Abstract

This paper intends to describe the conceptual design of a Cable Stayed Bridge at Taunton, Somerset, United Kingdom. The development of the detailed design and the critical issues associated with the aerodynamic effects, fatigue effect on cable stays, layout of cable stays, deck and pylon of the cable stayed bridge have been briefly discussed. The design aimed to meet the Client’s requirement of minimising the aerodynamic effect on the structure and optimise the ratio of back span to main span and pylon inclination in order to minimise the uplift effect at supports.

1. Introduction

Project brief

Taunton (Somerset, UK) desires to develop a vibrant mix of employment, retail, housing, cultural and leisure facilities by 2021. It is in the process of developing its transport infrastructure to meet the needs and demand of its growing population whilst retaining its distinctive and valued market town character.

The key infrastructure needed in the short-medium term has been identified as the Taunton ‘Third Way’ which has evolved from the proposals for an Inner Relief Road, and the Taunton Northern Inner Distribution Road (NIDR).

The NIDR provides strategic access to Firepool and in conjunction with the Third Way would enable regeneration of the large areas of derelict brown field land adjacent to the River Tone which runs through the centre of Taunton.

Firepool is the industrial heart of Taunton and provides a strategic employment site within the town centre. It has the potential to attract a range of new businesses including government departments/relocations by virtue of its strategic transport links, and the quality of its riverfront environment.

The NIDR scheme includes a new bridge link over the River Tone and the Taunton and Bridgewater Canal. The bridge will mark the transition point into the new and expanded Taunton town centre. The bridge caters to the requirement of motorists, pedestrians and cyclists. This bridge is henceforth referred as the ‘Main Bridge’ in this paper.

As part of the above bridge scheme, it is also proposed to construct a landmark structure in the form of a pedestrian cable stayed footbridge along the river/canal connected to the Main Bridge.

Scope of Work

The scope of the project included the Preliminary Design of Pedestrian Footbridge. Specific requirements of geometric and structural design were obtained from the client in the form of a design basis note.

The construction of the proposed bridge is at an early stage of development and needs to be taken through a number of Statutory Procedures, which include Planning Consent and Land Acquisition, before construction works can commence. It is envisaged that construction work will commence by mid 2009 and will be completed by the end of 2010. The development of the scheme will be undertaken by Somerset Highways and the construction works will be the subject of a competitive tendering process, which will involve national road building contractors.
Location of the bridge

The proposed footbridge is located at Firepool, Taunton, Somerset, UK. It is located between the Rive Tone and Taunton/Bridgewater Canal, as shown in the schematic location plan in Figure 1.

The footbridge connects to a vehicular-cum-pedestrian river bridge (Main Bridge) at an elevated level and gradually descends to meet an existing road at grade, spanning over a virgin territory. The conceptual plan is shown below in Figure 2.

2. Design specifications

Design Standards

The preliminary design was based on the British Standards and Design Manual for Roads and Bridges (DMRB).

The footbridge was designed in accordance with BS 5400:Part 3 2000. The loading was taken from BS 5400:Part 2 2006, Highways Agency standard, BD 37/01 (DMRB Volume 1, Section 3, Part 14). The bridge was designed for footways and cycle track loadings.

“Aerodynamic Susceptibility Parameter” was determined in accordance with the Bridge Directives and Advises, BD 49/01 (DMRB Volume 1, Section 3, Part 3). Technical Guidance Notes on assessment of vibration behaviour of footbridges under pedestrian loading published by SETRA, France, were also referred to during the design process.

Relevant data and specifications

Geometric design

The terrain profile at the proposed site is flat with minimal variations in the ground levels. The longitudinal gradient of the structure was restricted to 5% as per Clause 6.4 of BD 29/04 (DMRB 2.2.8). The minimum headroom was restricted to 2.4m as per Clause 8.5 of BD 29/04 (DMRB 2.2.8).

The proposed minimum clearance under the structure was restricted to 2.4m.

Structural design

The span of the proposed bridge is 60m and it consists of a single deck with carriageway width of 3.5m.

The preliminary design aimed to achieve maximum ratio between the main span and the back span whist ensuring minimum or no uplift in the backstay anchorage and satisfying the aerodynamic stability requirements of BD 49/01 (DMRB 1.3.3).

The design evaluated the aesthetic and the structural feasibility for the following pylon heights - Option 1: 14m and Option 2: 24m. The heights of the pylon were specified by the client. Option 1 was rejected based on aesthetic considerations.

Aesthetic appearance

To reinforce the distinctive character of the proposed footbridge location and its future role, the County Council desired to construct a landmark structure in form of a cable stayed footbridge, which aesthetically blends with the surroundings.
Conceptual design of cable stayed pedestrian bridge at Taunton, Somerset

Wind and temperature

The preliminary design was based on a basic hourly mean wind speed of 22 m/sec (Vb). This value was obtained from Figure 2 of BD 37/01 (DMRB 1.3.14). The temperature loading was calculated based on maximum and minimum shade air temperature of 34°C and -18°C for a group two type superstructure. As per BS 5400: Part 2, group two structures include steel deck on steel truss or plate girder structures.

Soil condition

Soil investigations revealed the presence of a significantly thick made ground and variability of the superficial sand and gravel deposit. Based on the preliminary estimated design loads it was recommended that the bridge be supported on pile foundations. The recommended pile capacities for different socket lengths are enumerated in Table 1 of this paper.

Analysis methods

The static, dynamic and non-linear analysis of the structure was carried out using LUSAS 14.1 software. The structural model is shown in Figure 3. Natural frequency corresponding to the sum of the mass participation factor greater than 90% was evaluated to determine the aerodynamic stability parameters.

The cable elements were modelled as bar elements with rotational releases at each end. The modulus of elasticity was modified manually based on the Ernst Equation, taking into effect the centenary profile of the cable.

The aerodynamic susceptibility parameter (Pb) was calculated as per BD 49 using the following equation:

\[ P_b = \left( \frac{\rho b^2}{m} \right) \times \left( \frac{16V_r^2}{bLf_{nb}^2} \right) \]

Where:

\( \rho \) = Density of air
\( b \) = Total width of the structure
\( m \) = Mass per unit length of the bridge
\( V_r \) = Hourly mean wind speed
\( L \) = Length of relevant maximum span of the bridge
\( f_{nb} \) = Natural frequency of the bridge

Based on the value of aerodynamic susceptibility parameter (Pb), BD 49 classifies bridge structures in the following categories:

(a) Bridges designed to carry the loadings specified in BD 37 (DMRB 1.3), built of normal construction, are considered to be subject to insignificant effects in respect of all forms of aerodynamic excitation when \( P_b < 0.04 \).

(b) Bridges having \( 0.04 \leq P_b \leq 1.00 \) shall be considered to be within the scope of the rules specified in BD 49, and shall be considered adequate with regard to each potential type of excitation if they satisfy the relevant criteria given in the code.

(c) Bridges with \( P_b > 1.00 \) shall be considered to be potentially very susceptible to aerodynamic excitation, and shall be verified by means of further studies or through wind tunnel tests on scaled models.

<table>
<thead>
<tr>
<th>Pile Diameter</th>
<th>Socket Length</th>
<th>Negative Skin Friction</th>
<th>Allowable Working Load</th>
<th>Allowable Net Working Load</th>
</tr>
</thead>
<tbody>
<tr>
<td>m m kN kN kN kN</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.75</td>
<td>4</td>
<td>245</td>
<td>1470</td>
<td>1225</td>
</tr>
<tr>
<td>0.75</td>
<td>6</td>
<td>245</td>
<td>2206</td>
<td>1960</td>
</tr>
<tr>
<td>0.75</td>
<td>8</td>
<td>245</td>
<td>2940</td>
<td>2695</td>
</tr>
</tbody>
</table>

TABLE 1
GEOTECHNICAL RECOMMENDATIONS (PILE CAPACITY)

Figure 3
LUSAS MODEL
Aerodynamic Susceptibility Parameter \((P_b)\) was calculated based on the natural frequency derived from LUSAS analysis and was found to be less than the limiting value of 0.04 and hence the structure was considered to be subject to insignificant aerodynamic effects.

The analysis evaluated the load effects on the various elements of the structure during the Service and the Construction Stage. The procedures for replacement of cable and bearings were proposed to be considered at the detail design stage.

### 3. Preliminary design considerations

**Introduction**

The footbridge was planned as an asymmetrical cable stayed bridge with span of 60m and a steel girder deck of 550mm deep. The deck is supported from a 24m steel pylon by 5 pairs of high tensile galvanized cable system. Three pairs of cable supported the main span and two pairs of cable supported the back span.

The superstructure deck is continuous over pylon and simple supported over two abutments. The end stays on the main span are anchored in the abutment at the north end, whereas those on the back span are anchored in the ground at the south end to eliminate the uplift forces on abutment. The structure is restrained in the transverse direction at the pylon.

Steel was selected as the material for deck and pylon because of the benefit derived in terms of the ease in construction.

The following section aims to outline the various options considered in the conceptual design of the various key structural elements of the cable stayed bridge. The choice was made after careful deliberation on their relative merits and demerits.

#### Structural interaction between cables, deck and pylon

The basic load bearing elements of the cable stayed structure are the cables, deck and the pylon. The relative stiffness of the deck and the pylon and the number of cables determine the behaviour of the structure. The following options in Table 2 were considered in this regard.

<table>
<thead>
<tr>
<th>Options</th>
<th>Deck Stiffness</th>
<th>Pylon Stiffness</th>
<th>Cable Spacing</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Option - 1</td>
<td>Very Stiff</td>
<td>Slender</td>
<td>Relatively Large</td>
<td>Bridges built in the recent past with this option, have had higher construction cost, which makes them economically non feasible.</td>
</tr>
<tr>
<td>Option – 2</td>
<td>Slender</td>
<td>Stiff</td>
<td>Relatively small</td>
<td>This option is usually more feasible for multi-span bridges.</td>
</tr>
<tr>
<td>Option – 3</td>
<td>Slender</td>
<td>Slender</td>
<td>In this option the cables act as the determining stabilising element of the structure</td>
<td>This is the recommended option for the following reasons:</td>
</tr>
<tr>
<td>1.</td>
<td>Acceptable structural behaviour.</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.</td>
<td>It leads to a relatively slender deck and pylon.</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3.</td>
<td>The slender deck and pylon improve the aesthetic value of the structure.</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**TABLE 2**

**EVALUATION OF OPTIONS: STRUCTURAL BEHAVIOUR**
Superstructure

Transverse cable layout
The central suspension and the lateral suspension options were considered for the transverse cable layout. The deliberation on their relative merits and demerits is set out in Table 3.

Longitudinal cable layout
The Harp, Fan and Semi Harp layout options were considered for evaluation. The deliberation on their relative merits and demerits is set out in Table 4.

Cable spacing
Multiple cables are proposed to be used with the following advantages:

- The cables act as an elastic support for the deck, hence multiple cable increases the number of elastic supports leading to moderate longitudinal bending in the deck.
- Multiple cables result in lesser forces on the cables.
- Replacement of cable is relatively simple.

Deck
The following deck cross sections were evaluated to determine the most aerodynamically stable structure. The selection was based on reaching the fine balance between the stiffness and the weight of the superstructure.

- Steel plate box girder
- Steel I - beam with steel plate deck
- Steel I - beam with thin RCC deck slab

The proposed deck consists of steel plate resting on two universal I-beams (Figure 4). The steel plate deck was overlaid by 5 mm thick anti-skid and anti slip system. A universal steel beam (UB) was used as cross girders at 2m intervals, which contributes to the transverse rigidity of the structure. The transverse rigidity of the structure influences the vibration behaviour of the structure due to the wind loads.

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### Table 3
**Evaluation of Options: Transverse Cable Layout**

<table>
<thead>
<tr>
<th>Options</th>
<th>Remarks</th>
</tr>
</thead>
</table>
| **Option – 1** Central Suspension | **Merits:**
1. This option is aesthetically superior.  
**Demerits:**
1. The deck requires higher torsional rigidity and the bending stiffness of the deck is not exploited to its capacity. |
| **Option – 2** Lateral Suspension (Recommended option) | **Merits:**
1. This option provides improved stiffness and stability of the deck, when used with A-frame pylon.  
2. This option provides greater aerodynamic stability to the structure.  
**Demerits:**
1. Head room clearance may be restricted.  
2. Erecting A-frame pylon is generally more complicated. |

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### Table 4
**Evaluation of Options: Longitudinal Cable Layout**

<table>
<thead>
<tr>
<th>Options</th>
<th>Remarks</th>
</tr>
</thead>
</table>
| **Option – 1** Harp | **Merits:**
1. This option is aesthetically superior.  
**Demerits:**
1. This layout is not the best from the static or economic point of view. |
| **Option – 2** Fan | **Merits:**
1. This layout is advantageous from the static point of view.  
**Demerits:**
1. This option is aesthetically poor. |
| **Option – 3** Semi Harp | Semi Harp is an intermediate solution, between extremes of Harp and Fan patterns. This pattern combines the advantages of both these systems, whilst avoiding their disadvantages.  
Hence Semi Harp layout is recommended to be used. |
The geometric proportioning of the deck also ensured that it met the criteria laid down in the British Standards, as well as the Client’s directive, that no wind tunnel tests should be required for evaluation of the aerodynamic parameters at the preliminary design stage.

The serviceability requirement for the superstructure has been met by ensuring that the fundamental natural frequency of vibration exceeds 5Hz in the vertical direction for the bridge, without any live load and 1.5Hz in the horizontal direction for the bridge with live loading. The structure meets the requirements of vortex shedding, classical flutter and divergent amplitude in accordance with the British Standards and Bridge Directives of British Highways Agency.

The longitudinal view of the superstructure is shown in Figure 5.

**Cable stays**

Full locked coil strands made from hot dip galvanized high strength steel wires are proposed to be used for cable stays. The strand comprises an inner core of round wires made of one or more external layers of Z shaped wires. The Z shape of the wires is specially prepared in a self locking formation to give a compact section as shown below.

The typical properties of the wires are:

- Tensile strength: 1570 to 1600 MPa
- Proof stress: 1180 to 1245 MPa
- Elongation at breaks: 4% minimum on 250mm gauge length

The critical aspect of the performance of cable stays is their behaviour under fluctuating loads. In the proposed Firepool Bridge, the cable stay anchorages are attached to the web of the universal beam with a pinned connection, as shown in Figure 6.

Locked coil tendons have a fatigue performance in excess of 2 million cycles at a stress range of 150 N/mm² and with a maximum stress of 45% of the ultimate tensile stress. The stress levels in the cable were restricted to meet the above fatigue criteria.

**Substructure**

**Pylon**

The form and properties of the pylon in a cable stayed bridge are very important because it provides the main vertical resistance to the bridge. The appropriate form of the pylon depends on balancing the structural, maintenance, geometrical and aesthetic factors. Hollow pylon sections provide the opportunity for fixed access to the top from inside the pylon, but for smaller bridges providing sufficient room for access makes the external dimensions of the pylon out of proportion with the rest of the bridge. In this case the external dimensions of
the pylon have been evaluated based on the structural requirement.

The 24m height of the pylon was primarily based on aesthetic reasons. An inverted Y-shaped pylon was proposed to enhance the torsion stiffness provided by the cable system. Details of the pylon are shown in Figure 7.

The forces in the pylon due to other load effects were calculated by performing a plane frame analysis using LUSAS 14.1.

Foundation

The superstructure is supported on electrometric bearing resting on bank seat abutment. The pylon is supported by 750mm pile foundation.

4. Method of construction

The erection procedure has a very strong influence on the design of the cable stayed bridge. However in this case the proposed bridge site has a uniform elevation with minimal ground obstructions; hence the construction is proposed on a continuous staging over the length of the bridge.

Construction on continuous staging eliminates the requirement for construction stage analysis and hence the same has not been considered in the preliminary design.

5. Health & Safety and Environment

Construction Design and Management Regulation (2007), requires integrating health & safety issues into the design and management of projects. The focus is on the actions necessary to reduce and manage risks associated with health & safety and environment issues connected with construction and usage of structures. A preliminary designer risk assessment was prepared for the proposed bridge. The key aim of the report was to:

• Improve the planning and management from the very start of the project
• Identify hazards early on so that they can be eliminated or reduced at the design or planning stage and the remaining risks can be properly managed
• Target effort where it can do the most good in terms of health & safety
References

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2. niels J. Gimsing – Cable Supported Bridges, Concept and Design
3. Taunton Deane borough council – Delivering the Taunton Vision, Summary Leaflet