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A CRITCAL ANALYSIS OF THE SERI SAUJANA BRIDGE (2002), PUTRAJAYA, MALAYSIA

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Abstract: This article provides an in depth critical analysis of the Seri Saujana Bridge, situated in Putrajaya, Malaysia. The main criterion for the analysis will be broken down into separate headings, each category has been researched and then summarised using my own personal preferences and engineering judgement.

Keywords: *Seri Saujana, cable-stayed arch,*

1 Background Information

The bridge was designed by Michael Yamout of the Malaysian consulting engineering and architecture firm PJSI consultants as part of series of ten bridges built to access the core island within the new governmental city of Putrajaya, commissioned by Putrajaya Holdings. The scope of these bridges was to reflect the modern architecture, each employing a different structural form for bridging the artificial island of a potential 300,000 occupants with the surrounding area. The main contractor was the road builder Hasrat Sedaja JV, the pre-stress contractor was BBR Construction Systems Malaysia, which Pascal Klein, of Klein Engineering assisted with the cable stressing process.

The bridge was completed in 2002, 32m wide and spanning 300m, as the first large scale use of a cable-stayed arch structural scheme. This revolutionary form provokes much response from engineers, residents and tourists alike.

2 Aesthetics

The main requirement of this bridge, as with many of the other modern bridges built in the Putrajaya area is to make a huge impact aesthetically, providing an ideal postcard and photo opportunity for their tourism industry.

The purpose of this bridge is to cross the artificial Putrajaya Lake connecting the main island (precinct 4) with precinct 7 from the south, via a new highway which also links the island with the Kuala Lumpur International Airport. The function of the bridge is therefore to carry a high volume of vehicles and pedestrians. The bridge provides dual three lane carriageways for vehicles, with footpaths running along the extremities.

The bridge therefore fulfils its primary function. Structurally it is difficult to declare whether the bridge fulfils its function, as the bridge is unique in that it uses a combination of two different structural schemes. If you were to assume either scheme alone, they are both very simple and effective ways of spanning across the lake, and each scheme has successful precedent bridges (inclined arches has the Oudry-Mesly Bridge by Calatrava, inclined pylon cable-stayed has the Rio Ebro bridge by Carlos Fernandez Casado S.L) which both schemes employed in this bridge are very similar to. The arches and pylons all have sufficient thickness to display to the observer that they provide the stiffness and strength to support the deck, while the cables slenderness displays their obvious function as tension members. The thin deck is also correctly suggesting to the observer that it relies on the cables for support. Structurally in this bridge the pylons are not providing much stiffness, but aesthetically its thickness is justified in that it is still the main route of vertical loading to the ground.

Looking at the bridge in elevation I therefore believe that either of the schemes appear functional, to the extent that you could believe that there is two separate bridges in your view, one in front of the other. It is fully conceivable in appearance that one of systems alone could support the deck.

This raises the issue regarding the reasons for using these two schemes, if one alone fulfils the function simply and efficiently, why add the extra cost, and the possibility

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of making certain structural elements of the bridge redundant by combining two simple yet effective schemes.

This was justified by the designer in defining the bridge as sculpture where cost and maximum efficiency was not an issue, which is believable seeing as its function is to cross an artificial lake. So if cost was a problem, the lake instead could have been avoided in the first instance. The bridge would serve little purpose if the chance is created for a unique series of bridges, and this was avoided with a common solution.

The proportionality of the structural members I believe to be well balanced. None of members appear to be oversized in relation to the span achieved. The deck has a section which induces a shadow below the soffit level in elevation, giving it the appearance in most lighting conditions that its depth is the solely the soffit, which is incredibly slender. The stay-cables are narrowly spaced, allowing each individual cable to be very slender, and from a large distance from the bridge appear almost transparent, leaving only what appears to be very slender traditional looking tied arch bridge. The inclined hangers holding the wings of the deck from the arches are a larger diameter than the cable stays, even though I believe in serviceability loading at least, they take less loading than the cable stays. This is reasonable, as they are a larger spacing than the cable stays, therefore they would look out of proportion if they were thinner.

The order of this bridge I believe has been compromised by the complexity of using two different structural forms. Looking at either system alone and each has been organized very well. The single plane of stay cables in an asymmetric 'semi-harp' arrangement is a relatively modern application, and is often as it is on this bridge, simple, clearly defined and beautiful. The two inclined arches show a similar elegance, with the bracing members between the arches showing ideal proportion and symmetry.

It is when these two systems are combined I believe it loses some of the simplicity, in true elevation it appears as if there are two independent bridges, one in front of the other, which I have no issue with. In fact your eye flows along the bridge very well, as the stay cables guide your eye seamlessly from the pylons into the line of the arch. This is probably a trick caused by the density of the cables being greater as they leave the pylon than when they spread out along the deck, this fan effect makes the cables less visible than at the point they leave the tower. Your eye then tends to concentrate on the arches.

As the bridge is symmetric, visually there are no obstacles in viewing the full length of the bridge. When the bridge is viewed from other angles, there become a few issues. Firstly the back stays are unlike the main stays in that there are two separate planes of cables, reaching anchors either side of the carriageway. As these planes of narrowly spaced cables meet at the same pylon, it is easy to consider and observe the untidy crossing which occurs when viewing from oblique angles. However, using two planes of cables provides an interesting visual approach to the bridge, as you can observe the spectacular triangular stay arrangement which channel you towards the bridge. There are also structural and construction benefits in terms of providing the pylon with some lateral support, and separating the ground anchors away from the central reservation.

The other issue is how the main stay cables must pierce the plane of the two arches to reach deck level. In doing this they have to pass between the bracing members. This produces a crowded interlocking effect (fig. 1), making you think 'how are these not colliding'. The only way you could reduce this effect when combining these schemes would be to have the stays at wider intervals, hence reducing this congestion as they cross the bracing. This would however have subsequent impacts on the sizing of the deck and cables, and knock on effects on the aesthetics of the bridge from other viewpoints.

Figure 1: Stays crossing bracing members

The major refinements implemented on this bridge are the inclined pylons, inclined arches and asymmetric arrangement of the cable stays. These are all clever tricks previously applied to modern bridges for their aesthetic benefits. The aesthetic success of a cable-stayed bridge is often governed by the pylons appearance, so the economic cost of inclining the pylon is fully justified. Interesting many cable stay bridges use a central grove cut down the front and/or side faces of their pylon to emphasise the presence of anchors, and especially on the side give the impression of slenderness by the introduction of shadows. I see no reason why this refinement has been overlooked, and not used on this bridge.

The way this bridge integrates into its environment is its major design achievement. As previously explained the topology of the area this bridge is upon is artificial. Unlike many of the bridges built in Europe, this bridge was being constructed at a time where it would set the tone for the surrounding environment. In Europe many bridges are built to suit established well developed environments, or the environment is less man-made in which case it must simply suit the valley or river it spans.

This bridge employs two schemes which are very appropriate for spanning a lake, in particular the most successful application of a cable stayed bridge is often over span of water. The overall appearance is of a very modern, highly engineered, yet slender bridge. This form is similar to the other bridges commissioned around the island. Together these bridges have blended well with the modern, highly engineered architecture within the island, helping to promote the city as modern and technically advance. In an area like this it would seem inappropriate to build a simple concrete slab bridge on series of piers, and would be wasted opportunity to try something new.

Although there are many sculptural over-engineered bridges in Europe, I do not believe this bridge would fit in, especially within England. Its appearance is too bold and over the top compared with our surroundings and views. This comparison holds true for buildings and even vehicles.

All the finishes on the bridge are smooth and painted producing a series of clean lines. Which is an attribute often associated with any modern consumer product.

The colour of this bridge has been chosen not solely based on aesthetics due to the impact it has on certain elements to temperature changes. The facia along the deck, and the main stay cables are a pure white. On the deck this helps to give the impression that the decks thickness is purely the facia covering the parapet, as the depth of concrete is left in its shadow. As the stay cables are thin the colouring allows them to appear transparent against the sky in some lighting conditions, while at night against a dark sky they are well lit, hence very noticeable. The back stays are blue, and I am not sure on the reason for this, but their visibility does vary in different lighting conditions. Perhaps it is an attempt to make only the main stays or the back stays visible at one instance, giving the pylon a cantilevering presence similar to a few of Calatrava's bridges. The pylons are white up to 42m, then black/graphite to the peak at 73m, I believe this is simply to show that a concrete was used for the lower portion and to distinguish the composite section above 42m. The arches and hangers are coated in a reflective paint, which up close looks like a grey finish, but when viewing the bridge from a distance appears close to white. These elements are sufficiently thick to be seen in any lighting condition, displaying no intention of hiding them.

I personally believe this bridge has a great deal of character. This is due to its unique structure, and way this provokes the viewer, whether they are educated in engineering or not, to take notice of the bridge and question it. If you have no knowledge of engineering you will see the bridge as a 'one-off', and cannot help to appreciate its appearance, even though many will see it as over the top. While an engineer will debate its purpose, and the reasons for combining the two bridge forms. In my opinion there is no way it can be perceived as dull, and I believe it shares similar qualities to many of Calatrava's bridge designs, where cost, logic, and simplification are often overlooked in order to produce unique bridges, which are rich in character.

There are certain issues with the complexity of this bridge which will vary depending on personal preference. I believe this bridge crosses the line in terms of achieving a unique character, resulting in its structure becoming overly complicated. Again if you go back to looking at each form alone, by the inclination of the pylons or arches, they have the necessary character and refinement to be aesthetically pleasing on their own. Combining them has added chaos from certain view points, as mentioned before where the stays need to pass through the arch.

I do not believe the requirement to imitate nature was ever an aspect that was considered during design, mainly because the bridge is in a heavily man-made environment. However, it can be seen that the inclined arches employ a structural form with obvious comparisons to a spine. I doubt whether this was necessarily an intentional reproduction of a natural form, it is more likely due to how effective this form has been on previous bridges, which have been used as precedents.

In conclusion, I have analysed the bridge against Fritz Leonhardt's guidelines that he believes governs whether aesthetically a bridge is a success, and the bridge fares well in the criterion it was aiming to meet. The radical concept and complexity of the bridge will lead to mixed opinions. Personally I like how this bridge looks, without necessarily liking the bridge itself. The following figure shows a night view which I personally find hard to find fault (fig. 2).

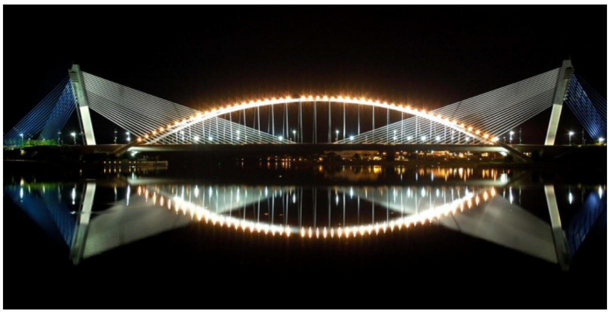

Figure 2: Bridge Lighting

3 Loadings

The loading and analysis will have been a complex process, as the effects of the different load cases will not have the same impact on both structural forms. For example a rise in temperature will have contradictory effects on the two systems.

I am not sure which series of loading codes, or building standards this bridge was designed to meet, but I am certain that the following loads will have required checking, dead, super-imposed dead, traffic, pedestrian, wind, temperature, creep, and earthquake. While I suspect in this case dead (including super-imposed) loading combined with vehicle loading and the large variations in temperature will have been governing the design process.

Where I have suggested loading guidelines for sizing these are likely to be British Standards (BS), which may not have been the standards used to design this bridge.

3.1 Dead and Super-Imposed Dead

The deck is the main element contributing to the dead loading of the bridge. A typical section through the deck is shown in fig. 3. The section is in-situ concrete with a symmetric series of longitudinal boxes which I would assume is uniform in terms of concrete along it span. The cables dead load can also be significant, as they can cause bending within the cables.

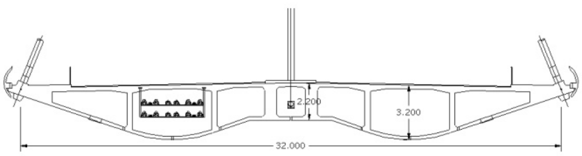

Figure 3: Typical Section Through Deck

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The super-imposed dead loading will consist mainly of the thin layer of fill and bituminous surface layer. There are also continuous lines of facia and rigid parapets/rails running the length of the bridge which are likely to be added to the contribution of services such as lighting and other cabling or tubing, and approximated as a uniformly distributed load. Within the voids in the profiled box sections of the deck, it would be wise to assume a worse case condition of these filling with water, rather than assume the most probable case that they are simply air voids. The tubular arch sections will also have voids, but it would be less likely that these would build up with water. All the super-imposed loading is difficult to distribute in terms of the exact positioning and weight hence these will have high safety factors associated.

3.2 Traffic and Pedestrians

The total width of this bridge is 32m, with three marked lanes going in either direction. If you assume that the central reserve in not capable of accommodating any traffic due to the cable stays, and the raised curb and inclination of the arch hangers would prevent vehicles from passing under them it is reasonable to assume a large proportion of the footway along either side of the bridge could never be used or converted to road. The potential carriageway width is therefore approximately 12m either side of the central reserve. This suggests that should the bridge have been analysed using BS, there will have been a total of 8 notional lanes with a width of 3m, even though only 6 are marked. There are many loading combinations of HA and HB loading, but seeing as I am unsure whether this bridge was designed to these codes, and the differences in structural systems, I can only suggest that these traffic loads (whether to BS or otherwise), were placed in the most adverse positions, rather than try to attempt to suggest all the possible load combinations. The HA loading I will apply in my hand calculation has been worked out using equation (1).

$$HA = \left(\frac{1}{L}\right)^{0.1} \tag{1}$$

$$HA = \left(\frac{1}{300}\right)^{0.1} = \frac{20.4kN/m}{lane} = 6.8kN/m^2$$

The bridge is straight therefore there should be no centrifugal loading. Skidding and collision loading will apply. As the parapets have been designed as rigid to protect the pedestrians, the loading transferred into the deck will also have be to be considered. This will probably not be part of a major load case, but it may be an important check to access what damage such an impact would have the structure as a whole.

The footpath along the edges of the bridge will have a comparatively small live load (less than 5kN/m^2). As the bridge is predominantly for vehicle use, the vibrations caused by pedestrians will most likely be insignificant.

3.3 Wind

The wind pressures acting upon the bridge, either in elevation area, or in terms of uplift or downwards pressure on the deck, will certainly have been calculated corresponding to local design guidelines. While the deck is relatively low in terms of distance from ground, the pylons, cables and some of arch structure are pretty high. The surrounding topology is flat and exposed, therefore I would imagine high wind pressures are possible.

The parapet has a solid facia, so the overall decks depth in elevation is approximately 5m. This I would imagine is not greater than the maximum height of live load, so the depth of maximum potential live load will need to be considered in calculations. Although a depth can be assumed by this method, I suspect that as the deck has specifically been designed as an aerofoil and it exceeds 200m, wind tunnel tests will have been carried out for accurate pressures rather than using basic assumptions from codes.

The effect of wind on the deck will not be limited only to pressure loading, wind-induced vibration will also need to be checked with a dynamic analysis, an effect often associated with less stiff suspension bridge decks.

3.4 Temperature

Putrajaya is situated very close to the equator, it will therefore experience a huge range of temperatures, both on the annual and daily cycle. The overall expansion or contraction of the bridge's structural elements, or the differential expansion or contraction of a single element (most likely the differences in temperature experienced within the deck) will need checking. There are guides available for finding the temperature variations in a concrete deck, which can be used to find an induced stress distribution in the section, as the deck is fixed both axial and cambering effects will be present.

3.5 Other Loading Conditions

Putrajaya is also situated in an area of high seismic risk. Therefore the structure will need to be checked for the required lateral loading which could be expected, the magnitude of which will most probably have been chosen for the same return period as the temperature and wind loadings.

Creep and shrinkage of the concrete will also have been considered, even if this was to simply eliminate its effects by considering the construction.

The differential settlement of the foundations is unlikely to be a large issue for this specific bridge, even though the bridge structure is indeterminate. It is likely that due to the man-made conditions this bridge is built in, the soil under the foundations will have extensively tested making it hugely predictable, if not brought in specifically as founding soil.

4 Bridge Structure and Strength

The overall structural scheme is of a continuous pre-stressed concrete box girder deck spanning between a singular central plane of cable stays, and two planes of hangers running parallel to the stays along the edge of the deck. The stays are supported by inclined concrete pylons, which in turn are anchored by two planes of back stays. The hangers are suspended by two steel inclined arches, which ends are tied through the deck. Both pylons and arches sit on abutments at either side of the lake. The scheme can be seen in fig 4 which is the computer model of the bridge.

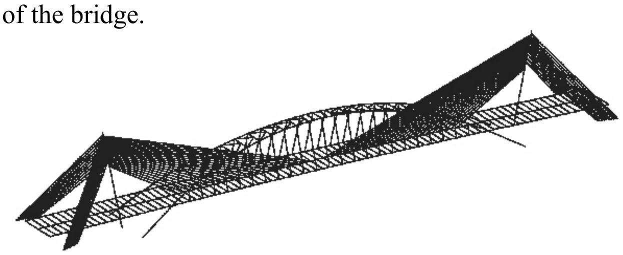

Figure 4: Structural Scheme

4.1 Cable-Stayed System

The cable stay system for supporting a deck is itself a relatively modern bridge form, the system employed in this bridge is not initially obvious.

Firstly there will have been a debate on the number of spans required for a total span of 300m. For this distance it would be common to use three, with the piers spaced proportionally close to the embankments, with a larger main span. In many cases there are tables/graphs which can be referred to in order to achieve the most economic arrangement of bridge and element variables such as spans, heights etc. In this case there is only a single span, which I assume was to span the entire lake without the need for any intermediate piers. Without these side spans, it does make the analysis slightly more straightforward, as the loaded region is only the main span, the potential effects adverse or beneficial of loading the side spans are avoided, as this section of carriageway sits inland.

Traditionally the main stays and the back stays were symmetric, the cables often used sparingly and located in positions where a pier was undesirable. The back stays would be tied to deck, which reduces the amount of bending which occurs within the pylon, allowing the pylon be a slender element. This method transfers all of the vertical load into the pylons while the horizontal forces from the inclination of cables is balanced through compressive forces in the deck, which results in a large and often unattractively deep deck section, and had imposed a physical limit for the spans achievable by this form of cable stayed bridge. It is therefore simple to see that this type of cable stayed bridge is used where the length of deck required is proportionally short, as this is where the majority of material and cost is associated.

This problem was overcome by Ricardo Morandi who created portal framed towers in the shape of an inverted V, acting as cantilevers from ground level, which the stay system could effectively hang from. By having the pylon substantially stiffer than the deck the stiffness previously designed in into deck is transferred into the pylon. It was then the engineers Ulrich Finsterwalder and Fritz Leonhardt who pioneered the use of flexible decks on cable stayed bridges. To accommodate a slender section of deck, the bending moments acting in the deck had to be reduced. This was achieved by reducing the spacing of the stay cables. This system is found in many applications, especially long span, where the length of deck of often a governing cost, to be the more efficient than the continuous deck approach. The problem with this system is that the deck can be weak transverse bending, and suffer from wind induced vibrations, often associated with slender suspension systems.

The system used most recently for bridges of three spans or less, is a compromise of the above systems, where the back stays are anchored in to the ground rather than the deck, which is obviously not possible for multiple cable stayed spans. It is therefore possible to have a relatively slender deck, as there may be a reduced compressive force near the pylon if any of the back stays attach to a side span, but these are a lot lower than when all the back stays are tied into deck. If they are all tied back into ground then there should be no compressive axial forces in the deck. Depending upon the inclination of the cables it is likely that there will be tension generated in parts of the deck, which will peak at mid span (between the opposing directions of the main stays). The sizing of the deck therefore allows the spacing of the cables to be a compromise of the above systems. Where if the deck only experiences tension it would seem sensible to have a steel deck, while if the tension is low, and parts will still receive high levels of compression, additional steel pre-stress will need to be placed mid span. It is essential that the stays are heavily pre-tensioned as they are the predominately stiff element in this system.

The pylon can also be relatively slender, as it should predominately experience compressive forces. The pylon will experience a small amount of bending depending on how balanced the horizontal component of the cable stays and back stays are. Often the stays on this type of cable stayed bridge are asymmetric, which will induce some bending into the pylon, if the pylon is not inclined.

The Seri Saujana appears from visual observation to be the later of these types, it also appears this way from the computer model, but the article [1] written by the consultant explains that the earth anchors only counterweight the vertical force from the back stays, and the forces are balanced by tying the ground anchor back to the abutments, in a similar way to the Rio Ebro Bridge. As a concrete deck has been used, I have taken this as clarification that axial compression will be experienced in the deck, otherwise it defies logic to use a concrete deck, especially as it is also acting as a tie beam for the arches. My initial thoughts are that it would have been more sensible to design the earth anchors to resist horizontal loads using inclined piles, rather than just gravity from an earth filled box, and use a steel deck and design for tension in the deck.

The next design decision will have been to decide on the arrangement of stay cables. Seri Saujana uses a singular plane of main stays, which is often used for aesthetic reasons to avoid criss-crossing of cables. This can lead to inefficiently stiff deck sections to resist the torque caused by any imposed loading which will not act directly along the axis of the support, causing the deck to twist. For this bridge the effect is not a major issue as the deck is supported at its edges by the hanger cables. The spacing of the cables is very small, this reduces the bending moment in the deck, but amount of cables required results in the less efficient semi harp arrangement of stays, radiating from the pylon. This is often used on bridges with two or more stay planes to again avoid criss-crossing, but on this bridge it is a practical issue with having a large number of narrowly spaced cables converging onto one point, and by spreading them along the pylon leads to a simpler connection detail. Even with this detail, it was necessary on this bridge to use a steel collar, to house all the steel transfer ribs, rather than attempting to cast this in concrete.

Interestingly the back stays have been separated into two planes, diverging symmetrically from the central pylon to meet anchor blocks at either side the approaching carriageway. From certain angles these now cause criss-crossing due to issues concerning the way one plane will move from the pylon into the foreground while the other travels into the background. I can see a few structural benefits from this, as these will provide the pylon with lateral support from wind loading and seismic behaviour, where the deck may flex transversely pulling the cables slightly out of plane, and hence load the pylon in its minor axis as well as its major. The other rather obvious benefit is that twice the number of back stays can be achieved for a short span back to the anchors. Should these all occur in one plane, they would either need to be more closely spaced, or use larger diameter cable to handle the required tensions with the required stiffness. However on this bridge, the distance the backstays may diverge from the pylon is not an issue, so this does not seem like it would be governing reason. On a practical side, it is no doubt easier to cast and detail the anchors if they are located at either side of the carriageway, rather than in the central reservation. Overall the benefits structurally must have outweighed the aesthetic issues associated.

The bridge has inclined pylons, which is a refinement commonly used on pedestrian cable stayed bridges. It is structurally beneficial in that the inclination can allow the horizontal components of the stays to balance with an asymmetric stay arrangement, for the service state load case. This allows the pylon to be more slender than if it was vertical, as the resultant bending moments will be lower. The pylons seem reasonably sized for an element which under is its service load state at least, will only have axial compressive forces. It is worth noting that the pylon will have been designed to provide buckling resistance, and the extra loading and safety impacts of a vehicle collision loading. This can often result in what appear to be oversized members.

To initially size the members of the cable stayed elements within this bridge is not as simple as it would be for standard cable stayed bridge. Without knowing the relative stiffness of the cable stayed system and the arch system, loading would have to be distributed as you ideally like it occur when built, but as I will explain later this is a function of other variables such as temperature changes which in turn will be effected by the initial choice of member size and position, making this an iterative process.

I would assume that the stay system takes 50% of the deck's dead and superimposed dead loading and 75% of any traffic imposed loading (fig 6), because depending on the codes used there will be different levels of loading, (in British code HA and HB), and it is possible the heavier load could be positioned nearer the cable stay supports. For a rough hand calculation, the deck continuous over vertically fixed supports where the cable stays are expected is adequate. This assumes the cables will not relax elastically proportionate to their tension, these varying levels of support will often give a less even bending moment diagram (fig 5), as the longer cables will tend to relax greater, providing the deck with less support.

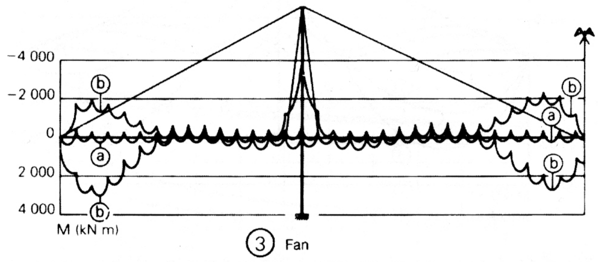

Figure 5: Example Bending Moment Distribution

For my hand calculations I will assume the heaviest loading is over the entire span, and none of the vertical loads are beneficial therefore should be fully factored (2). The tensions in the stay cables can be resolved to find the maximum compression in the deck, which combined with the bending moments, and the arch calculations can be used to initially size the deck. The sizes gained from these assumptions can be used in a plane frame computer analysis where more load combinations can be used to adjust the sizing for a 3D model. For example the worse load case for deck, maybe different to the worse case for pylon, but in hand analysis it would be impractical to vary the live loading and factoring over the deck.

The following simple calculation will consider the permanent dead and primary live loading. Assuming the dead load of the bridge as 12kN/m², and a super imposed load of 5kN/m², with the full HA loading in the two lanes nearest the cable stays, and a third of the HA loading in the remaining lanes. I have also made the assumption that as the deck is continuous over 22 stays in a half span, each 5m span of deck can be approximated as a fixed end beam.

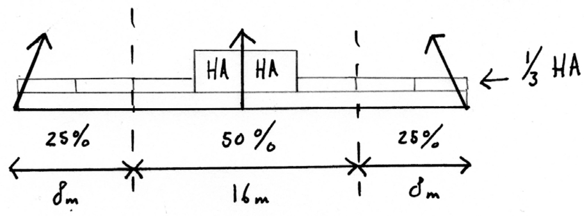

Figure 6: Assumed Load distribution (in section)

$$\begin{aligned} &ULS\ Design\ Loading \\ &= \gamma_{fl}.\gamma_{f3}.Dead + \gamma_{fl}.\gamma_{f3}.Super\ Imposed\ Dead \\ &+ +\gamma_{fl}.\gamma_{f3}.Live \end{aligned} \tag{2}$$

$$\begin{aligned} \omega = 16 \times (1.15 \times 1.1 \times 12 + 1.75 \times 1.1 \times 5) \\ + 2 \times 1.5 \times 1.1 \times 20.4 + \tfrac{1}{3} \\ \times 2 \times 1.5 \times 1.1 \times 20.4 \\ = 487kN/m \end{aligned} \tag{3}$$

$$V_{max} = \frac{\omega.l}{2} = \frac{487 \times 5}{2} = 1217.5kN \tag{4}$$

$$M_{hog} = \frac{\omega.l^2}{12} = \frac{487 \times 5^2}{12} = 1014kNm \tag{5}$$

$$M_{sag} = \frac{\omega.l^2}{24} = \frac{487 \times 5^2}{24} = 507kNm \tag{6}$$

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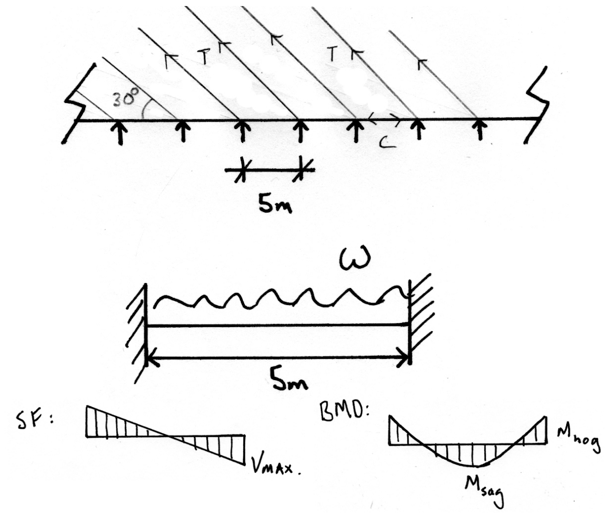

$$T = \frac{2.V_{max}}{sin\theta} = \frac{2 \times 1215.5}{sin30} = 4870kN \quad (7)$$

$$C = T.cos30 = 4870 \times cos30 = 4217kN \quad (8)$$

$$C_{max} = C \times no\ of\ stays = 4217 \times 22 = 930000kN \quad (9)$$

Assuming the ultimate strength (UTS), β, of the cable is 1700N/mm², the stays can be sized:

$$A_{steel} = \frac{T}{\beta} = \frac{4870000}{1700} = 2865mm^2 \quad (10)$$

Imposed vehicle loading over the main span will cause the deck to deform cambering downwards, this will induce tension in the main stay cables, which will try to pull the pylon towards the main span, which in turn will be resisted by tensions developed in the back stays. The benefit this bridge has over bridges with unsupported side spans is that all the back stays will experience tension variations, which helps to balance forces and prevent bending in the pylon, when some of the back stays support a deck, the deck will flex, and tensions will not vary, concentrating the tension changes into a limited number of stays which are properly anchored, as shown in fig 7. Where a. is a flexible back span, the variation in T is large compared to t, in the rigidly anchored b. the variations in T and t are similar hence the pylon experiences less bending.

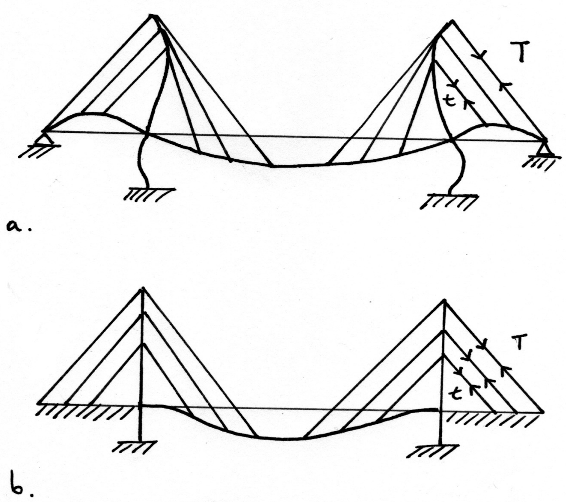

Figure 7: Comparison in Back-Span Stiffness

The final configuration for the cable stayed elements is with the pylon inclined away from the main span at an angle of 78°, held by 20 back stays spread into two planes, and 22 main stay cables in a singular plane picking up the middle of the deck. The stay cables would have been pre-tensioned to the required tension to hold the deck and pylon in their desired positions for service state loading. The resultant forces between the main stays and back stays will have analysed to achieve minimum eccentricities, to eliminate bending in the pylons.

In this bridge the earth anchor foundation is tied back to the base of the pylon through a linking layer of concrete beneath the carriage way, which can carry the compression required to balance the forces with the deck. This seems rather bazaar to me, as it seems like the side span deck has not been eliminated at all, it has just been buried under ground level, designed to take the necessary compressive forces as with the main deck, but experiences no bending. I would imagine a design which makes more efficient use of concrete would see the pylons move closer together, introducing small side spans and reducing the main span, with intermediate piers in the side spans to keep the deck fixed.

4.2 Arch System

The two inclined arches appear parabolic in shape constructed from 2.2m diameter rolled steel sections, inclined to the point they almost meet at their peak, 34m over the mid span of the deck. They are braced together using longitudinal and tangential stiffeners, forming a K shape. The hangers are therefore inclined from the arches picking up the edges of the deck. The hangers would have to be symmetrically spaced and the tensions distributed evenly along the arch to attempt to ensure both arches were evenly loaded.

For this type of bridge the arches will provide the stiffness, mainly through the compressive capacity of the arches. The deck is often suspended from the arches by hangers, similar to how it would hang from the suspension cables on a traditional suspension bridge. This allows the deck to be flexible spanning continuously between hangers.

Longitudinally the deck acts as a tie beam which closes the lateral forces produced by the trusting action the arch will try to exert upon its supports. This allows the foundations to be designed primarily to support only vertical forces, which reduces their design effort, and overall cost. Due to the vertical inclination of the hangers in the transverse direction the deck will be compressed, which will be beneficial in stiffening it against wind loading, but adversely may introduce buckling issues.

The transverse bracing between the arches ties them together, preventing them from buckling sideways independently. The longitudinal bracing will prevent any change in shape due to imposed loading or oscillations of one arch in respect of the other.

In the initial sizing of the arch elements, again the load distribution will have to have been assumed as with the cable stay system. I would assume one of the arches supporting 25% of the decks dead and superimposed dead loading, and initially model the deck as a continuous beam over rigid supports where hangers are expected. The arch could be sized on its axial force, but will then require

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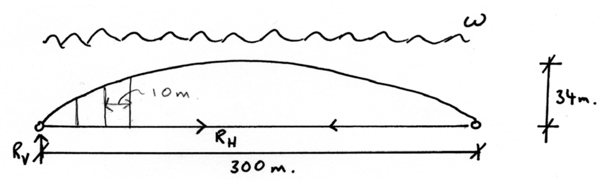
checking by loading on one half with fully factored dead and live loading, and only the un-factored dead loading upon the other half, which should give a maximum bending moment. This could then be modelled in a plane frame computer model, releasing the supports and modelling as elastic cables to give a more accurate bending moment in the deck and load distribution in the arch. The sizes from this can then be input into a 3D computer model, which models the entire deck combined with the other arch and cable stays.

The following hand calculations will assess an arch using similar assumptions to the cable stayed calculations. The hangers are at 10m spacing. By assuming the arch is parabolic under the uniform loading it should not experience any bending, I have also ignored the inclination of the hangers, but note this would only slightly increase the loading on the arch.



$$\omega = 8 \times (1.15 \times 1.1 \times 12 + 1.75 \times 1.1 \times 5) + \tfrac{1}{3} \times 2 \times 1.5 \times 1.1 \times 20.4 = 221kN/m \quad (11)$$

$$V_{max} = \frac{\omega . l}{2} = \frac{221 \times 10}{2} = 1105kN \quad (12)$$

$$M_{hog} = \frac{\omega . l^2}{12} = \frac{221 \times 10^2}{12} = 1842kNm \quad (13)$$

$$M_{sag} = \frac{\omega . l^2}{24} = \frac{221 \times 10^2}{24} = 921kNm \quad (14)$$

Finding the arch's reactions:

$$R_V = \frac{\omega . L}{2} = \frac{221 \times 300}{2} = 33150kN \quad (15)$$

$$R_H = \frac{\omega . L^2}{8 \times H} = \frac{221 \times 300^2}{8 \times 34} = 73125kN \quad (16)$$

$$Axial\ Force\ in\ Arch = \sqrt{(15)^2 + (16)^2} = 80300kN \quad (17)$$

I will now attempt to determine the area of pre-stress required to eliminate the tension experienced in the deck, I will need to a assume a simplified section (fig 8) which allows me to find an approximate I-value ($24m^4$). The resultant axial effects were found by combining the compression from the stays, and the tension from the arches (fig 9). I have also used the sagging moments (which gives greater stress despite being lower than the hogging due to the increased lever arm.) found between arch hangers for the entire width, (i.e. multiplied the value from equation (14) by 4), which is an overestimate, and have assumed the pre-stress (P) acts along the central axis.

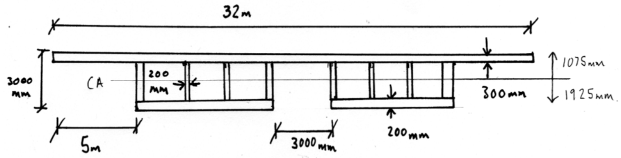


Figure 8: Simplified Section

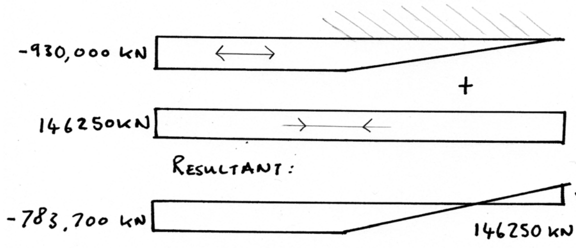


Figure 9: Resultant Axial Forces along half span

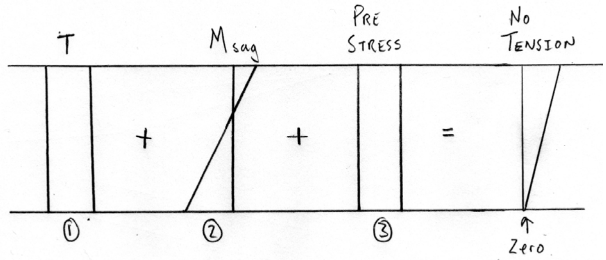


Figure 10: Stress blocks for worse tension (mid-span)

$$1.\ \sigma = \frac{T}{A} = \frac{146250000}{17400000} = 8.4N/mm^2 \quad (18)$$

$$2.\ \sigma = \frac{My}{I} = \frac{3684e6 \times 1925}{2.4e13} = 0.3N/mm^2 \quad (19)$$

$$3.\ \sigma = \frac{P}{A} = -8.7N/mm^2 \quad (20)$$

$\therefore P = 8.7 \times 17400000 =$151380kN

The area of tendons required assuming UTS is 1700N/mm², can be calculated:

$$A_{steel} = \frac{P}{\beta} = \frac{151380000}{1700} = 89000mm^2$$

The main thing I have noticed about these deck calculations is how small the stresses are, especially those due to bending (19). This maybe a case of me over sizing /over stiffening the simplified section, but I suspect it is mainly due to how crude the simplifications I have made are. In reality as I as previously mentioned all the cables and hangers will relax, therefore the supports are less rigid, and hence the sagging moments will be great deal larger than my calculations, especially at mid span, where the relaxation will be at its worst.

4.3 Combining the two systems

In some respects combining these schemes seems beneficial. Firstly it could potentially allow the deck to be lightweight and slender as each carriageway is supported along their edges, which eliminates the inefficiencies which occur from the over-stiffening required in most single plane cable stay bridges to resist torsion. The span to depth ratio achieved is about 1:100. While this proves it is a success in that this is a pretty slender ratio for a single plane of cable stays supporting such a wide deck. I would have assumed that the slenderness ratio should be somewhere between that achievable for a suspension system (often over 1:300) sized primarily on bending, and a single plane cabled stayed system sized on bending, torsion and axial forces. I would suggest the slenderness achieved is closer to the single cable stayed system, suggesting that deck slenderness was not the overall design objective.

The level of compression in the deck should be relatively low, as the tie action from the arch may relieve some of the higher compressive forces in the concrete. However depending how high this tension actually is, it could be a hindrance. As explained previously the type of cable stay system employed in this bridge is likely to induce a certain amount of tension into the deck at mid span. This tension coupled with the tie beam tension may result in the deck requiring a large amount of steel pre-stress to balance the tension.

I believe the main problems occur with the variations in stiffness between the two systems. The structure is indeterminate in both the longitudinal and lateral directions, which makes the sizing and pre-stressing of all the elements crucial to how the loading will distribute itself between the systems, as it possible to impose the required distribution, for service loading at least. Ideally I would imagine in justifying the use this combined approach the designers will have been looking to distribute the load evenly between the systems. From the information documented by the designer [1], an elastic computer analysis told them that 92% of the deck dead load was supported by the stays, leaving the arches 8%. This shows that this stage the arches are contributing very little. However the designer and contractor have assured that during the monitoring period, the cable and hanger tensions were adjusted to share the load more evenly. I believe the intention was that the cable stays take the major variable loading such as traffic, while the arches were intended simply to take a portion of the service state loading. The effect of temperature has a large impact on the load distribution, as its effect is contradictory on the respective systems.

With a rise in temperature, the cable stays will expand, this relaxation will allow the deck to drop in the centre. The hangers will also expand, but as the arches expand they will pull the hangers upwards, the net effect will mean that the relaxation at edges of the deck will either be less than at the centre, or depending on the expansion of the arch could even be elevated. The impact of this is that the relative stiffness of both systems will swap, and it is conceivable that the arches will now take the majority of the load. While this issue was I believe reduced by painting the arch in a highly reflective paint to limit its heat absorption, the issue with loads constantly redistributing with changes in temperatures seems to me to be bad design, as it means that both structural systems have to be over engineered, as well as the fatigue issues which may result from swapping load distributions.

In conclusion there is not much evidence from this bridge that suggest the hybrid scheme is structurally beneficial, apart from the reasonably slender deck. The elements associated with each system, the arches, the pylons, seem to have been designed using refinements which you associate with structural efficiency, by inclining the arches for example. But overall the contradictory effects of each system, especially under the daily temperature changes have resulted in a structure which is over-engineered both in materials and design effort. This reinforces my opinion that this bridge was built more as an expensive experimental sculpture than as an efficient method of spanning 300m.

4.4 Deck

The deck is a multi cell concrete box girder, which from the section's profile appears to have two main wings, which span between a central box housing the cable stay anchors, and the extremities which are propped by the inclined hangers. The box design is very common and relies on membrane action to concentrate the vertical loading to areas of stiffness. I am sure that although the overall section is of two wings, it has still been modelled as continuous (rather than two simply supported spans) across the middle support from the stay cables, as here is it still over 2m deep, and this seems rational as the model predicted the cable stays take a large portion of the loading.

The differential action in terms of bending in the deck caused from the alternating load distribution between the stays and hangers due to temperature fluctuations, has lead to the inclusion of transverse pre-stressing. Longitudinally there is pre-stressing both to aid the concretes bending moment capacity, and to cater for the tensions induced from the tie beam action.

As the deck is continuous with no expansion joints, the temperature effects on the deck, will have a similar effect as live loading, where it will camber, and therefore induce or relax tensions in the stays or hangers.

The decks profile is incredibly smooth with a flowing shape, for aerodynamic reasons. The design I would imagine is therefore dynamically stable in the likely wind conditions. The benefits of this are that wind oscillations, such as flutter which have proved an issue with lightweight decks, will be avoided, or reduced.

4.5 Foundations

The foundations of the main abutments which the pylon and arches both sit consist of a 25.5m long pile cap, with 82 1.3m diameter piles. This seems quite large, but if the structures systems mentioned before are all designed correctly should only be designed to resist vertical loading which evidently much be pretty high, as there should be very little moment, and all the horizontal forces should balance.

The back stay are anchored into earth filled concrete boxes, designed to counterweight the tensions in the stays. As mentioned before, this block is linked back to the abutment with effectively a concrete deck under the road acting as a strut. These balances the horizontal forces from the inclination of the cables so the foundation can be designed to resist primarily uplift.

5 Construction

It is worth noting that during construction of the bridge, the lake and landscaping were still in development themselves, so while the profile of the lake was there, it had no water. I will simply explain my understanding of the process outlined in a BBR report [2].

I believe the foundations were in fact installed at the same time the banks of the lake were initially filled. Due to the unrestricted access of the area below the proposed deck, the requirement to launch, suspend, or cantilever the deck from the abutments for construction was not necessary, unlike a bridge spanning a large valley, or deep water. The simplest and I presume the cheapest option was to cast both the deck and majority of the pylons in-situ.

The concrete box girder deck will have been cast in a travelling formwork on centring, starting at each abutment casting 10m increments simultaneously towards mid span. At mid span the closing portion was only cast after the arch had been constructed, this was simply to allow the concrete time to creep and shrink once it had been pre-stressed. By doing this it eliminates the need to check the deck for any forces resulting from these effects, which can be significant in a continuous deck. As each 10m section was cast, it remained necessary to prop these with a series of temporary supports.

The lower part of pylons were cast with climbing formwork, while the top section which housed all the densely spaced anchorage blocks for the cables was prefabricated in steel. This is due to the limitations of constructing such a complex 3D arrangement in-situ. By prefabricating it was possible to ensure the locations of connections for the stay cables were highly accurate, which simplifies the design process significantly as any eccentricities caused by lack of fit can cause increased bending moment in the pylon. As this block is 31m high, it was necessary to divide it into 9 pieces for assembly on site, where it could be filled with concrete and strengthened with pre-stress. As the pylons I assume are not designed to cantilever from the ground, some of the lower front and back stays will need to have been installed initially simply to stabilize the pylon.

The arches were installed in 30 ton sections starting only from one end and similar to the deck, each module propped from deck level using temporary towers, and welded together. As the working area on the propped deck was getting crowded, a specialist contractor was used to install the stays on the opposing pylon to the end the arches were starting from. The stays were initially stressed to fix the pylon and support entire deck self weight. When both teams were complete on their respective end of the bridge, they effectively just swapped sides to complete the arch and stay installation. The hangers were now installed, but not initially stressed. It was now possible to finalise casting the deck, and release its props as the stay cables were now taking its weight. A few of the hangers had to be stressed to ensure the arches would not become instable, and the arch's props could also be removed.

The cables were constructed from between fifty-three to ninety-one 15.24mm diameter PE sheathed non galvanized strands, protected by HDPE tube, and cement grouted after completion. The Strands were anchored using wedges in the anchor head which allows some adjustment by a threaded lock ring. The initial tensioning was done using 1200ton jacks from inside the back stay counterweight boxes, and another mounted on a sliding plate under the deck to stress the main stays simultaneously. The hangers consisted of four to six 36mm diameter pre-stressing bars. Stressing of the hangers could be done from within the steel arch.

Now that all these supports were removed the cables and hangers were all tensioned to distribute the loading between the stays and hangers, based on computer predictions for different load cases. Over 2 weeks the temperature changes in the concrete and steel, and the tensions in select cables were monitored to get an idea of whether the load distributions predicted during different temperatures were correct, and acceptable.

By fine tuning the tensions, the construction tolerances such as the actual concrete density, support settlements, and locked in temperature stresses could be adjusted for, and deflections experienced in the deck significantly reduced.

As I have mentioned there were practically no building constraints when building this bridge, and in my opinion there is no way that the construction process could have been improved on.

6 Future of the Bridge

I do not see the future expansion of this bridge as a design requirement or even that it is structurally possibility to widen the deck. It could be conceivable they may want to widen the lanes or add an additional lane, by moving the guard rail and narrowing the walkways, but the inclination of the hangers would limit access for taller vehicles in the outer lanes, hence I cannot reasonably consider it actually happening.

The bridge may need future repairs against accidental damage, intentional damage, or corrosion. The bridge is not in an area particularly susceptible to major intentional damage such as terrorism, but smaller acts of vandalism can never be dismissed. The main element requiring protection will be the cables, which is provided by housing them in a HDPE tube, which will act to guard against water damage, and protect from vandalism at the same time. The steel elements will need to have been sized for fire resistance at deck level. Vehicle impact will have been considered for the design of all the guard rails, while it may also have been necessary to check the anchors and pylons for this loading as well if it was suspected there was a possibility that the guard rails were removed, or they were not to function properly.

Acknowledgments

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