A CRITICAL ANALYSIS OF THE CAMPO VOLANTIN FOOTBRIDGE  
BILBAO, SPAIN  

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Abstract: This paper explores the aesthetics, structural performance and design issues of the Campo Volantin Footbridge, Bilbao. Particular emphasis is placed on the context of the bridge within a declining urban environment, highlighting the effect symbolic infrastructure can have on an urban or peripheral space. A simplified structural analysis is carried out for the arched span and its concrete abutments using British Standards for Steel, Concrete and Composite Materials. The practicalities of the Footbridge are also discussed, posing the question of whether the structure has been intelligently designed.

Keywords: Campo Volantin Footbridge, inclined arch, curved deck, symbolic infrastructure

1 Introduction

Santiago Calatrava, the Catalan born Architect and Civil Engineer is well known throughout the world for combining the knowledge and skills of his two professions to represent engineering as a useful art. Since establishing his offices in the 1980’s, he has created structures full of character, from his signature cable stayed bridges to railway stations, airports and skyscrapers. Beginning his career as a bridge engineer in 1984 with the Bach de Roda built for the Olympics in Barcelona, Calatrava has established his name by designing structures which prioritise form but have well defined function. He has frequently produced structures to signify occasions; the aforementioned Bach de Roda, the Puente del Alamillo for the 1992 World Exposition and the recently completed Jerusalem Chords Bridge in honour of Israel’s 60th anniversary. This reputation for designing significant and influential structures can be seen in the subject of this paper; the Puente del Campo Volantin or Campo Volantin Footbridge (see Fig. 1).

Known locally as ‘Zubizuri ’or ‘White Bridge’, the expressive footbridge was part of a multimillion pound plan to reinvigorate the city of Bilbao to coincide with its 700th anniversary; it explores the civic and social impact that significant infrastructure can have on a declining urban environment.

The Campo Volantin footbridge is situated approximately 900m upstream of Frank Gehry’s Guggenheim Museum, spanning 75 metres across the Nervión, a river which meanders through Bilbao on its way to the Bay of Biscay. The Bridge and the Guggenheim, along with Norman Foster’s underground line and Calatrava’s new airport terminal are all landmark features representing the regenerative ambitions of the Basque city.

The distinctive inclined arch structure of the Campo Volantin footbridge links the derelict Uribitarte warehouse zone on the Southwest bank with the lively river promenade of the Campo de Volantin on the opposite Northeast Bank (see Fig 2).
Its introduction was intended to symbolise the infrastructural improvements being made to the disused areas along the river left by the now obsolete maritime industry. Like the Trinity Bridge in Salford, the ‘white bridge’ has a civic agenda, the colour representing a revival and the form a city renewal, bridging a gap between two previously socially divided parts of Bilbao. It is wholly representative of Calatrava’s thoughts on peripheral and urban spaces [1].

The Campo Volantin footbridge, commissioned by the local authorities, was the second proposed design for the site, intended as a replacement for that designed as ‘Uribitarte Bridge’ [1] which came into trouble under its previous commissioners. The arch is very much a Calatrava speciality, whether braced (Bach de Roda), vertical (Lusitania Bridge), or as here inclined. Its form is easy to control structurally so can be repeatedly changed and adapted, often resulting in a hugely symbolic structure when dramatically presented and lit up. Completed in 1997, the Campo Volantin Footbridge is as much a sculpture as a functional structure with its lighting, like in the majority of the architects bridges, becoming a fundamental part of the footbridge as a whole.

2 Aesthetic Design

Fritz Leonhardt, in his book Brücken published in 1982 stated that there are 10 areas of aesthetics which need be considered when designing a bridge [2]. These are not however a set of guidelines which if seamlessly adhered to would produce a structure of outstanding aesthetic quality. Whilst many of the areas of aesthetic consideration can be seen in the bridges of Santiago Calatrava – in particular proportions, character and incorporation of nature – Leonhardt’s design approach greatly contrasts that of Calatrava in terms of the views on complexity and cost. Bridge engineers such as Fritz Leonhardt and Christian Menn advocate in their published literature a moralistic, purist approach where the most suitable structural system where cost is minimised and design simplified, will produce the best bridge. In contrast to this, Calatravas structures come at a high financial cost due to their high degree of structural complexity. However, despite this he is an extremely successful and in demand bridge designer, principally due to the public appeal of his structures and their iconic status. The Campo Volantin footbridge is certainly beautiful despite not being, as Menn put it, the “least expensive structural solution” [3]. The key point is whether what is considered as a structurally efficient design appeals to the public who use and pay for the bridge.

Three key features make this footbridge unique. Firstly, the strict directionality of a stereotypical bridge has been replaced by the curve displayed in plan (see Fig. 2), creating movement within the structure; a concept which is repeated throughout in the inclination of the arch and the flow of the stairways. Secondly, the slenderness of the arch (see Fig 4), which arguably makes the structure so expressive, indicates its shape is not arbitrary having been extensively and complexly analysed to produce a form which experiences minimal bending moments. This epitomises the approach Calatrava takes to his designs, finding a structurally efficient form whilst delivering an exceptional aesthetic quality. Thirdly, and most distinctively unique, the deck and the arch do not lie above one another in plan (see Fig. 2) so that the longer set of steel cables pass over the deck, installing a sense of security. An open tent-like feeling is created within the bridge, which whilst being light and permeable, creates a stable three-dimensional space.

A principal requirement of any bridge is fulfilment of function, carrying the load between the two supports. It is clear to see how the Campo Volantin footbridge does this and the structure is clearly defined as a deck suspended from a steel arch by steel cables. The steel, and in fact all of the materials used are
exposed so that their structural purpose is clear. It is also clear that the deck provides a tie for the arch, demonstrated by the slenderness of the arch and the abutments. These appear to offer no horizontal restraint showing that the force which resists spreading of the arch is provided solely by the deck. The shape of the arch is both aesthetically and structurally ideal, minimising its size.

Functionally and structurally the bridge is well proportioned and refined. It relies primarily on the balance between solid and void, displaying well-proportioned amounts of both solidity and delicacy. The superstructure draws the eye from the arch down to the abutment where there exists a void where a support would conventionally be.

forces would be taken from the arch directly through a solid embankment and into the ground. In Bilbao however, the bridge is supported on pre-cast concrete cantilevered sections rising up from the river bank, replacing the solidity of a typical solid embankment with an empty space. This creates an interesting effect, the void complimenting the openness of the bridge structure and adds to the directionality expressed by the arch and deck, dictated by the flow of the river. Furthermore, the sculptural, half arch cantilevered supports are well refined and structurally efficient, tapering towards their end whilst resisting a diminishing Bending Moment.

Calatrava has always placed a particular emphasis on stairways (see Fig. 6). Attention is taken away for the stairways purpose and focus is given to them providing a sense of continuity and synchronicity between the bridge and the ground. The solid/void balance is enhanced by the lightness of the structure, achieved primarily by the glass decking. The translucent deck shows the bare bones of the supporting structure making it clear how the load is carried and enhancing the shallow deck, which like the rest of the structure appears to be very delicate and as such very expressive.

The entire structure has an obvious connection with nature, the basic composition of steel and glass representing a skeletal framework (see Fig. 7). The panels of translucent glass are supported by narrow stainless steel ribs resembling the backbone of a fish; a nice effect for a structure which spans a river, especially considering the reflection in the water. This is particularly effective due to the bridges lighting, as the white structure is and translucent deck are illuminated. This accentuates the symbolic appeal of the bridge making it a beacon within the area.

The complexity of the curves in the structure give the Campo Volantin a stronger presence than other bridges of such a span, something which is useful due to the structures proximity to Frank Gehry’s Guggenheim. All connections appear to be minimally elegant, especially the cables which connect simply and seamlessly with the arch and perimeter circular hollow sections (see Fig 8).
The Romanian sculptor Constantin Brancusi said “the beautiful is the absolute balanced” [1] – this can undoubtedly be applied to the Campo Volantin Footbridge.

2 Structural Design

Simplistically, the bridge is formed around the two opposing curves of the supporting cradle and the deck (which has a slightly tighter radius), and the supporting, 15.3 metre deep arch. The arch is a distorted parabola, tied by the deck so it resists spreading at the abutments and is inclined at an angle of 80 degrees to the horizontal. The 75 metre spanning bridge deck, with a varying width of 7.5 metres at mid-span and 6.5 meters at its ends, is flanked on either side by 39 steel cables, which appear seamlessly attached to the arch at 1.8 metre intervals [3]. The curve of the deck is for more than just visual effect; it counters the torsion induced by the cables due to the unevenness of the load supplied by the arch by curving in the opposite direction to the supporting cradle, thereby transferring the load through the concrete abutments to concrete foundations. On a global scale, torsional effects are balanced within the deck due to the surface area of the deck on one side of the central circular hollow section equaling that on the other side in surface area [4]. The same torsional forces are therefore induced by each side of the deck, so in the centre of the deck and at the ends forces are perfectly balanced. To stop the entire superstructure from rotating about the end connections, the principal beam in the supporting cradle is fixed to the each abutment by welding it through an end plate (see Fig. 9)

4 Loading

For the analysis of a structure the loads incident on it need be identified. As the Campo Volantin footbridge is located in Bilbao, it will have been designed to the loading criteria specified in Spanish Norma IAP-98. For simplicity however this assessment will be carried out to British Standards [5], which follows similar principles.

Section 4.4 of [5] indicates the different loading combinations that need be considered during bridge design. The limited information available on the structure of the footbridge and the construction process mean that erection loads, secondary live loads and friction at supports would not be assessable; as a consequence only combinations 1, 2 and 3 are considered within this report.

4.1 Dead and Superimposed Loads

4.1.1 Dead

This includes the self-weight of the bridge and its structure. These values have been calculated based on the dimensions given in technical drawings [1] and the densities given in [6].

Deck steel: $8.22 \text{m}^3 \times 7850 = 64527 \text{ kg} = 645.3 \text{ kN}$

Deck glass: $18 \text{ m}^3 \times 2500 = 45000 \text{ kg} = 450 \text{ kN}$

Deck total: $1133.7 \text{ kN} = 14.6 \text{ kN/m}^2$

Arch: $5.43 \text{ m}^3 \times 7850 = 42626 \text{ kg} = 426.3 \text{ kN}$

Hangers: $0.8 \text{ m}^3 \times 7850 = 6280 \text{ kg} = 62.8 \text{ kN}$

Arch total: $489.1 \text{ kN} = 6.52 \text{ kN/m}^2$

4.1.2 Superimposed Dead

This is all permanent loads that are non-structural. Due to the material simplicity of this design, this only includes the parapet and lighting.

Parapet: $4.5 \text{ m}^3 \times 7850 = 35325 \text{ kg} = 353.3 \text{ kN}$

Lighting: 80 units at 2kg = 160kg = 1.6 kN
Total: 354.9 kN = 5.00 kN/m

There is a large safety factor [5] for these loads due to the likelihood of conventional superimposed loads – road surfacing and services – to vary significantly over the bridges life time. For the Campo Volantin Footbridge this safety factor would be largely unnecessary, however as the value is an estimation and the exact weight is not known accurately, here the safety factor will still apply.

4.2 Live loading

Clause 7 of [5] states that for loaded lengths in excess of 36m the standard pedestrian loading of 5kN/m² can be reduced by a factor k. The loading is calculated using Eq. 1, where k is given by Eq. 2. Crowd loading does not need to be considered as the bridge is unlikely to experience large volumes of people.

\[ w_{sed} = 5.0k \]  
(1)

\[ k = \frac{\text{nominal HA UDL for loaded length}}{1+270} \times 10 \]  
(2)

HA UDL = 36 \times \frac{1}{75^{1.1}} = 23.377

\[ k = \frac{23.377 \times 10}{75 + 270} = 0.6776 \]

\[ w = 5.0 \times 0.6776 = 3.38 \text{ kN/m}^2 = 13.5 \text{ kN/m} \]

Vehicular loading needn’t be considered; the bridge is intended for pedestrian use only and there is no chance of accidental vehicle loading.

4.3 Wind Loading

The bridge has been analysed for wind loading using section 5.3 of [5] and [6]. To calculate the wind loading the wind speed must be known. The average wind speed based on observations taken at Bilbao Airport between October 2000 and March 2011 [7] give a steady wind speed of 7 knots, equivalent to 3.6 m/s. Multiplying this average monthly wind speed by a factor of 5 should give a conservative estimate of a 1/120 year event. This factor must be conservative as the prevalent wind in Bilbao comes from the NW and the bridge is orientated facing that direction.

\[ \text{Mean hourly wind speed} = 3.6 \times 5 = 18 \text{ m/s} \]

The maximum wind gust speed \( v_c \) must first be calculated using Eq. 4. Section 5.4 in [6] sets out a procedure from which the maximum wind gust speed can be obtained.

\[ v_c = v \cdot K_1 \cdot S_1 \cdot S_2 \]  
(3)

The values used are in table 1

| \( K_1 \) | Wind Coefficient |
| \( S_1 \) | Funnel Factor |
| \( S_2 \) | Gust Factor |
| \( v \) | Wind Speed (m/s) |
|---|---|---|---|---|
| 1.48 | 1.00 | 1.00 | 18 |

\[ v_c = 18 \times 1.48 \times 1.00 \times 1.00 = 26.6 \text{ m/s} \]

As it is a footbridge, this value may be reduced by a factor of 0.8 (6).

\[ v_c = 26.6 \times 0.8 = 21.3 \text{ m/s} \]

4.3.1 Horizontal wind load

This value, \( P_t \), can be calculated using Eq. 4 from section 5.3.3 of [6]. The value of \( q \), the dynamic pressure head, is calculated from eq (5)

\[ P_t = q \cdot A_1 \cdot C_D \]  
(4)

\[ q = 0.613 \cdot v_c^2 \]  
(5)

The values used are in table 2.

| \( A_1 \) | Solid horizontal projected area m² |
| \( C_D \) | Drag coefficient |
|---|---|---|
| 207.2 | 2.0 |

\[ q = 0.613 \times 21.3^2 = 278.1 \text{ N/m}^2 \]

\[ P_t = 278.1 \times 207.2 \times 2.0 = 115.2 \text{ kN} \]

\[ P_t = 1.54 \text{ kN/m} \]

4.3.2 Vertical Wind Loading

It is also important to consider the action of uplift and vertical down force, using Eq. 6 and values in Table 3.

\[ P_z = q \cdot A_3 \cdot C_L \]  
(6)

| \( A_3 \) | Solid horizontal projected area m² |
| \( C_L \) | Lift coefficient |
|---|---|---|
| 457.2 | +/- 0.75 |

The lift coefficient is higher than usual as the deck has a 2% gradient, giving an angle of super elevation of
1.14⁰. As this value is > 1 a higher coefficient is used [5].

\[ q = 0.613 \times 21.3^2 = 278.1 \text{N/m}^2 \]

\[ P_{cl} = 278.1 \times 457.2 \times 0.75 = \frac{+95.3}{-95.3} \text{kN} \]

\[ P_{c} = 1.27 \text{kN/m} \]

### 4.3.3 Longitudinal Wind Load

This is the sum of the nominal longitudinal wind load on the bridge \( P_{LS} \) and the nominal longitudinal wind load on the live load, \( P_{LL} \). It is calculated using Eq. 7.

\[ P_{L} = P_{LS} + P_{LL} \]  \hspace{1cm} (7)

\[ P_{LS} = 0.25 \times q \times A_{11} \times C_{D} \]  \hspace{1cm} (8)

\[ P_{LL} = 0.5 \times q \times A_{11} \times C_{D} \]  \hspace{1cm} (9)

The values used are in table 4.

| \( A_{1} \) | Solid horizontal projected area \( \text{m}^2 \) | 3.04 |
| \( C_{D} \) | Drag coefficient | 2.0 |
| \( A_{11} \) | Projected area of live load \( \text{m}^2 \) | 5.0 |

\[ P_{LS} = 0.25 \times 278.1 \times 3.04 \times 2 = 422.7 \text{N} \]

\[ = 0.42 \text{kN} \]

\[ P_{LL} = 0.5 \times 278.1 \times 5.0 \times 2.0 = 1390.5 \text{N} \]

\[ = 1.39 \text{kN} \]

\[ P_{L} = 0.42 + 1.39 = 1.81 \text{kN} \]

### 4.3.4 Wind Loading Combination

The wind loads \( P_{L} \), \( P_{L} \) and \( P_{V} \) shall be considered in four separate combination cases, as specified in clause 5.3.6 of [5] and shown in table 5.

<table>
<thead>
<tr>
<th>Case</th>
<th>Combination</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>a)</td>
<td>( P_{L} ) alone</td>
<td>115.2 kN</td>
</tr>
<tr>
<td>b)</td>
<td>( P_{L} ) +/- ( P_{V} )</td>
<td>210.5 kN</td>
</tr>
<tr>
<td>c)</td>
<td>( P_{L} ) alone</td>
<td>1.81 kN</td>
</tr>
<tr>
<td>d)</td>
<td>0.5( P_{L} ) +/- 0.5 ( P_{V} )</td>
<td>107.1 kN</td>
</tr>
</tbody>
</table>

Table 5

It is evident that combination b) is dominant. The unfactored wind loading is therefore 210.5 kN or 2.81 kN/m

### 4.4 Temperature Loading

A change in the temperature of an object can induce strains, and as a consequence stresses within an object. There are two ways in which temperature differences can cause stress within a bridge, as specified in section 5.4 of [5]. These are:

- Changes in the overall effective temperature of the bridge superstructure
- Differences in temperature between the top and bottom surfaces.

There is the potential for large temperature variations in Spain. As the arch is tied by the deck, there is restraint of associated expansion or contraction, known as temperature restraint [5]. This can cause large stresses within the superstructure.

#### 4.4.1 Effective Temperature

Footbridges are designed to a 50 year design period so are subject to reduced maximum and minimum temperature. However, temperature information is only available for the past two decades, so this paper will take these values as they are. The highest temperature was experienced in August (46 C) and the lowest in February (-8). From tables 10 and 11 in chapter 5.4 of [5], assuming the bridge behaves similarly to those in group two, the assumed and effective temperatures can be derived and are shown in table 6.

<table>
<thead>
<tr>
<th>Assumed Air Shade Temperature</th>
<th>Effective Bridge Temperature</th>
</tr>
</thead>
<tbody>
<tr>
<td>Minimum -8</td>
<td>Minimum -9</td>
</tr>
<tr>
<td>Maximum + 38</td>
<td>Maximum + 48</td>
</tr>
</tbody>
</table>

Table 6

As the bridge opened in May 1997, this paper assumes the bridge was placed on its supports in Spring, at a temperature of 18⁰C.

The maximum temperature change is therefore +30 C or -27 C. Taking \( a = 12 \times 10^{-6} \), Eq. 10 gives strain, \( \varepsilon \).

\[ \varepsilon = a \times \Delta T \]

\[ \varepsilon = 12 \times 10^{-6} \times (+30) \]

\[ \varepsilon = 12 \times 10^{-6} \times (-27) \]

This gives a strain of +360 \( \mu \varepsilon \) or -324 \( \mu \varepsilon \) leading to the extensions below.

Deck, length 75m = +27.0mm/-24.3mm

Cables, max length 15m = +5.4 mm/ -4.9 mm
The maximum deck extension is therefore 0.027 m and easily accommodated for with an expansion joint. If expansion or contraction is restricted:

\[ \sigma = \varepsilon \times E \]  
\[ \sigma_{\text{max}} = 3060 \times 10^{-6} \times 200000 = 72 \, \text{N/mm}^2 \]

The deck restraining the arch during its expansion will increase the tension within the cables as it tried to expand outwards, increasing the stress within the deck where the cables are attached.

### 4.4.2 Temperature Differences Across the Deck

As the structure is so light, the deck so shallow and decking translucent there is unlikely to be a temperature difference between the top and the bottom of the deck.

### 4.5 Natural Frequency

It is important to design footbridges which don’t vibrate excessively. The Natural Frequency should lie within the range of 5 Hz < f_o < 75 Hz [5].

\[ f_o = (\beta_n l)^2 \times \sqrt{\frac{EI}{mL^4}} \]  
\[ f_o = 5.74 \, \text{Hz} \]

This value is just acceptable.

### 4.6 Design Loads

Having assessed the nominal loads experienced by the bridge, final design loads can be calculated by applying a safety factor of which there are two types. These are the partial load factor \( \gamma_f \) and the inaccuracy factor \( \gamma_f \).

<table>
<thead>
<tr>
<th>LOAD Component</th>
<th>ULS (kN/m)</th>
<th>SLS (kN/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dead</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Deck</td>
<td>15.3</td>
<td>14.6</td>
</tr>
<tr>
<td>Arch</td>
<td>6.85</td>
<td>6.52</td>
</tr>
<tr>
<td>Total</td>
<td>23.2</td>
<td>21.1</td>
</tr>
<tr>
<td>Superimposed Dead</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Total</td>
<td>6.00</td>
<td>5.00</td>
</tr>
<tr>
<td>Pedestrian and Cycle</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Vertical + Horizontal (b)</td>
<td>20.3</td>
<td>14.9</td>
</tr>
<tr>
<td>Cycle</td>
<td>16.9</td>
<td>13.5</td>
</tr>
<tr>
<td>Temperature Expansion restraint</td>
<td>3.1</td>
<td>2.81</td>
</tr>
</tbody>
</table>

### 4.6.1 Combinations

Combination 1

The permanent loads and primary live loads

\[ w_{\text{design}} = 23.2 + 20.3 = 43.5 \, \text{kN/m} \]  

\[ \text{(14)} \]
Combination 2
The loads from combination 1 plus wind loads
\[ w_{\text{design}} = 43.5 + 3.1 = 46.6 \text{ kN/m} \]  \hspace{1cm} (15)

Combination 3
The loads from combination 1 plus temperature loads
\[ w_{\text{design}} = 43.5 + 5.12 = 48.6 \text{ kN/m} \]  \hspace{1cm} (16)

Combinations 4 and 5 have not been considered within this paper.

5 Structural Analyses

5.1 Bending moment in Arch

The ULS design loads from combination 1 in Eq. 14 have been used to calculate the maximum bending moment, with the live loads applied to one half of the arch shown in (see Fig 11).

\[ \sigma = \frac{My}{I} \]  \hspace{1cm} (18)

\[ \sigma = \frac{1963 \times 10^3 \times 0.457}{\pi \times (\frac{0.457}{2})^3 \times 0.05} = 239 \text{ N/mm}^2 \]

The factored design strength is likely to be 365 N/mm² so this stress is acceptable.

5.2 Bending moment in the deck.

To simplify the calculation, the bending will be calculated by considering the main circular hollow section on its own, treating the deck as a simply supported beam, (see Fig 5).

From section 4.6.1 it is clear that combination 3 gives the largest design loads.

\[ M = \frac{wi^2}{8} \]  \hspace{1cm} (19)

\[ M_{\text{max}} = \frac{48.6 \times 1.8^2}{8} = 19.7 \text{ kNm} \]
\[ \sigma = \frac{My}{I} \]
\[ \sigma = \frac{19.7 \times 10^3 \times (\frac{0.610}{2})}{\pi \times (\frac{0.610}{2})^3 \times 0.03} = 2.25 \text{ N/mm}^2 \]

This value is low as the cables are positioned at small intervals, generating small moments within the deck.

5.3 Bending moment in the abutment

Modelled as a simple cantilever.

\[ M = wi \]  \hspace{1cm} (20)

\[ w = 48.6 \times \left(\frac{75}{2}\right) = 1823 \text{ kN} \]
\[ M = 1823 \times 3 = 5469 \text{ kNm} \]

This is the bending moment experienced at the base of the abutment.
5.4 Cables

Under the assumption that the cables transfer all live load and the dead weight of the deck, parapets and lighting to the arch, the axial force in the cables can be calculated. This also takes into account stress induced by temperature change.

From diagrams it has been assumed the cables are separated at equal intervals of 1.8m [1].

\[
Axial\ force = \gamma_f \times \frac{W}{2} \times 1.8
\]

\[\text{Axial force} = 1.1 \times \frac{48.6}{2} \times 1.8 = 48.1 \text{ kN}\]

\[
\sigma = \frac{F}{A} = \frac{48100}{(\pi \times (\frac{12}{2})^2)} = 50.0 \text{ N/mm}^2
\]

This is well below the design strength of steel.

6 Construction

Tied arches, due to the horizontal thrust being contained within the structure by the deck can be prefabricated off site and then lifted into position. This is advantageous as some erection loading conditions, which often result in the most adverse combinations, needn’t be considered. Information regarding the construction of the Campo Volantin footbridge is limited; this paper considers potential likely construction methods based on similar steel bridges, such as the York Millennium Bridge [9].

The sections of the bridge components; cradle structure, arch, structural glass decking and cables will have been delivered to site individually. Temporary frame work will have supported the arch and deck, which were erected in pre-set positions within the frame. The arch and central circular hollow section would then have been welded together (see Fig. 1) and the arch released to take its shape.

The cables would then be attached and individually hand tightened through access holes in the perimeter circular hollow sections (see Fig 14). The deck would then be released, allowing the steel cables to take up the load from the deck in tension. Once the skeleton of the structure had been erected, the structural glass deck would be added in sections and bolted, or more likely welded onto the main frame.

The supports are likely to have been cast off site, post tensioned, delivered to site and placed above their small foundation. The superstructure could have been positioned in a number of ways. One option is for it to have been launched across the river on barges and positioned onto its supports. Another would have been to use cranes to lift the main span into position, which would have enabled the river to be open sooner than if barges had launched it. However, due to the span of the bridge (75 meters) and its weight, a very large crane would have been required, adding to the cost of the project. The superstructure would also have had to be designed with high strength lifting points; difficult to incorporate into such a light and slender structure.

7 Foundations

The tie in the arch means that the only significant loads the abutments resist are vertical, from the weight of the structure bearing down on them. Due to the site being either side of a river and on the coast, the ground conditions would prove unfavourable and pile foundations would be necessary. Due to the support being cantilevered, the foundation would need to be designed to sustain moments, meaning a bigger foundation. This is a lot more expensive; however the alternative would be a structure which exhibited none of the elegance and continuity that is currently exhibited by the Campo Volantin footbridge.

8 Durability and Vandalism

The bridge was only opened in 1997 so it’s durability has not withstood a significant test. As the structure is steel, corrosion could be a potential problem. The steel would have multiple layers of corrosion protection; as long as this coat remains and is regularly maintained steel corrosion should not be a significant problem. Another issue is the monitoring and maintenance of the arch cables, which should be regularly monitored. There is evidence that this however has not been the case (see Fig 14). A structure as iconic as the Campo Volantin Footbridge would need to be regularly maintained to retain its symbolic status. The ‘white bridge’ however is not currently living up to its name (see Fig. 16.)
9 Improvements and Future Changes

Fundamentally, a bridge is a functional piece of infrastructure. Its primary requirement is to be usable and provide a safe crossing point—arguably Zubizuri does not deliver this. It is obvious that when designing this bridge the main focus was architectural effect; usability seems to have been an afterthought and somewhat neglected. The choice of structural glass for the decking was undoubtedly a mistake, and although it is a perfect choice visually, adding to the ephemeral quality of the structure, there is a reason that structural glass is not a popular choice for an external decking material, especially in a rainy city such as Bilbao. When wet the surface becomes slippery, impractical and unusable. To make the bridge safe black strips of high grip material have been stuck on the deck at regular intervals. Although this is a necessary addition, it detracts from the simplicity of the structure which is so well visually balanced. The brittleness of glass has also caused problems as the curved panels cannot resist the stresses induced by loading and shatter. To date the local authorities have had to pay thousands of Euros replacing the glass tiles as each panel is a different shape and must be specially ordered and manufactured, a process which is neither cheap nor fast.

At the 1992 Royal Fine Art Commission Santiago Calatrava stated that “when we discuss the aesthetics of something that primarily has to be stable, we can only link beauty and stability with intelligence” [10]. Neglecting to consider two fundamental properties of glass when assessing its suitability as decking for the Campo Volantin Footbridge is certainly not intelligent design. Furthermore, it is clear that the wet climate present in Bilbao has also been overlooked, to the detriment of the people of Bilbao and Calatrava’s reputation. Although an iconic symbol of urban renewal, little attention has been paid to context, the local area or history.

References


Bibliography


Tzonis A. Calatrava Bridges. Thames and Hudson, 2006