MANUSCRIPT GUIDELINES FOR THE BRIDGE ENGINEERING 2 CONFERENCE

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Abstract: This sample article provides information and instructions for preparing in a standard format the papers to be included in the Proceedings of Bridge Engineering 2 Conference 2010. It illustrates the paper layout and describes points you should notice before you prepare your paper. In order to enable the publisher to

Keywords: Metsovitikos roadbridge, Egnatia highway, prestressed concrete, cantilever construction

1 General Introduction & History

The Egnatia highway spans Northern Greece for 687 km and links Igoumenitsa to the Turkish border, greatly improving passageways across the country. Egnatia Odos AE, the government company in charge of delivering the road, seized the opportunity to create a dramatic structure within the Pindos mountains, making the Metsovitikos Bridge one of the roads two landmark structures. The bridge spans 565m in total, reaching an impressive height of 150m above the river.

Originally, a ‘pier-less suspension bridge’ developed by Ove Arup & Partners, using cable anchorages, was selected as the winning design. The concept was to make a powerful statement with minimal intervention. Although situated in an area of moderate seismic activity, the natural flexibility meant only the anchored foundations were designed with seismic considerations. This elegant design was abandoned when after detailed investigation into the rock anchors, the seismic slope stability was not deemed sufficient. [6]. The subsequent balanced cantilever design made use of technology well understood in the area, to construct the longest spanning bridge of its kind in the Balkans.

2 Aesthetics

The aesthetics of bridge design are very subjective, it is important to remember the perspective from which the user will view the bridge. In the case of a highway bridges such as the Metsovitikos, its main viewers are those who live in the nearby town and use the local roads in the valley. It could therefore be argued that its shape and presence in the valley have more aesthetic importance than its finer details.

2.1 Leonhardt Fritz Analysis

Authors such as Leonhardt Fritz have attempted to compile aesthetic guidelines by which a bridge can be designed or assessed. His ten rules, collated in Brücken [7] will aid an aesthetic analysis of the Metsovitikos. The designers, recognizing the natural beauty of the surrounding area, aimed to create an impressive structure that adorned the mountain landscape; increasing its beauty and not subtracting from it.

2.1.1 Fulfillment of function

As well as fulfilling its function of spanning between the two tunnels and connecting the Egnatia highway, the bridge should clearly present to the user the method by which it does so. [8] The structural strategy in its simple design is instantly apparent; both the beam and column elements contribute successfully to the overall design and no unnecessary elements are added. The consequence for the user is an impression of the bridges’ stability. Appropriate dimensions are used for the decks and columns, further promoting a perception of stability. The deck is divided in two, each supported by separate columns, effectively creating two individual bridges with near identical profiles.

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2.1.2 Proportions

The proportions of a structure can greatly affect the impression conveyed; there should be a balance between solids and voids. The Metsovitikos responds to the shape of the valley by altering each span with the varying valley depth, maintaining similar aspect ratios for each area enclosed by the deck, piers and columns. A geometric balance is also maintained as the depth decreases away from the two main piers at the same curvature. This decrease in depth towards the centre of the span displays more of the bridge’s structural strategy and construction method; the smallest depths showing the areas with the lowest bending moments.

2.1.3 Order

The number of lines and edges are minimal. Although each pier varies in cross section and design, due to their small number and the relatively large separation, this is hardly noticeable. As already noted, certain patterns can be observed in the bridge’s design, the proportion of the voids is maintained along with the depth’s curvature from each of the two main piers.

2.1.4 Refinements

It should be noted that the soffit remains at a constant depth across the bridge; the admirer’s eye is then drawn towards the smaller soffit depth, increasing the perceived slenderness of the bridge. The desire for voids of a constant proportion has lead to larger spans at the centre of the valley; the deck depth then decreases towards the tunnel entrances, where smaller spans mean it is not as necessary. The most noticeable central pier is tapered inwards from the base, strengthening the area of largest moment and further enforcing a sense of stability. The split deck design has lead to two identical columns at each support, the large separating spans allows opaque views to be avoided.

2.1.5 Integration into the Environment

As discussed, the bridge appears elegant and slender in its design; refinement and sensible structural proportions are used throughout to produce a design that although unsubtle, graces the landscape, creating a suitable connection between the cliff faces.

2.1.6 Colour and Texture

The Metsovitikos appropriately uses concrete as the only significant building material to further integrate into the environment; the material’s texture and colour pick out similar materials in the cliff face and the local roads winding through the valley. The deck profile is offset from the soffit; causing a shadow to be cast on the deck profile beneath, the observer’s eye is then drawn to the thinner fascia, emphasizing the deck’s slenderness. The column’s apparent slenderness is enhanced by the shadow cast on the corner cut square cross sections; the eye is drawn only to the thinner front face.

2.1.7 Character

At first glance, the Metsovitikos may seem to lack character, easily compared to many other bridges around the world, especially others on the Egnatia highway. The bridge designers set out to interpret the figures of a pair of dancers, moving through the valley; the more slender column representing the ballerina, holding the hand of the taller, stronger dancing partner. Although this metaphor, shown in Fig. 1, is not instantly recognisable, it has led to an aesthetically appealing design which has a simple and elegant form.

Figure 1: Architectural Interpretation

2 Structural Overview

The Metsovitikos is separated into two near identical bridges; therefore this paper only considers one half of the entire bridge structurally. The deck is divided into four spans, its cross section at the extents and in the centre of its longest span is detailed in Fig. 2. It has two 3.75m wide traffic lanes as well as an emergency lane and two pedestrian walkways.

Figure 2: Deck cross section at centre of span

The bridge profile is shown in plan and elevation in Figs. 3,4 respectively. It should be noted that both halves of the bridge have a constant longitudinal slope of 2.6%.

The connections to the deck at piers M2 and M3 is rigid, therefore any longitudinal movement due to wind, traffic or temperature change will induce a moment in the supporting piers. The connection at each M1 pier uses a sliding pot bearing to avoid overstressing in these piers due to variation in ambient temperature, outlined in 3.4.1. Sliding Pot bearings at the abutments allow the structure to rotate and move longitudinally, movement in the transverse direction is restricted by guided bearings.
The bridge deck was constructed as a balanced cantilever, using form travellers and strengthened with post-tensioned prestressing tendons. The cantilevers supported by the M2 pier are 125.5m long, consisting of 33 segments; the cantilevers supported by the M3 pier are 108m long, made up of 26 segments. There are 138 internal tendons in the bridge, 98 are placed in the top slab over the built in piers M1 and M2, to relieve hogging moments, 40 are used in the bottom slab to prevent sagging moments during service.

The design of bridges is carried out according to limit state analysis; the bridge design should be checked at both Ultimate Limit State, to prevent collapse at peak design load, and at Serviceability Limit State to ensure the bridge remains functional for its intended use. There are five load combinations to be checked at both the SLS and ULS, shown in Table 1. Nominal Loads must be calculated for several load conditions individually and later multiplied by two variable partial load factors to give the worst case loading conditions.

<table>
<thead>
<tr>
<th>Load Type</th>
<th>Load Combinations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Permanent</td>
<td>1</td>
</tr>
<tr>
<td>Vehicular</td>
<td>2</td>
</tr>
<tr>
<td>Wind</td>
<td>3</td>
</tr>
<tr>
<td>Temperature</td>
<td>4</td>
</tr>
<tr>
<td>Erection</td>
<td>5</td>
</tr>
<tr>
<td>Secondary Live</td>
<td></td>
</tr>
<tr>
<td>Friction at Supports</td>
<td></td>
</tr>
</tbody>
</table>

3.1 Permanent Loading

3.1.1 Dead

The permanent dead load is attributed to the self-weight of the structure. It is difficult to calculate a uniformly distributed load on the bridge deck, as its cross section profile is changes non-linearly throughout the majority of its length. This paper therefore approximates the varying profile as a linearly changing distributed load, which follows the minimum and maximum cross section areas shown in Fig. 5.

The density of reinforced concrete is taken as 2400 kg/m³, giving a maximum and minimum value for the linearly varying UDL of 853.8 KN/m and 312.8 KN/m.

3.1.1 Superimposed Dead

This includes all loading due to non-structural elements included in the completed bridge such as parapets, road surface, fill and services. These loads can vary throughout the life of the bridge as maintenance and expansion works are carried out. These associated uncertainties mean they are usually given a high partial load factor; any relieving action can also disregarded in design checks for the worst case loading condition.

The Superimposed Dead Load can be divided into an estimate for bridge furniture across the width of the deck of 0.5KN/m² and the 100m asphalt road surface across the 10.5m carriageway with a unit weight of 2300Kg/m³.

\[
\begin{align*}
\text{Furniture:} & \quad 0.5 \times 13.45 = 6.7KN/m \\
\text{Surface:} & \quad 0.1 \times 10.5 \times 2.3 \times 9.8 = 23.6KN/m \\
\text{Total:} & \quad 6.7 + 23.6 = 30.3KN/m
\end{align*}
\]

3.2 Vehicular Loading

To apply highway traffic loading to bridges, the bridge deck should be divided into notional lanes. The total carriageway width is 10.5m, giving 3 notional lanes, each 3.5m wide. Each lane should then be loaded with the appropriate combination of HA and HB loading.

3.2.1 HA Loading

This loading is designed to represent heavy, fast moving traffic and already includes an impact factor. It consists of a uniformly distributed load and knife-edge load; these will fully load two notional lanes and act at a third of the size on the remaining lane. Eq. (1) relates the nominal uniformly distributed load per metre length to the bridge loaded length. A minimum value of 9KN/m is recommended, and is therefore used in this analysis.

\[
w = 151 \left( \frac{1}{L} \right)^{0.475} = 7.62KN/m \tag{1}
\]

3.2.2 HB Loading

HB loading represents an abnormal truck load, the length of which is variable between 6m and 26m to
produce the most adverse effect. A full HB load is considered to be 45 units, one unit represents 10 KN, the weight of one axle. The HB loading is 3.5m wide and so will fall within the width of one lane, it requires a 25m clearance in front of and behind the vehicle.

3.3 Wind Loading

When calculating wind loading, a number of factors should be taken into account including the bridge height, location and the surrounding topography. The aerodynamic effects dependent on the size, shape and surface properties of the bridge are not considered in this paper.

The maximum wind gust acting on the bridge is given by Eq. (2). The equation includes a wind coefficient, K1; taken as 1.77, a funneling coefficient, S1; taken as 1.08 to account for the surrounding steep valley, and a corresponding gust factor S2; taken as 1.59.

\[ v_c = v K_1 S_1 S_2 \]  

(2)

The wind speed at deck level used in the design for the bridge in serviceability conditions was 35.2m/s and 27m/s in construction conditions, which taking into account the roughness of the forest floor, was reported 3 times during construction [8]. Using Eq. (2), maximum wind gusts of 107.0 m/s and 82.0 m/s for serviceability and construction conditions respectively.

3.3.1 Transverse Wind Load

The horizontal wind load acting on the centroid of the structure, is given by Eqs (3,4).

\[ P_T = q A_s C_D \]  

(3)

\[ q = 0.163 v_c^2 \]  

(4)

The cross section is a single box with vertical sides and so the drag coefficient \( C_D \) can be calculated using BS 5400 [5] and a b/d ratio of 13.25/8.5 = 1.56 to be 1.7. To calculate the projected area \( A_s \) for a live loaded bridge with an open parapet, the depth should be taken as \( d_s \), the sum of \( d_i \), \( 8.5m \) and \( d_c \), 2.5m for a highway carriageway, for an unloaded \( d_i \) is simply taken to be \( d \).

The projected unloaded area is therefore 4564.5m², and the wind load during construction is calculated as 20.5 KN/m. The projected live loaded area must include the projected area of the traffic and is calculated as 5907m², giving a serviceability wind loading of 59.6 KN/m.

3.3.2 Vertical Wind Load

Uplift or vertical downward force on a bridge can have a significant effect on deflections experienced by the deck. It can also be applied as a relieving action in bending moment analysis. A nominal force can be calculated in a similar way to the transverse wind load.

\[ P_v = q A_s C_L \]  

(5)

The vertical wind load is calculated to be 21.8 KN/m and 37.2 KN/m for construction and serviceability load cases respectively.

3.4 Temperature Loading

3.4.1 Effective Temperature

A change in ambient temperature will cause the material in the bridge to seek to expand or contract. The Metsovitikos therefore uses bearing joints at the abutments and at the connection to pier M1. However the connections between piers M2 and M3 are rigid, giving the fixity required for cantilever construction.

The degree to which the concrete will expand if allowed is calculated using Eq (6).

\[ \varepsilon = \alpha \Delta T \]  

(6)

Extreme air shade temperatures have been assumed to be -13°C and 42°C, as it was not possible to find adequate data for the 120 year return period. Air shade temperatures are converted to effective bridge temperatures in tables 10 and 11 in ref [5]. Assuming the bridge was installed at an average temperature of 18°C the maximum temperature changes can be found and the strains calculated using a coefficient of thermal expansion for concrete of 12x10⁻⁶/°C.

<table>
<thead>
<tr>
<th>Inaccuracy Factor, ( \alpha )</th>
<th>Air shade (°C)</th>
<th>Effective (°C)</th>
<th>Strain</th>
<th>Extension (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum</td>
<td>42</td>
<td>41</td>
<td>288με</td>
<td>67.7</td>
</tr>
<tr>
<td>Minimum</td>
<td>-13</td>
<td>-8</td>
<td>-312με</td>
<td>-73.3</td>
</tr>
</tbody>
</table>

The extensions between the M2 and M3 piers are restrained which generates a maximum stress in the bridge deck between them, and their stiffness will result in a large bending stress. At M1, the force acting on the pier will be a result of the friction at the bearing.

The extensions between the M2 and M3 piers are restrained which generates a maximum stress in the deck of 8.64 N/mm². If the deck has a cross sectional area of 32.2m², a 139.1 MN force is spread across the height of the pier head at each pier.

3.4.2 Temperature Difference

Temperature can also have a local effect with differing temperatures through the bridge deck. Although the cross section can be modeled as entirely concrete and so theoretically has constant thermal conduction and expansion properties, heat transfer can lead to a difference in temperature between the top and bottom slab and within the cross section. Without a
constant thermal expansion through the deck, bending stresses will be induced. Figure 5 shows the temperature profile for the Metsovitikos bridge deck.

![Temperature Profile](image)

**Figure 5: Temperature Profile**

### 3.5 Seismic Loading

The Metsovitikos is located in a class 1 seismic zone according to the Greek Earthquake Code and has therefore been designed to resist a horizontal peak ground acceleration (PGA) of 0.16g and a vertical PGA of 0.102g. These were upgraded by a factor of 1.3, corresponding to a return period of 1000 years. The design horizontal acceleration generates force acting on the bridge deck proportional to its mass. The total horizontal force is calculated below, assuming the majority of the mass comes from concrete, which has a volume of 50700m$^3$.

$$F_h = V \cdot 0.16 \cdot g = 50700 \times 2400 \times 0.16 \times 9.8 = 190.79 \, M\Lambda$$

The equivalent uniformly distributed load is 332.97 KN/m, this induces a worst case bending stress of 35.7 N/mm$^2$ in pier M2. The vertical uniformly distributed load is calculated in a similar way to be 226.5 KN/m. Both earthquake loadings were applied at half the size in construction calculations.

Potential deck uplift during major seismic events is prevented at the east abutment by the provision of anti-uplift brackets attached to the bearings.

Vertically placed elastomeric bearings are included between the deck and side walls of M1 pier-heads, and between the deck-ends and abutment cheek-walls, to act as bumpers during seismic events of intensity greater than the design earthquake.

### 3.6 Design Loads

Design loads should be derived from the nominal loads to obtain the worst effect for each combination. Two partial factors are applied to varying amounts to give greater loads in desired areas. Any relieving actions resulting from live or super imposed dead loads can be neglected, as these loads may not always be present.

#### 3.6.1 Inaccuracy Factor

This factor allows for possible inaccuracy in the structural analysis of the bridge.

<table>
<thead>
<tr>
<th>Load</th>
<th>Load Combination</th>
<th>1</th>
<th>2</th>
<th>3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete</td>
<td>ULS</td>
<td>1.15</td>
<td>1.15</td>
<td>1.15</td>
</tr>
<tr>
<td></td>
<td>SLS</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td>SID</td>
<td>ULS</td>
<td>1.75</td>
<td>1.75</td>
<td>1.75</td>
</tr>
<tr>
<td></td>
<td>SLS</td>
<td>1.20</td>
<td>1.20</td>
<td>1.20</td>
</tr>
<tr>
<td>Wind</td>
<td>ULS</td>
<td>1.40</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>SLS</td>
<td>1.00</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Temperature</td>
<td>ULS</td>
<td>1.30</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>SLS</td>
<td>1.00</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Vehicular</td>
<td>ULS</td>
<td>1.50</td>
<td></td>
<td></td>
</tr>
<tr>
<td>HA</td>
<td>SLS</td>
<td>1.20</td>
<td></td>
<td></td>
</tr>
<tr>
<td>HB</td>
<td>ULS</td>
<td>1.30</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SLS</td>
<td>1.10</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The factored loads are then summarized in Table 5. At this stage, the ratio between the dead load and live load can be noted. A cantilever constructed concrete deck bridge with these dimensions is likely to experience the majority of its loading from its self weight. Other variable actions such as traffic and environmental loads seem small in comparison. It is necessary to apply these in the worst possible loading condition in order to understand their effects on the bridge structure.

### Table 3: Inaccuracy Factor

<table>
<thead>
<tr>
<th>Inaccuracy Factor, $\gamma_3$</th>
<th>SLS</th>
<th>ULS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel</td>
<td>1.00</td>
<td>1.10</td>
</tr>
<tr>
<td>Concrete</td>
<td>1.00</td>
<td>1.15</td>
</tr>
</tbody>
</table>

The inaccuracy factor is taken to be 1.15, the value used when checking the elastic capacity of concrete.

#### 3.6.2 Partial Load Factor

This factor is altered depending on the load type, limit state and load combination, Table 4 shows a summary of the values to be used in this paper.

<table>
<thead>
<tr>
<th>Load</th>
<th>Load Combination</th>
<th>1</th>
<th>2</th>
<th>3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete</td>
<td>ULS</td>
<td>1.15</td>
<td>1.15</td>
<td>1.15</td>
</tr>
<tr>
<td></td>
<td>SLS</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td>SID</td>
<td>ULS</td>
<td>1.75</td>
<td>1.75</td>
<td>1.75</td>
</tr>
<tr>
<td></td>
<td>SLS</td>
<td>1.20</td>
<td>1.20</td>
<td>1.20</td>
</tr>
<tr>
<td>Wind</td>
<td>ULS</td>
<td>1.40</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>SLS</td>
<td>1.00</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Temperature</td>
<td>ULS</td>
<td>1.30</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>SLS</td>
<td>1.00</td>
<td></td>
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</tr>
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<td>Vehicular</td>
<td>ULS</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>HA</td>
<td>SLS</td>
<td>1.20</td>
<td></td>
<td></td>
</tr>
<tr>
<td>HB</td>
<td>ULS</td>
<td>1.30</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SLS</td>
<td>1.10</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

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### Table 4: Partial Load Factor

<table>
<thead>
<tr>
<th>Load</th>
<th>Load Combination</th>
<th>1</th>
<th>2</th>
<th>3</th>
</tr>
</thead>
<tbody>
<tr>
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<td>ULS</td>
<td>1.15</td>
<td>1.15</td>
<td>1.15</td>
</tr>
<tr>
<td></td>
<td>SLS</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td>SID</td>
<td>ULS</td>
<td>1.75</td>
<td>1.75</td>
<td>1.75</td>
</tr>
<tr>
<td></td>
<td>SLS</td>
<td>1.20</td>
<td>1.20</td>
<td>1.20</td>
</tr>
<tr>
<td>Wind</td>
<td>ULS</td>
<td>1.40</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>SLS</td>
<td>1.00</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Temperature</td>
<td>ULS</td>
<td>1.30</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>SLS</td>
<td>1.00</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Vehicular</td>
<td>ULS</td>
<td>1.50</td>
<td></td>
<td></td>
</tr>
<tr>
<td>HA</td>
<td>SLS</td>
<td>1.20</td>
<td></td>
<td></td>
</tr>
<tr>
<td>HB</td>
<td>ULS</td>
<td>1.30</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SLS</td>
<td>1.10</td>
<td></td>
<td></td>
<td></td>
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### Table 5: Load summary (KN/m)

<table>
<thead>
<tr>
<th>Load</th>
<th>Nominal Loads</th>
<th>Adverse Span</th>
<th>Beneficial Span</th>
</tr>
</thead>
<tbody>
<tr>
<td>Permanent</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dead min</td>
<td>312.80</td>
<td>413.68</td>
<td>413.68</td>
</tr>
<tr>
<td>Dead max</td>
<td>853.80</td>
<td>1129.15</td>
<td>1129.15</td>
</tr>
<tr>
<td>SID</td>
<td>30.30</td>
<td>60.98</td>
<td>41.81</td>
</tr>
<tr>
<td>HA only</td>
<td>21</td>
<td>36.23</td>
<td>-</td>
</tr>
<tr>
<td>HA &amp; HB</td>
<td>18</td>
<td>26.91</td>
<td></td>
</tr>
<tr>
<td>Variable</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Wind, vertical, service</td>
<td>37.20</td>
<td>59.89</td>
<td></td>
</tr>
<tr>
<td>Wind, vertical, construction</td>
<td>21.80</td>
<td>35.10</td>
<td></td>
</tr>
<tr>
<td>Wind, trans, service</td>
<td>59.60</td>
<td>95.96</td>
<td></td>
</tr>
</tbody>
</table>
4 Structural Analysis

4.1 Structural Model

As outlined in 2 the connections at the top of each of piers M2 and M3 are connected rigidly while pier M1 meets the deck with a roller connection. At the abutments roller connections are also used to allow expansion due to variations in temperature.

![Figure 6: Structural scheme](image)

The bridge deck has been approximated as a constant cross section to aid analysis, the dimensions were estimated using the change with length between the maximum and minimum cross section. The nominal cross section is 10 m deep and has a 2 m deep bottom slab. A second moment of area has then been calculated to be 509.2m$^4$ using parallel axis theorem and assuming the neutral axis to be at 0.45d.

![Figure 7: Image showing pier connections to deck](image)

4.2 Serviceability Load Cases

Combination 1 loading is applied to the bridge deck, maximum and minimum values of $\gamma_S$ are applied appropriately to the super imposed dead loads and live loads in order to achieve the worst case hogging or sagging moment. HB loading is also positioned in the worst location to achieve maximum bending moments in the bridge deck.

4.2.1 Transverse Bending Moments & Torsion

An uneven application of live loads can cause significant torsion moments in the bridge deck. The worst case scenario shown in Fig 7, leads to a torsional moment in the deck of 2.13 MNm, this value is relatively low because the eccentric loading is only a result of the live loading which is relatively small in comparison to the dead load. The shear force generated can be calculated using Eq. 7, Ref [3] where $T$ is the torsion moment, $t$ the wall thickness and $a$ and $b$ the centre line dimensions of the sides of the box.

$\nu_t = \frac{T}{2abt} = \frac{2.13 \times 10^6}{2 \times 8.5 \times 6.4 \times 0.6} = 32.6\text{KN}$ (7)

The stress of 32.6 MPa could not be taken by the concrete but is a reasonable quantity to be taken by the area of reinforcement in the column.

4.2.2 Longitudinal Bending Moments

The bridge structure was analysed using QSE Plane frame as shown in Fig 6 under the loading conditions shown in Figs. 9,10 to achieve maximum hogging and sagging moments.

![Figure 8: Transverse Loading](image)

![Figure 9: Maximum sagging moment loading](image)

![Figure 10: Maximum hogging moment loading](image)
The maximum hogging and sagging moments were obtained for each element and are shown in Fig. 11. Because the variable loads are very small, the bending moments are very similar in both loading cases. They have therefore been combined onto a single bending moment diagram, and the worst case moments labeled.

### 4.3 Prestress

The Metsovitikos uses 138 post-tensioned, internal prestressing tendons each consisting of 19, 15.7 mm superstrands. Table 1 from ref [5] shows the corresponding characteristic strength to be 1.5 KN/mm² and Young Modulus to be 195 KN/mm². The British Standards state the maximum allowed force in the tendon is equal to 70% of the characteristic strength for post tensioned tendons. The Force in the Prestressing tendons is then calculated using Eq. (8).

\[ F = 0.7 f_c A_p. \]  

Using a tendon cross sectional area of 150 mm², the total force in the top and bottom decks can be calculated. This force is used to derive the resultant axial compressive stress due to the prestressing using Eq. 9, summarized in Table 6.

\[ \sigma_c = \frac{F}{A} \]  

<table>
<thead>
<tr>
<th>Prestressing Actions</th>
<th>Area (mm²)</th>
<th>Force (MN)</th>
<th>Stress (N/mm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top</td>
<td>14700</td>
<td>15.44</td>
<td>0.616</td>
</tr>
<tr>
<td>Bottom</td>
<td>6000</td>
<td>6.30</td>
<td>1.15</td>
</tr>
</tbody>
</table>

### 4.4 Bending Stresses in the Deck

The maximum hogging bending moment is 3.15GNm and maximum sagging bending moment 1.58GNm. Having found the column’s second moment of area, bending stresses are calculated using Eq. (11).

\[ \sigma = \frac{M_y}{I} \]  

The resultant bending stresses are then added to the resultant stress due to the prestressing in Table 6 to give the worst case stresses, summarized in Table 7. The greatest compressive strength is below the concrete’s strength and any tension stress would be carried by the longitudinal reinforcement.

<table>
<thead>
<tr>
<th>Bending Stresses (N/mm²)</th>
<th>Top</th>
<th>Bottom</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hogging</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sagging</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bending</td>
<td>-27.94</td>
<td>34.15</td>
</tr>
<tr>
<td>Prestress</td>
<td>0.97</td>
<td>0.25</td>
</tr>
<tr>
<td>Resultant</td>
<td>-26.97</td>
<td>33.9</td>
</tr>
</tbody>
</table>

### 4.5 Bending Stresses in the Columns

The rigid connections at M2 and M3 pier heads form a rigid frame between the piers and the deck, this is beneficial in taking longitudinal loadings from effects such as temperature expansion, however any longitudinal or transverse loading will generate stresses.
in the piers themselves. As they have different heights, they require different cross sections giving bending capacities that ensure bending stresses are evenly shared. The maximum bending moments in the piers were found, having been checked under transverse wind loading as well as under maximum sag serviceability conditions. The resultant bending stresses were found after estimating second moments for the piers and are shown in Table 8.

Table 8: Bending stresses in columns

<table>
<thead>
<tr>
<th>Max moment</th>
<th>I value</th>
<th>Bending stress</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.05 GNm</td>
<td>784.4 m²</td>
<td>6.67 N/mm²</td>
</tr>
<tr>
<td>1.35 GNm</td>
<td>895.5 m²</td>
<td>4.50 N/mm²</td>
</tr>
</tbody>
</table>

The columns have therefore been effectively designed, with sufficient design strengths of 45N/mm².

5 Foundations and Geotechnics

The ground at the foundations typically comprises alternating beds of thickly bedded sandstone of flysch and siltstone with some limestone intercalations. Some minor tectonic faults were found nearby, however they were judged as inactive.

To ensure the foundations of piers M2 and M3 have sufficient moment capacity, they are founded on 12 m diameter circular rock sockets with depths of 25 m and 12 m, respectively. Both the abutments and the foundation of the M1 Pier have spread footing foundations, neither having to resist any moments.

6 Construction

The Metsovitikos was built using balanced cantilever construction, a technique in which the bridge deck is built outwards in both directions from one pier simultaneously. The deck requires permanent fixity at the connection with the pier, which in the case of this bridge is permanent. The fixity allows a hogging moment to be generated at the support, this is much larger than during serviceability, as the connection of the cantilevers gives the deck a sagging moment capacity. Construction ran from Oct 2005 until 2008.

6.1 Construction Process

6.1.1 Foundations

After stabilizing the rock face and constructing access roads, the ground was leveled to allow the installation of mobile overhead gantries. These temporary frames allowed the drilling of the sockets for each of piers M2 and M3, the rock was excavated by drilling at the perimeter to depths of 2.5m followed by sequential firing of explosive charges placed in a helical formation. To reach acceptable tolerances on the deeper M2 socket, the drilling machine had to be lowered into the socket after a depth of 1.5m, before the second stage of drilling was carried out. The excavated face of the sockets was regularly inspected and was stabilized using rock anchors, steel beams and corrugated metal sheets, then further strengthened with mesh reinforcement and shotcreting. The sub ground level M3 sockets required 6m circular retaining walls, which were place onto the rock sockets via elastomeric bearings so as not to transfer too much of the pressure from the surrounding soil onto the foundations.

The sockets were then concreted in multiple lifts, each requiring up to 500m³ of concrete, care was taken to ensure suitable temperatures were maintained to avoid micro cracking. Reinforcement was installed, most of which was located vertically around the perimeter of the sockets. The sockets were completed with a 3m deep, heavily reinforced pile cap. The deep spread footings for the M1 piers included high strength anchors through the foundations to prevent uplift during construction.

6.2.1 Piers & Pier Heads

M2 piers were constructed using a self-climbing formwork system, each segment was 4.4m tall and was built around 7m prefabricated reinforcement cages, taking six days to construct. At the connections, longitudinal bars were installed at double length to avoid lapping of the bars in areas where fixity was required. Conventional climbing formwork was used to construct the M1 and M3 piers.

It was impractical to provide support for the pier construction from ground level and it was therefore
necessary to support the required formwork from the top of each of the piers. The M2 and M3 pier heads contain two diaphragms and access necessary for ongoing maintenance of the inside of the piers and box girders. They were constructed in five phases with the use of a proprietary formwork system, which could allow for details such as the openings.

6.3.1 Superstructure

According to Ref. [2], the main sequence of construction for each deck was:

- Construction of M2 balanced cantilever
- Construction of M3 balanced cantilever
- Construction of connecting segment between M2 and M3 balanced cantilevers
- Construction of M1 pier-head, jacking up the deck over M1 pier and installing bearings
- Construction of a temporary steel frame to jack up the free end of M3 balanced cantilever
- Deck-end construction on temporary supports
- Construction of the deck beyond M1 pier and temporary frame using a form-traveller
- Construction of the connecting segments between cantilever-ends and deck-ends.

The deck itself was cast in-situ, segmentally, using a form traveller, it was necessary for this to have the capacity to support the weight of an entire segment as well as equipment whilst having its centre of gravity beyond the end of the last completed segment.

Four form travellers were used in construction to minimize construction time. Each form traveller was anchored to the completed deck and held a truss from which the necessary formwork could be suspended through the use of high-strength hangers. The form travellers were moved using two hydraulic rams. The casting of the first segments on either side of the pier head had to be carried out separately, as the 10m pier head thickness was too small too allow sufficient anchoring for the form traveller. After the first pair of segments was cast, the remaining pairs were constructed simultaneously to reduce unbalanced cantilever loading minimizing bending stresses in the pier. Also taken from Ref. [2], each segment of the bridge deck was constructed as follows:

- Moving the traveller pair to a new position
- Adjusting the formwork levels for a new segment pair
- Installing the reinforcement, prestressing ducts and tendons.
- Concreting and prestressing new segment pair
- Positioning internal box girder formwork
- Casting the segment in two stages.

Measures were taken during the cantilever construction to ensure tolerances were met; the position of each new segment was measured using steel markers after casting, prestressing and moving the form traveller. Regular concrete samples were also taken during casting to determine properties such as creep, shrinkage and modulus of elasticity. These readings were then used to account for any deflection and plan the coordinates for the next segment accordingly.

Early prestressing can help reduce the likelihood of thermal or shrinkage induced cracking and is also in the interest of productivity. However, it is important when prestressing concrete to ensure it has reached a minimum strength before prestressing is applied to allow the anchors to be loaded [9]. In the Metsovitikos, a minimum strength of 32MPa was generally achieved after 3 days.

To create the 1.5m long connecting segment, a levels survey was carried out to find the height difference between each cantilever tip. External formwork and temporary steel I beams provided support for the shuttering across the gap. After concreting, some of the bottom tendons were partially stressed to prevent horizontal movement.

![Fig 15: Form traveller](image)

After connecting the balanced cantilevers, the eastern end was jacked up for the construction of the pier head and installation of the bearings at M1. At the western end, a temporary steel frame support was then erected and the free end was jacked up 150mm. Form travellers were then once again used to continue the cantilever construction to meet each abutment where the deck ends were constructed on scaffolding.

6.1 Construction Load Case

During construction of the deck, whilst the cantilevers are not connected, no sagging moments can be generated, which means all bending stresses
generated are taking in hogging at the pier heads, and additional load also acts in the form of the 850 KN form traveller.

The worst case hogging moments at the pier M3, were found to be 4.62 GNm, when wind load, super imposed dead load and the form traveller load were both acting. Neglecting the wind and super imposed loads on one side generated a bending moment of 0.32 MNm in the pier, which is smaller than that moment checked in 4.5 when the full frame had been constructed. Adding the wind loading to just one half of the cantilever generated a 0.16 MNm torsion moment in the pier. Using Eq (7) this generates a shear force in the pier of 0.93 KPa, taken by the longitudinal reinforcement.

7 Natural Frequency

Vibrations can cause serious structural damage to a bridge as well as reducing the user comfort levels. The bridge will tend to oscillate at its natural frequency, to mitigate resonance, it should be checked that this frequency is above 5Hz to ensure it is not similar to the frequency of the wind. The natural frequency can be calculated using Eq (12), Ref [5].

\[ f = \frac{k}{2\pi^2} \sqrt{\frac{EI}{m}} \]  

(12)

The value EI is the deck flexural stiffness in vertical bending, L is the central span length and m is the mass per unit length including both dead and super imposed dead loads. K is a dimensionless constant taken as 4.730 for a multi-span simply supported bridge with built in supports. The Metsovitikos is then found to have a natural frequency of 42.6 Hz in its deck, further research could include an investigation into the natural frequency the entire structure, taking into account the stiffness in the column and foundation.

8 Time Increasing Load Effects

Creep and shrinkage in the concrete deck is likely to cause additional stresses to the piers as possible contraction occurs. Creep of concrete can be divided into basic creep and drying creep. Drying shrinkage may induce bending moments as the bottom slab is significantly thicker than the top, their variable drying times could potentially lead to variable shrinkage, having a similar effect to temperature difference through the deck. Segmental construction will allow for shrinkage effects to some degree as each section is allowed to cure and shrink before the next is constructed, where there is an opportunity to compensate for it.

With straight tendon profiles, force is lost during prestressing due to extension of the cable. However stress relaxation of the cables will decrease the prestress in the cables over time as will creep and shrinkage of the concrete.

Differential settlement has the capacity to induce very large bending moments in such a stiff structure, however the bedrock and very heavy duty concrete socket foundations should be sufficient to resist the axial compression and prevent large settlements. As each foundation is founded on a common rock type, differential settlements should be minimal.

9 Conclusion

The Metsovitikos succeeds in providing a striking, elegant structure within the Pindos mountains. Although it uses well understood technology to fulfill its function, the challenging design and construction has been effectively implemented and the bridge now spans the furthest of its kind in the Balkans.

References