A CRITICAL ANALYSIS OF THE DESIGN AND CONSTRUCTION OF THE WADI ABDOUN BRIDGE, AMMAN, JORDAN

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Abstract: This article provides a critical analysis of the Wadi Abdoun Bridge in Amman. In response to its context, the cable-stayed bridge crosses a valley in a seismic zone, with an unusual, yet dynamic, ‘S’ shaped horizontal alignment. The bridge’s unusual aesthetics and innovative design and construction features are discussed. Bridge loadings are defined and used to verify the main structural elements. Alternative design and construction options, and possible improvements are suggested throughout.

Keywords: Wadi Abdoun, cable-stayed, balanced cantilever, pre-cast, semi-box girder

1 General Introduction

The Wadi Abdoun Bridge is a cable-stayed dual two-lane concrete road bridge across a valley in Amman, see Fig. 1. The bridge forms part of a new ring road network that ensures Amman’s infrastructure is well connected, crucial for any city with a growing role as a centre for trade and services.

Since opening to traffic in December 2006, the bridge has had a positive impact on Jordan’s economy, promoting trade and a more efficient infrastructure network. Indeed, by increasing the accessibility of the city and its amenities, reducing congestion and traffic times, and considerably shortening driving distances, the bridge has resolved many of the city’s transport problems.

The Municipality of Greater Amman as the client, chose the global engineering consultants Dar Al Handasah to undertake the engineering design of the Wadi Abdoun Bridge. The construction tender was awarded in August 2002 to India’s Larsen and Toubro in a consortium with the Arab Technical Construction Company of Jordan, Ref. [1].

2 Structural Arrangement and Geometry

The bridge’s concrete pre-cast deck is 390m long, 29.36m wide, and follows a sinuous ‘S’ shaped horizontal alignment. The deck is supported by inclined cable-stays off of three ‘Y’ shaped concrete pylons. The deck is split into four continuous cable-stayed spans, with the two central spans at 132m and the two end spans at 63m, Ref. [1].

The three pylons are repeated, though each varies in height between the deck and the valley floor, with the deck at a constant height of 45m relative to the base of the central pylon. All pylons increase in height by an extra 26m above the deck to accommodate the cable stays, Ref. [1]. The central pylon is tallest, at a total height of 71m.

The deck is supported by a double plane of stay cables. Each plane uses five parallel stays positioned regularly along each of the ‘Y’ shaped pylons arms in a harp arrangement.

3 Aesthetic Considerations

Measuring the aesthetic success of the Wadi Aboun is difficult, as it is not quantifiable. However, Fritz Leonhardt’s ten areas of bridge aesthetics offer the basic areas for judging the bridge’s aesthetics.

It is unclear if the designers of the Wadi Abdoun had a clear aesthetic concept for the bridge. However, it is apparent that the bridge’s sinuous ‘S’ shaped deck, ‘Y’ shaped pylons and harp arranged cables combine to give the bridge a high degree of complexity and structural innovation. The bridge has a dynamic character that contains enough complexity to make the public question how the bridge works, adding to the bridge’s overall aesthetic appeal.
The aesthetics of cable-stayed bridges are often well suited to valley crossings, and the Wadi Abdoun is no exception. The bridge “glides” across the valley when viewed from a far, and drivers on the bridge follow a gentle sinuous route, with constantly changing views of the bridge and Amman. Using a less rigid alignment may be a questionable attempt to integrate the bridge into the varying landscape. Instead, the modern bridge seems to cut through Amman’s traditional architecture, see Fig. 2. The contrast may have been intentional to clearly demonstrate the new structure and the technology it used, however a less elaborate aesthetic may have been more appropriate.

The bridge’s structural hierarchy divides into three elements: deck; pylon; and cable-stays. The deck conveys the traffic across the valley, the three pylons support the deck at three points, and the cable-stays support the deck at closer spacings. Most cable-stayed bridges have this structural hierarchy, which usually clearly displays direct load paths from the deck to the pylons through the cable-stays. The Wadi Abdoun struggles to do this clearly, with its structural purity confused by the deck alignment.

The ‘S’ shaped horizontal alignment is an illogical form for bridge decks, and is rarely justified due to the higher construction complexity. Indeed a curved horizontal alignment usually excludes the use of cable-stayed bridges, Ref. [2]. Why then does the deck have this dynamic alignment? The alignment may be necessary for founding conditions, pylon locations, and or alignment into connecting infrastructure. It could also be to improve the safety of drivers using the bridge, with the alignment forcing a reduction in speed and an increase in concentration. Alternatively, the alignment may have been chosen to offer something unusual and visually stimulating. However, are the aesthetic benefits of a dynamic shaped deck justified by the higher costs and complexity of construction? Building such a complex structure seems uneconomical for a lower income country such as Jordan. However, the higher cost of a landmark structure may be justified as the bridge will certainly encourage investment into the country and improve its construction expertise. Indeed, Amman is becoming a global trading hub, and this bridge shows it is investing heavily into its transport infrastructure.

In comparison to the curving horizontal alignment, the vertical alignment is simple with a continuous deck line. Any more geometric complication would have confused the structure further. That said, the decks soffit has voids between the transverse diaphragm elements, see Fig. 3. Although there is no structural requirement to close these voids, the simplicity of a continuously flush soffit may have appeared better. The joint between the pre-cast deck segments also breaks the flow of the deck. However, it is appreciated that the nature of a pre-cast segmental deck makes this joint difficult to hide. Similarly, the cable-stay anchorages details ruin the flow, though most concrete cable-stayed bridges have this detail at the soffit for accessibility purposes. More positive is the refinement of the deck’s curved aerodynamic cross section.

Using repetition of three pylons elements is rational, and gives the bridge order. Three pylons seem to be right for creating a good balance between the deck’s depth and span and between the deck’s height and overall length. Using shorter side spans may be an attempt to address the geometry of the valley and to achieve a constant aspect ratio between the spans and height of deck above the valley floor. However, this seems unlikely as using only three pylons positioned symmetrically means the aspect ratio is unnoticeable.

If more that three pylons were used the appearance below deck would certainly be chaotic and more imposing on the surroundings. Indeed, the benefit of cable-stayed bridges is that they need few structural supports from the ground, which combined with the high level deck means that shadows and areas of darkness are not really an issue.
bridges usually have pylons with rough surfaces, Ref. [2], and using a ribbed surface finish may be an attempt to achieve a roughness to contrast with the smooth finish of the deck - typical of concrete bridges. However, the pylons may have looked better smooth. Indeed, a ribbed finish increases the construction cost and complexity as plastic liners have to be used on the internal surfaces of the formwork, Ref. [1], and the concrete has to have higher workability characteristics to flow into the ribs. That said, the combination of horizontal banding and vertical ribs gives the pylons a surface quality, which unusually for concrete structures, looks fresh and modern.

The cable-stays are in a harp arrangement. A fan arrangement would have been more structurally efficient as all the cable-stays would have been anchored to the top of the pylons at maximum eccentricity to the deck. Although the sinuous horizontal alignment means that when viewed obliquely, cable-stay crisscrossing occurs, the fan arrangement lessens its visual impact, as does only having five stays from each side of pylon’s arms, see Fig. 5. Criss-crossing could have been eradicated if a single plane of cable-stays were used. However, although a single plane of cables would have possibly given the bridge a better aesthetic, the deck would have required a higher torsional rigidity, especially being in a seismic zone, and this would have required a very deep deck structure. The harp arrangement also means that all cable-stays carry the same maximum design forces and therefore can be the same size, which in turn allows the cable-stay anchorage points to use the same details, Ref. [3]. This is both optimal and rational.

The bridge’s lighting details are impressive, see Fig. 5. Lampposts are not used to illuminate the deck, and instead special luminaries run along the sides and centre of the carriageway, promoting the clean deck lines. Fibre optics run up the sides of the pylons to highlight the pylons edges, Ref. [3], and spotlights at the pylon bases up-light the bridge, making the bridge incredibly appealing by night.

The aesthetic consideration of highest importance is usually that the bridge has a simple function and structure, Ref. [2]. The Wadi Abdoun has a lot going on structurally, and this brings unnecessary structural confusion and complexity, mainly due to its curving horizontal alignment. Nonetheless, it is a very aesthetically pleasing bridge, with many superb refinements. However, the aesthetics of the bridge would certainly lessen if there were further complexities, especially if more pylons were used.

Figure 5: Cable criss-crossing and the bridge by night

4 Structural and Construction Considerations

At concept level, the Wadi Abdoun’s successful construction must have been questionable considering the bridge had to be constructed in a country with relatively low construction skills. However, by using relatively simple, repeated parts, the design has been perfectly rationalized and successfully constructed.

4.1 Ground Works

The foundation system for the bridge’s three pylons uses a total of 88No. 1200mm diameter concrete bored piles socketed into the bedrock to varying depths of 12-21m, Ref. [1]. The bridge abutments rest on simple footings, possible as the bridge puts relatively low vertical forces onto the abutments.

The central pylon sits on a pile cap sized at 18.5x14.5x3m whilst the outer pylons sit on slightly larger pile caps at 22.5x18.5x3m, Ref. [1]. The reasoning behind this variation in dimension of pile cap is unclear. Indeed, the outer pylon’s larger pile caps imply the use of more piles for these pylons, which seems irrational as the central pylon supports the largest spans.

A likely reason for the requirement of more piles for the outer pylons is the curvature of the horizontal alignment, which induces large rotational and out of balance forces on the outer pylons – the central pylon supports a straight deck alignment. It is likely that these forces will be transferred to the piles through prestressing tendons or reinforcement bars running down through the pylons, terminating at the pile cap.

The foundation system has had to consider seismic induced rotations and forces on the bridge. Therefore, the pile caps will most certainly use prestressed anchorage piles to transfer any such forces to the ground.

Using three pylons requires only three separate ground works. This is beneficial as ground works are often the most expensive aspect of bridge projects. Hence, it was a good design decision to bifurcate the three pylons rather than using six separate pylons.

4.2 Pylons – Structure and In-Situ Construction

The three ‘Y’ shaped pylons were cast in-situ. The pylon’s constantly changing profiles certainly required different prefabricated formwork moulds for each lift. The same moulds were likely to have been reused for all three pylons. Alternatively, pre-cast sections could have been post tensioned together, and this would have achieved a better quality finish and accuracy. Indeed, the textured finish achieved by casting the pylons in-situ is testament to the quality of construction achieved. However, Ref. [1] suggests that fly ash was included in the concrete mix to improve its workability, Ref. [1].

Using parallel leaves for the pylons was certainly to achieve a slender appearance, but at the same time, the twin leaves allow the pylons structurally flexibility. This is desirable in terms of longitudinal deck expansion and seismic loading, Ref. [3]. However, the adequacy of the pylon’s flexibility is questionable as they appear quite stocky.
The pylon’s twin leaves and their arms are connected by a web above and below deck, Ref. [1]. This ensures the pylon arms cantilever from deck level, rather than at the point of bifurcation. The pylons arms seem to be angled in plane to the cable-stays, certainly to reduce out of plane bending in the arms, and instead transferring the stay forces as axial compression into the arms.

4.3 Pre-cast Deck Segments

The deck is a pre-cast segmental concrete semi-box girder, held by two planes of cable-stay. Each bay of the deck uses “3m long segments, mate cast in three parts”, Ref. [1]: two outer triangular voided girder elements and one central diaphragm element, see Fig. 6.

Each bay of the deck was certainly split into three segments for ease of transportation and handling. That is, transporting large pre-cast segments to site is expensive and dangerous, and often requires very large transporters. The valley’s sloping terrain would have also made it difficult to transport the segments to the pylon locations. Further, lifting the segments from the ground to the proposed deck level would have been dangerous considering the height above the ground.

Figure 6: Pre-cast triangular voided girder element

The deck’s curving horizontal alignment means each segment has slightly varying dimensions. To achieve the correct geometry, high quality and accuracy, and very small tolerances are required, hence the use of pre-cast concrete deck segments rather than casting in-situ. These pre-cast segments would certainly have been mate-cast, that is, using the previously cast segment as part of the formwork for the adjoining segment.

The depth of the deck could have been reduced had more cable-stays been use. The closer stay spacings would have reduced the spans between cable supports, therefore reducing the bending in the deck. However, it is noted that in cable-stayed bridges the compression in the deck due to the cable-stay forces causes far larger stresses in the deck than any bending, hence it is likely that more stay cables will only reduce the depth of the deck slightly.

Microsilica was included in the concrete mix for durability and strength, and pulverised fly ash was used to reduce shrinkage and improve workability, Ref. [1].

4.4 Erection of Deck Segments

The deck is constructed by balanced cantilever erection, with the use of lifting gantry. That is, erecting the deck symmetrically off of each of the pylons, “so that a balance is maintained over the pylon” Ref. [2].

Generally in balanced cantilever construction, as each deck segment is lifted, it has to be temporarily held in position using temporary stressing bars or tendons in the deck. When the segment that the permanent stay cable anchors to is lifted into position, the permanent stay cable can then be installed to support the deck.

In the case of the Wadi Abdoun, during the lifting of the deck segments, temporary stressing bars seem to have been used, see Fig. 7. This appears to have been combined with the use of temporary stays at every second bay of segments, see Fig. 8. That is, every first and second bay of deck segments (after every permanent stay cable anchorage point) were balanced cantilevered out and held by temporary stressing bars. A pair of temporary stay cables pinned to the top of the pylon arms would then be attached to the end of the second bay of segments to temporary support the deck, see Fig. 8. The temporary stressing bars used at the first and second bays could then be de-stressed and reused for the balance cantilevering out of the third and fourth bay of segments. When the fourth bay of segments is in place, the pair of permanent stay cables could be installed to support the deck, and the temporary stay-cables removed. This erection sequence would have continued until the deck reached the point of closure, see Fig. 9.

Figure 7: Temporary stressing bars on top surface of deck

To ensure perfect deck alignment each of the deck segments will have been carefully lifted into position, glued with epoxy and stitched to the previous segment. The epoxy, as well as being glue to join and seal the segments, it also lubricates the joining surfaces, making the positioning of the segments easier, Ref. [1]. This process required a gantry assembled on the deck along with what seems to be three launching girders, see Fig. 8. The gantry is moved forward after the lifting, gluing and stitching together of each bay of deck segments. Indeed, as each segment was lifted into position a high level of surveying most certainly took place to ensure correct alignment, of incredible importance to ensure the deck cantilevering from each pylon meets accurately at the closure points.

Figure 8: Erection of deck using temporary stays

The temporary stays were certainly anchored to the top of pylons, see Fig. 8, to achieve the most structurally efficient vertical angles and to allow the use of only one temporary stay anchorage point. Due to the changing horizontal alignment of the deck, and varying angle of inclination of the temporary stay, the temporary stay’s anchorage point used a pin to accommodate the temporary stay’s varying angles in both the horizontal and vertical planes, Ref. [1].
Using cable-stays to support the deck during balanced cantilever construction removes the otherwise huge hogging moments in the deck where it meets the pylon, allowing for a relatively slender deck.

Using temporary stays seems to have been a sensible decision. If these were not used at every second bay, the temporary stressing bars would have needed to have carried four times the force to support the segments until the installation of the permanent stay. Further, using temporary stays ensures that the deck never cantilevers further than two segments from a cable-stay support point. It is important to minimise this cantilevering length, since cantilevers are vulnerable to problems of lateral stability due to high wind loads etc, particularly just before the point of closure. However inclining the cable-stays out of the line of plane will have provided lateral stability. Indeed, since the pylons are their anyway they may as well be used for the temporary stays.

With the above in mind, the question that arises is, why were temporary cable-stays not used at every bay to support the deck? Indeed, if temporary stays were used at every bay then temporary stressing bars would have needed to carry only a quarter of the force required in reality. However, it is expected that temporary stays out of plane would have taken more time and effort than using temporary stressing bars for every two cantilevering segments, as done.

4.5 Closure, Post Tensioning Works and Final Tuning

As the balanced cantilever construction progresses from either side of the considered span, the deck is closed. Up until this point, the two decks have been structurally independent, and to ensure they come together at the correct geometry, the deck levels have to be adjusted by tuning the cable-stays at their anchorage points.

Judging from the appearance of the deck’s soffit, it seems that the span’s central segment has an inferior and discoloured surface finish compared to the rest of the span’s segments that were pre-cast. This indicates that an entire, 3m wide, key segment was cast in-situ at the closure point. At the spans closing point, shear reinforcement is required at the connection to ensure the cantilevering ends do not move up and down.

Casting the keying segment in-situ will have posed many problems. The confined space would have caused difficulties. Indeed, it is unknown as to whether the keying segment was cast from a pair of gantry, or if the 3m gap allowed hanging formwork to be used, see Fig. 10. There would certainly have also been an issue of expertise with the pre-cast contractor having to cast the most important section in-situ.

Once the deck was closed, tendons running through the top and bottom of the deck were certainly post tensioned, to pull together all the deck segments and to improve the deck’s robustness. These post tensioned tendons running through the entire length of the ‘S’ shaped deck could potentially induce large rotational forces in the deck (on plan), see Fig. 11. This rotation needs to be resisted by supplying horizontal forces at the abutments. Better still, the pre-stressing tendons at the abutments could have been angled in plan to be on the same axis as the centroid of the deck, therefore removing any eccentricity between the prestress and the centroid of the deck, thus removing the rotation.

With the deck post tensioned together, and all construction loadings removed, the stay’s tensions were certainly adjusted to the desired stress levels. This tuning is crucial. The deck and pylons are both rigid and the stay cables have quite a high stiffness, that is, they are hard to stretch. If any of the stay cables are either too long or short, their axial tension would be considerably different from that of the designed forces, and this will have meant the actual forces put into the deck will again different from the forces the deck was designed for. Further, adjusting the stays will allow the desired alignments to be achieved. The anchorage points viewable from the soffit, hence have to be accessible for future tuning. According to Ref. [1] all concrete surfaces were painted with anticarbonation paint.

4.6 Feasibility of Alternative Construction Strategies

Casting the deck in-situ using a casting girder, a launching girder or indeed casting in-situ segmentally, are not feasible options. As well as being generally more expensive and slower, the segment’s geometry due to the varying horizontal alignment means that casting in-situ would have forfeited the required accuracy and tolerances, certainly leading to an inaccurate alignment and an inferior finish. Further, large quantities of concrete will have been needed to be pumped to deck level at the location of the cast, and thus the concrete would have been of inferior quality.
With the above in mind, cable-stayed bridges are more usually constructed by incremental launching or the chosen balanced cantilever construction using pre-cast elements. Incremental launching would have been ideal, had the bridge’s horizontal alignment been straight or a constant radius. Indeed, there would have been no expensive pre-cast transportation costs, fewer heavy cranes required for lifting, and casting of the segments in factory conditions at the bridge abutments. Indeed, there would have been no hardening of each segment – similar to the Millau Viaduct. In light of the above, the chosen balanced cantilever erection of pre-cast segments seems to have been the only realistic choice for the construction of the Wadi Abdoun.

4.6 Temporary Prestressing Deck Requirements

The minimum total prestressing force required in the temporary stressing bars that stress segment 1 back to segment 4, as in Fig. 7, can be found by considering the lifting of segment 2. During the lifting of segment 2 the stressing bar in segment 1 must be sufficiently prestressed so that there is an “additional zero tensile stress” at the pylon, see Fig 12. This is required because although the maximum allowable tensile stress at the pylon is -1N/mm$^2$ (assumed), the stressing of the previous segments may have used this allowance. Therefore, Eq. (1) must be satisfied.

$$P \div A + \frac{P \cdot e \cdot y}{I} - \frac{M \cdot y}{I} = 0$$

(1)

Where:
- $P$ is the minimum prestressing force in the stressing bars;
- $A$ is the cross sectional area of the deck. It is assumed that the two side girder elements of the deck take all the stress due to the prestressing of the temporary stressing bars. Assuming the side girder elements are triangular in cross section, of depth 3m and width 6m, with a void area of approximately 50%, gives a total cross sectional area of two girder elements of 9m$^2$, found in Eq. (2).

$$A = 2 \times \left(0.5 \times (3m \times 6m) \times 50\% \right) = 9 \text{ m}^2$$

(2)

e is the eccentricity to prestress. Assuming, the centroidal axis of the deck is 1m below the deck’s top surface and the temporary prestress bar is applied at deck level, $e = 1$m;
-I is the second moment of area of the deck. Assuming the girder elements are triangular, $I$ is found in Eq. (3).

$$I = I_{\text{girder}} - I_{\text{void}} = 9 - 2.24 = 6.76 \text{ m}^4$$

(3)

Where,
- $I_{\text{girder}} = bd^{3} / 36 = 12 \times 3^{3} / 36 = 9 \text{ m}^4$
- $I_{\text{void}} = bd^{3} / 36 = 8.48 \times 2.12^{3} / 36 = 2.24 \text{ m}^4$

$y$ is the distance between the centroidal axis and the top surface of the deck, $y = 1$m. $M$ is the moment at the anchorage point of the permanent stay cable due to the lifting of segment 2, as found in Eq. (4).

$$M = (W_{\text{seg}2} + W_{\text{lifting gantry}}) \times \text{Leverarm} \times \gamma \phi \times \gamma f_3$$

$$M = (900 + 2000) \times 4.5 \times 1.15 \times 1.10 = 16510 \text{ kNm}$$

(4)

Where,
- $W_{\text{seg}2} = 900 \text{ kN; } W_{\text{lifting gantry}} = 2000 \text{ kN; }$
- $\text{Leverarm} = 4.5 \text{ m; } \gamma \phi = 1.15; \text{ and } \gamma f_3 = 1.10$

Each of the deck’s two outer girder elements weighs 50t with the diaphragm element weighing 20t, Ref. [1]. The lifting gantry and launching girders are assumed to weigh a total of 200t. The centre of mass of segment 2 is 4.5m from the permanent stay anchorage section.

Satisfying Eq. (1), see Eq. (5).

$$\frac{P}{9} + \frac{P \times 1 \times 1}{6.76} - \frac{16510 \times 1}{6.76} = 0 \Rightarrow P = 9430 \text{ kN}$$

(5)

The minimum area of the stressing bars, $A_{ps}$, required is found in Eq. (6) by assuming high tensile steel stressing bars with yield strength of 1000N/mm$^2$, stressed to 60% of ultimate capacity. Considering the use of 50mm diameter stressing bars, the number of bars required is found by Eq. (7). 8No. 50mm diameter high tensile stressing bars are required.

Considering Figs. 7, 10, it can be seen that 8 temporary stressing bars are used - 2 pairs of two stressing bars on each of the bridges box girder’s top surface.

$$A_{ps} = \frac{9430000}{0.6 \times 1000} = 15720 \text{ mm}^2$$

$$15720 = N \pi (50/2)^2 \Rightarrow N = 8$$

(6)

(7)

4.7 Temporary Stay Cables Sizing

The minimum temporary stay dimension can be found by considering the removal of the temporary stressing bars supporting the first and second deck segments after the section at which the fourth permanent stay is anchored. This is the worst angle the temporary cable stay will be at, see Fig. 13.

The section where the fourth permanent stay is anchored, has a moment, $M$, due to the lifting the first and second segments, given in Eq. (8).

$$M = (W_{\text{segment}} + W_{\text{lifting gantry}}) \times \text{Leverarm} \times \gamma \phi \times \gamma f_3$$

$$M = (1800 + 2000) \times 3 \times 1.15 \times 1.1 = 14420 \text{ kNm}$$

(8)

![Figure 12: Stress analysis of deck for temporary stressing bar requirements](image-url)
Figure 13: Temporary stay to support deck

During the removal of the temporary stressing bars supporting the first and second segments, the temporary stay cables must be sufficiently prestressed so that there is an “additional zero tensile stress” at the pylon. This is required because although the maximum allowable tensile stress at the pylon is -1N/mm² (assumed), the stressing of the previous segments may have used this allowance. Therefore, Eq. (1) must be satisfied. It is assumed that the temporary stays are anchored at the centroid of the section, hence there is no eccentricity.

Satisfying Eq. (1) see Eq. (9).

\[
P = \frac{14420 \times 1}{6.76} \Rightarrow P = 19200 \text{kN}
\]  

But there are two temporary cable stays, see Eq. (10),

\[2P = 19200 \text{ kN} \Rightarrow P = 9600 \text{ kN}\]

The temporary stay angle of inclination to the horizontal and vertical are found as 25° and 21° respectively. The vertical angle is found by assuming the top of the pylon, where the temporary stay is anchored, is 26m above the deck and 10m out of plane of the deck. Tension in temporary stay cable, \(T\), is then found by geometry in Eq. (11)

\[T = \frac{(9600/\cos 25°)}{\cos 21°} = 11350 \text{ kN}\]

The minimum area of the temporary stay cable, \(A_{ps}\), required is found in Eq. (12) by assuming high tensile steel stressing wires with yield strength of 1700N/mm², stressed to 60% of ultimate capacity. Considering the stay is formed from bundles of 15.7mm diameter seven strands, a 74No. strand bundle of 15.7mm diameter strands is required. See Eq. (13).

\[A_{ps} = \frac{11350,000}{0.6 \times 1700} = 11130 \text{ mm}^2\]

\[11130 = N \times [7 \times \pi \times (15.7/6)^2] \Rightarrow N = 74\]

5 Preliminary Static Analysis and Loadings

The Wadi Abdoun structure is many times redundant. This indeterminacy makes it difficult to analyse the bridge without software. That said, a static analysis can be carried out for preliminary sizing.

Preliminary analysis will consider a continuous deck over the cable support points and applying the most adverse loading combinations, see Fig. 14. The deck and pylons bending stiffness’ will be ignored and assumed to not deflect. Ignoring the bending stiffness of the pylon seems unrealistic; however, this will be far less than the axial stiffness of the cable stays, allowing the assumption that each backstay has the same force as its corresponding forestay. The analysis considers only span with the straight horizontal alignment.

Figure 14: Compression and tensile structural members

5.1 Traffic Loading

The Wadi Abdoun is a dual two-lane carriageway road bridge. Each of the four marked lanes is 7.28m wide. For loading calculations, each carriageway is 14.56m wide, so that each carriageway has 4 notional lanes. Therefore, there are a total of 8 notional lanes.

5.1.1 HA Loading

The loaded length of the bridge is 390m. Hence, the nominal un-factored HA UDL is 9kN/m of notional lane.

For HA Loading, two of the bridges notional lanes have full HA loading, with the other six lanes having 1/3rd of the full HA loading. A knife edge load of 120kN per width of notional lane is positioned to act transversely along the deck at the most adverse position.

5.1.2 HB Loading

A HB vehicle has 4 axels. Full HB loading is 45 units, where a unit represents an axel load of 10kN. Therefore for full HB loading, each axel load is 450kN.

5.1.3 Combining HA and HB Loading

HA and HB Loading can be combined to give the most adverse loading combination for the considered structural members. Worst case assumed as: 1 lane with full HA Loading; 6 lanes with 1/3 HA Loading; 1 lane with a HB vehicle of length 9.6m acting midspan between cable stay support points; and all lanes with a KEL acting midspan between the cable-stay support points.

5.2 Superimposed Dead Loading

The superimposed dead loading on the bridge (pavement – 75mm asphalt, Ref [1], edge parapets, crash barriers) is assumed as 2kN/m². This corresponds to 58kN/m longitudinal length for the 29m wide deck.

5.3 Wind Loading to BS5400

Wind traveling transversely across the bridge deck induces torsional motion in the deck section and causes the deck to bend vertically due to the vertical component of the wind. The Wadi Abdoun’s deck alleviates the wind forces and consequent effects by having a relatively streamlined aerodynamically stable deck cross section. This reduces vortex induced vibrations, plunging/valley effects, and lessens the drag forces.
The sinuous horizontal road alignment means that the deck profile certainly banks gradually round corners, which increases the wind induced plunging effects – though it is unknown if the analysis considered this effect. Inclining the stays transversely toward the deck certainly reduces the transverse deck movement, akin to a cradle.

Cables in the stay system are prone to oscillation due to wind loads, though in most instances, certainly with the Wadi Abdoun, dampeners fixed between the stays and their anchorage points mitigate oscillations.

Maximum wind gust on the bridge is found in Eq. (14).

\[ v_c = v \times K_1 \times S_1 \times S_2 = 63 \text{ m/s} \]

Where, 
- \( v \), the mean hourly wind speed is 30m/s
- \( K_1 \), the wind coefficient assumed as 1.59
- \( S_1 \), the funneling factor assumed as 1.00
- \( S_2 \), the gust factor assumed as 1.32

5.3.1 Transverse Wind Load
Transverse wind load on the deck is \( P_c \). See Eq. (15).

\[ P_c = q \times A_t \times C_D = (3.4 \times A_t) \text{kN} \]

Where,
- \( q = 0.613 v_c^2 = 0.613 \times 65^2 = 2590 \text{ N/m}^2 \)
- \( A_t \), the horizontal projected area in m²
- \( C_D \), the drag coefficient is 1.3

5.3.2 Vertical Wind Load
Vertical wind load (uplift or downwards) on the bridge is \( P_v \). See Eq. (16).

\[ P_v = q \times A_t \times C_L = (1 \times A_t) \text{kN} \]

Where,
- \( q = 0.613 v_c^2 = 0.613 \times 65^2 = 2590 \text{ N/m}^2 \)
- \( A_t \), the plan area of the deck in m²
- \( C_L \), the drag coefficient is 0.4

5.4 Permanent Cable Stay Design
The two planes of stay cables that support the deck are assumed to be fixed to pylons arms and deck with fixed and adjustable anchorages respectively. However, Ref. [3] suggests that the cables pass through steel tubular saddles in the pylons. This implies that the cable stays are continuous over the saddles, so that the back and forestays are continuous. Although this would permit the use of a fixed anchorages on one side of the pylon and adjustable anchorages on the other, the friction at the saddle would be huge. This means that when the stays are adjusted, the friction at the saddle will exert a huge horizontal force on the pylon arm which is not ideal. Further, the stays will have to curve around the saddle. The maximum radius, \( R \), required so that the bundle does not yield can be found by considering Eq. (17).

\[ \sigma = \frac{E}{\gamma} \Rightarrow R = \frac{E \cdot \gamma}{\sigma} = \frac{200000 \times 7.8}{1020} = 1530 \text{ mm} \]

Where,
- \( E = 200000 \text{ N/mm}^2 \), youngs modulus of steel
- \( \gamma = 7.8 \text{ mm} \), radius of bundle
- \( \sigma = 1020 \text{ N/mm}^2 \) Stressed to 60% of \( \sigma_{\text{yield}} \)

The lack of feasibility of allowing the stays to pass continuously over saddles in the pylons is apparent by the bundles being required to curve over the saddles at a large radius of at least 1530mm. Hence the assumption that the stays terminate at the pylon and deck seems sensible.

5.4.1 Loading Combination 2 – Longitudinal Bridge Load
The harp cable stay arrangement means that all stay cables will have the same design force. Loading Combination Number 2 gives the highest tensile force in the permanent stay.

Dead Load is 300kN/m deck length \( \rightarrow 36000 \text{kN over 12m span} \).

Factored Dead Load =

\[ 3600 \times (\gamma_f = 1.15) \times (\gamma_f = 1.10) = 4555 \text{kN} \]

Superimposed Dead Load is 58kN/m length of deck \( \rightarrow 696 \text{kN over 12m span} \).

Factored Superimposed Dead Load =

\[ 696 \times (\gamma_f = 1.75) \times (\gamma_f = 1.10) = 1340 \text{kN} \]

Traffic Load is (Full HA = 1 x 9 x 12) + (Reduced HA = 6 x 1/3 x 9 x 12) + (HB = 1800) + (KEL = 8 x 120) = 3084kN over 12m span.

Factored Traffic Load =

\[ 3084 \times (\gamma_f = 1.10) \times (\gamma_f = 1.10) = 3730 \text{kN} \]

Vertically Down Wind Load is 1kN/m². Assuming deck width is 30m \( \rightarrow 1 \times 30 x 12 = 360 \text{kN over 12m span} \).

Factored Vertically Down Wind Load =

\[ 360 \times (\gamma_f = 1.10) \times (\gamma_f = 1.10) = 435 \text{kN} \]

Transverse Wind Load is 3.4kN/m². Assuming deck and parapet height is 4.5m \( \rightarrow 3.4 \times 4.5 \times 12 = 180 \text{kN over 12m span} \).

Factored Transverse Wind Load =

\[ 180 \times (\gamma_f = 1.10) \times (\gamma_f = 1.10) = 220 \text{kN} \]

5.4.2 Total Factored Loading over 12m span
Total Loading acting Vertically Downwards =

\[ 4555 + 1340 + 3730 + 435 = 10060 \text{ kN} \]

Total Transverse Loading = 220kN

5.4.3 Design Tension in Permanent Cable Stay
Consider two notional vertical forces, \( 2P \), placed where the two considered permanent stay cables are attached to the deck, which the two stay cables need to support alone. See Eqs. (18,20).

\[ 2P = 10060 \Rightarrow P = 5030 \text{kN} \]

Consider a notional horizontal force, \( H \), placed where the considered stay cable is attached to the deck, which one stay cable need to support alone (the cable on the other side slackens due to horizontal load). See Eqs. (19,21).

\[ H = 220 \text{kN} \]

Tension in permanent stay cable, \( T \), found by geometry. See Eq. (22). Permanent stay angle of inclination to horizontal and to the vertical are \( \theta_H = 23^\circ \) and \( \theta_v = 21^\circ \).

\[ T = \frac{(5030/\sin 23^\circ)}{\cos 21^\circ} + \frac{(220/\sin 21^\circ)}{\cos 23^\circ} = 14460 \text{kN} \]
The minimum area of the permanent stay cable, $A_{ps}$, required is found in Eq. (23) by assuming high tensile steel stressing wires with a yield strength of 1700N/mm$^2$, stressed to 60% of ultimate capacity. Considering the stay is formed from bundles of 15.7mm diameter seven strands, a 95No. strand bundle of 15.7mm diameter strands is required. See Eq. (24).

$$A_{ps} = \frac{14,460,000}{0.6 \times 1700} = 14180 \text{ mm}^2$$

$$14180 = N \times \left[ 7 \times \pi \times (15.7/6)^2 \right] \Rightarrow N = 94.2$$

5.4.4 Ensuring Cables are sized for Sufficient Redundancy

During the life of the bridge the stay cables may need to be replaced. Therefore there should be sufficient redundancy in the cable system to allow replacement.

It is unlikely that the stays will have been designed to allow an entire stay to be replaced in one go. Such a design decision would mean that the two cables next to ‘missing’ cable will have to temporarily support the ‘missing’ cable’s loads. Therefore, all cables will have had to be designed to support approximately 1.5 times its usual design load, leading to over designed cables.

More likely is that a few strand bundles at time will be taken out and replaced until all bundles are replaced. Therefore, it is suggested that the number of strands required in the bundle, as found in Eq. (24), should be increased by five, allowing for the safe replacement of five bundles at a time. Therefore the stays should be a 100No. strand bundle of 15.7mm diameter strands.

The bridge’s stays were actually all 73 strand bundles of 15.7mm diameter strands, Ref. [1]. The discrepancy between the reality and the calculated is due to differences in analysis techniques, assumptions made, and certainly the design loads used. Indeed, bridges built abroad certainly do not use UK Loading as it is higher, than for example, the American codes.

5.4.5 Self Weight of the Cable

It should be noted that the self weight of the cable stays were ignored. If their weight had been included the design tension in the cables would increase, and the compression in the tower would also increase.

5.6 Deck Design

A preliminary assessment of the bridge deck stresses can be estimated by considering the deck is continuous over the cable stay support points.

5.6.1 Bending Stresses

Loading Combination 1 causes the highest bending moment in the deck. Similarly to the definition of the most critical loading combination for cable-stay design, the factored ULS longitudinal bridge loading on the deck, $W_{uls}$, is calculated in Eq. (25).

Factored Total Loading, $W_{uls} = [ (\gamma_{fl} = 1.15) \times \text{Dead Load}] + [ (\gamma_{fl} = 1.75) \times \text{Superimpose Dead Load}] + [ (\gamma_{fl} = 1.3) \times \text{Traffic Loading}] \times (\gamma_{fl} = 1.10)$

$$W_{uls} = [ (\gamma_{fl} = 1.15) \times 300] + [ (\gamma_{fl} = 1.75) \times 58] + [ (\gamma_{fl} = 1.3) \times 257] \times (\gamma_{fl} = 1.10) = 860 \text{kN/m}$$

The maximum hogging moment in the deck, $M_{ult \, hog}$, is found by applying $W_{uls}$, found in Eq. (25), to the maximum deck span between cable supports of 12m, see Eq. (26). The resulting bending stresses found in Eq. (27) are very small. Indeed, the section is likely to never go into tension under normal loading since compression stresses in the deck due to the tension in the stays is huge in comparison.

$$M_{ult \, hog} = \frac{W_{uls} \times L_{span}^2}{12} = \frac{860 \times 12^2}{12} = 10320 \text{kNm}$$

$$\sigma = \frac{M_{ult \, hog} \times y}{I_{deck}}$$

$$\sigma_{top} = \frac{10320 \times 12}{6.76} = 1.5 \text{N/mm}^2 \quad \text{(tension)}$$

$$\sigma_{bottom} = \frac{10320 \times 2}{6.76} = 3.1 \text{N/mm}^2 \quad \text{(compression)}$$

5.6.2 Compressive Stresses

The cable-stays induce a high level of compression into the deck. This stiffens the deck considerably. Indeed, the fact that the cables are inclined transversely to deck, mimicking a cradle, helps restrain lateral movement of the deck. However the deck does not want to be too stiff as it needs to be flexible under seismic loading.

The maximum tension in each cable-stay is 14460kN, see Eq. (22). This induces huge compression in the deck. The maximum compression, $F_{comp}$, in the deck due to 10 stays can be found by geometry. $F_{comp}$ acts over the decks cross sectional area. The corresponding compressive stress is found in Eq. (28). It is assumed that the permanent stay cables are anchored at the centroid of the section, hence there is no eccentricity.

$$F_{comp} = \frac{10 \times \cos 21^\circ}{\sin 23^\circ} \times \frac{10 \times 14460 \times \cos 21^\circ}{\sin 23} = 345500 \text{kN}$$

$$\sigma_{comp} = F_{comp} / A_{deck} = 345500/9 = 38.4 \text{N/mm}^2$$

5.6.3 Thermal Expansion and Expansion Joints

Temperature variations will cause the deck to move longitudinally. Any expansion and shrinking may be allowed by expansion joints, assumed at the bridge abutments. This assumption is based on there being no apparent “ideal” locations for expansion joints in the spans as prestressing tendons are assumed to run continuously through the length of the deck.
Expansion joints can become blocked, which prohibits expansion. Therefore, even if expansion joints are used, it is conservative to assume they are blocked. Therefore, the expansion joints may as well not be used.

Further, no expansion joints in the Wadi Abdoun’s deck span may be allowed because the deck has a flexible curved horizontal alignment, that allows some deformation due to expansion of the deck’s length. Therefore, the compression in the deck due to thermal expansion will be less than, for example, if the bridge had a straight alignment that could not deform as easily.

Further evidence of their being no expansion joints in the deck is the use of flexible twin leaf pylons, which permits longitudinal movement and so reduce the longitudinal compressive stresses due to expansion.

To calculate the maximum compressive stress in the deck due to temperature variations it is assumed that the deck does not deform/buckle; the temperature across the entire deck cross section increases by 25°C; and that there are no expansion joints.

The longitudinal strain of the deck due to temperature changes is \( \varepsilon = \alpha \times \Delta T \), where \( \alpha = 12 \times 10^{-6} / ^\circ \text{C} \) is the concrete coefficient of thermal expansion and \( \Delta T = 25^\circ \text{C} \) is the change in temperature of the deck. See Eq. (29).

The deck is assumed to not be able to expand, therefore the apparent compressive stress from the extension being prohibited is \( \sigma_{\text{apparent comp}} = E \times \varepsilon \), where \( E = 30000 \text{N/mm}^2 \) is concrete’s Young’s modulus. The compressive stress is found to be 9 N/mm². See Eq. (30). This compressive stress is an unfactored load.

\[
\varepsilon = 12 \times 10^{-6} \times 25 = 0.0003 \text{ strains} \quad (29)
\]

\[
\sigma_{\text{apparent comp}} = 30000 \times 0.0003 = 9 \text{ N/mm}^2 \quad (30)
\]

### 5.6.4 Total Design Compressive Stress

The maximum total compressive stress in the deck cannot be found by simply summing the calculated compressive stress contributions from the bending of the deck, the stay cables and thermal expansion because different loading combinations were used. However, it is anticipated that combination 3 will give the highest compressive stress in the deck, estimated at 45 N/mm². Prestressing tendons also run throughout the deck, and will certainly increase the maximum design compression in the deck to 50 N/mm². Therefore, the pre-cast segments of the deck should be cast from C50 concrete.

### 5.7 Pylon Design

The pylons would have been designed for the M-N interaction. The maximum compression will be from applying the highest arrangement of deck loads. Bending in the pylon will be from applying maximum loading to one side of the deck, with minimal loading on the other side, with the difference in loading causing bending in the pylon. Indeed, the pylons required internal prestress to “accommodate the out of balance deck loads”, Ref. [3].

### 5.8 Creep and Deflection

The Wadi Abdoun’s deck has relatively short spans between cable support points of 12m. Therefore, creep and deflection are not an issue. Indeed, a calculation for the maximum sag between the cable support points due to long term creep was found to be negligible at 19mm. Therefore, the deck segments need not have a precamber.

### 5.9 Seismic Loadings and Design Features

The Wadi Abdoun sits on a major fault line. Assessing the bridge’s seismic response is difficult due to the complex 3D interaction of the structure. In simple terms, a flexible structure reacts better to seismic loadings than a rigid one, Ref. [2]. Indeed, although cable-stayed bridges are inherently flexible, the Wadi Abdoun has design features which further improve its seismic performance. For instance inclining the cables transverse to the deck for lateral stability.

A continuous deck, with no expansion joints, as assumed, is best for seismic design. If expansion joints are used, seismic induced motion can cause unseating and crushing of the deck.

Twin leaf pylons ensure the pylons are flexible against lateral movements. Indeed, it is unknown whether the deck is “rigidly” attached to the pylons, however it would be best if the deck was able to pass freely over the pylons, maybe with Teflon pads, to allow the cables to absorb any seismic energy rather than the pylons resisting the lateral forces.

### 6 Other Suggested Improvements

Although the Wadi Abdoun was successfully constructed, and stands as a fantastic feat of engineering, the challenges posed by the ‘S’ shaped horizontal alignment has required many innovative design decisions to be made. The structural confusion and challenges could have been eased if the alignment was straight. This would have lessened the accuracy required for geometrical control and led to quicker and cheaper overall construction. That said, a straight alignment would certainly have not made such a dramatic and unusual bridge. Further, it may not have had the same seismic flexibility allowed by the curving alignment.

Since the ‘y’ shaped pylons require the deck to sit between the pylon arms, there seems to be no obvious solution for widening the deck, save the deck is elevated. This would keep the cable stays in plane with the arms of the pylon as required. However, this may be unfeasible as the bridge pylons would certainly not have been designed to carry the extra load.

The bridge has no pedestrian footway, and since the bridge connects across the valley, people will certainly cross the bridge by walking alongside the parapet, which is obviously dangerous. This could have been avoided, simply by including a pedestrian footway.

### References

