.2 Nominal Vertical Wind Load

The final wind loading effect to be considered is wind uplift acting on the deck. The bridge has an exposed underside which can benefit from wind uplift, acting against the live and dead load forces. However, the force calculated in Table 5 is comparatively small; consequently it is safe to assume this force is taken by the deck alone.

\[ P_{V} = q A_{d} C_{L} \]
12 Natural Frequency

Many different types of structures have a unique natural frequency which is dependent on its general dimensions, including length and weight and determines how the bridge reacts at high and low frequencies. Frequencies below 5Hz are critical, as it puts the structure at risk from gusting winds and consequently the limit of the vertical acceleration of any part of the bridge has to be carefully considered. Structural collapse may occur if the natural frequency of the bridge is equal to that of gusting wind or mechanical (vehicle engine) or natural vibrations (pedestrians walking). At the higher frequencies, anything above 75Hz, the problem is associated with psychological effects as opposed to structural collapse. At these higher frequencies the bridge should oscillate more, tending to cause unease and nausea to users.

The fundamental natural frequency of the bridge is calculated using a simplified equation, Eq. 20 [7]. The mass part of the equation refers to all permanent loads associated with the bridge, so it ignores live-loading conditions. There are two separate criteria to check with the equation; clamp-clamp and clamp-pinned modes, and they must fall between in the range between 5Hz<ω<75Hz to avoid the problems mentioned above. The results are shown in Table… below.

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

(20)

<table>
<thead>
<tr>
<th>m=39x10^3kg/m</th>
<th>l=52m</th>
<th>E=200x10^3</th>
<th>I=9.14m^4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clamp-clamp:</td>
<td>(p_n)l^2 = 22.37 ⇒ ω_n = 56.6Hz</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Clamp-pin:</td>
<td>(p_n)l^2 = 15.42 ⇒ ω_n = 39.0Hz</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

13 Durability and Vandalism

The Hulme Arch Bridge is a visual focal point for the regeneration of the area, highlighting the importance of protection from vandalism. Before the regeneration scheme began in 1995, the area had been associated with high crime rates and excessive vandalism [1]. The prevention of vandalism is therefore imperative, as any vandalism could detract from the aesthetic impact.

The majority of the vandalism will be small scale, therefore posing little threat to the structure itself but maintenance and cleaning costs can increase with frequent attacks. The most exposed sections of the bridge are the deck soffit and the arch bases at either end; these would be targeted the most as they are the most accessible. The aluminium finishes of the arch should be fairly easy to clean and the concrete composite deck would be fairly simple to clean, maybe with a jet spray or strong industrial cleaner.

The durability of the steel cables is important due to their structural function, as they transfer the permanent loads from the deck to the arch. The cables are fabricated from 80 galvanized 5mm dia. steel cables which are spun into a single 51mm dia. cable that is protected by an aluminium flake coating applied during the fabrication process [4]. This should provide sufficient protection for the recommended design life of the bridge.

There is a requirement for regular inspection and maintenance of the structure. The bridge deck is fairly accessible, from the large central reservation below or from deck level, this ensures any maintenance issues can be solved without any major demands placed on the existing transport networks.

14 Future Changes

The bridge has no real provision for highway expansion due to the complex geometry of the arch and the careful orientation of the asymmetric cables; any major change in the deck structure will add to the permanent load, increasing the required pre-stress in the cables, as well as changing their positions.

There is provision for the installation of services above the friction slabs (<2m) and permanent fixings were also provided in the arch soffit to allow maintenance and inspection equipment to be installed in the future [4].

The only feasible change in use for the structure is if it became purely pedestrianised should the use of the area change, however due to its re-establishment as a key transport route, this is fairly unlikely.

15 Conclusion

This paper gives considerable insight into how the bridge pushes the boundary of engineering design and also how the socio-economic impact of the structure has helped reform the area of Hulme.

The Hulme Arch bridge is a ‘landmark’ structure for Hulme and by leaving it untouched and unobstructed for its design life, it will act as a reminder of Hulme’s new beginning and become synonymous with the regenerated area.

References