REPORT ON THE MILLAU VIADUCT

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Abstract: This paper provides a critical analysis of the world’s highest road bridge and longest multi-span cable stayed, the Millau Viaduct. Areas covered include aesthetics, loadings, design and construction.

Keywords: Cable stayed bridge, viaduct, multi-span.

1 General Introduction

The Millau Bridge or Viaduct as it should technically be known as provides the final missing link in the A75 autoroute ultimately connecting Paris to Barcelona. Prior the viaducts construction traffic would have had to descend the Tarn Valley causing a bottleneck in the town of Millau especially during the summer months of July and August.

The multi-span cable stayed bridge passes over the Tarn valley at its lowest point between two plateaus. In order to do this it had to become the tallest road bridge in the world creating the world’s tallest bridge piers standing at 242m, the structure rising to 343m at the top of the pylon. The bridge also holds the title of the world’s longest multi-span cable stayed bridge with a total length of 2460m. There is a slight gradient of 3% from North to South as well as a slight curve about a radius of 20,000m. The piers are of post tensioned reinforced concrete and the deck and pylons are of steel.

Several initial proposals were at first considered by SETRA (the French Highway Department) in linking the existing autoroutes to the north and south of the Tarn Valley. These included the idea proposed by initially by bridge designer Michel Virlogeux. This included a route that would partially descend the valley, cross the Tarn with a 700m span cable stayed bridge and then tunnel through the steeper North side of valley until and joining up with the autoroute. However this proposal was to prove too expensive and too damaging on the local environment.

In 1990 the decision was made to pass the valley at the bridges current location with a 2500m bridge. Michel Virlogeux proposed a design very similar to the final structure consisting of seven main piers with approach piers and a slightly different cable arrangement. A change in director at SETRA saw a controlled design competition take place in 1993 to ensure that the best solution was found. The only design similar to Virlogeux’s was forwarded by Foster and Partners. However it was announced that SETRA design offices would no longer continue an active role in the design so Virlogeux left the department and teamed up with Lord Foster. In 1995 the second design phase took place in which it was decided that the design proposed by Foster and Virlogeux would be used.

The decision to construct the bridge under concession agreement was made in 1998 and the competition for construction tender took place in 2000. It was announced in March 2001 that Eiffage would be concessionnaire under a new subsidiary company created for the construction - Compagnie Eiffage du Millau Viaduct. The company holds a 75 year operating concession with income from the tolls paying for the construction.

The bridge was inaugurated on 14th December 2004 and opened to traffic two days later.
2 Aesthetics

Analysis of the aesthetics of the bridge will be categorized according to the ten different areas highlighted by Fritz Leonhardt.

2.1 Fulfillment of Function

The huge concrete piers help to portray the magnitude of the construction and the huge task of building across the Tarn Valley at such height. These also help to make it clear how the bridge is supported and the importance of the piers being strong, rigid structures. It is clear when looking at the bridge which members are most important from the sizing of them. Nothing has been hidden and load path is obvious throughout the structure.

2.2 Proportions of the Bridge

Looking perpendicular to the bridge it appears that the pylons and abutments are of identical width at the deck with the abutments and pylons both splitting and tapering out to meet there. However looking almost parallel along the bridge it can be seen that this is not the case with the concrete abutments being considerably wider than the steel pylons.

In my opinion one reason for the piers tapering out is to create good proportions. The outside of the piers remain constant and all that occurs is an increase in the distance between the two pier ‘halves’ as they rise. Where the piers join the deck is well proportioned especially with the wind barrier seemingly giving increased depth to the deck.

2.3 Order within the Structure

Although the tallest bridge in the world, the bridge appears simple with good order. The repetition of the pylons across the bridge is easy on the eye as is the constant height at which piers ‘split’ regardless of their starting height. The distances between piers appear equal as does the effective spans between cables. The way in which the piers and pylons seem to flow as one when reaching the deck is good continuity.

2.4 Refinements of Design

Usually when crossing a valley it is good to keep the aspect ratio of rectangles between ground, piers and deck constant. However this hasn’t been done on Millau and with good reason. With differing spans between the piers there would have to be different numbers of cables supporting the deck between each pier which would not look good. Also the pylons may also have to differ in height if constant spacing of cable anchorage at the deck is to occur.

When looking longitudinally along the bridge in the presence of sunlight it appears that the piers are of equal thickness to the pylons. The piers have been deliberately made hexagonal in shape to produce this effect with the leading face of the hexagon reflecting the sunlight and the other sides in shadow.

2.5 Integration into the Environment

It is extremely unusual for a cable stayed bridge to span across an entire valley so integration with the environment is very important. By choosing to have a cable stayed bridge serious consideration had to be given to make the bridge look as natural as possible.

The morning mist and low lying cloud hide the concrete piers and gives the impression that the deck is delicately floating on them. This natural feature would have been noted and taken into consideration in the design to produce this effect. The apparent thickness added to the deck by the windshield helps to enhance this.

This area of aesthetics varies considerably with the direction from which the bridge is viewed. Looking perpendicular to the bridge on both clear and cloudy days the cables merge into the sky resulting in a rather elegant appearance. On a sunny day the smooth finish of the pylons cause them to sparkle and glisten and as a
result make the presence of the bridge even more noticeable.

The action to make the cables blend into the sky is however where designed integration into the environment ends. The bridge in my opinion is a stark contrast to the surrounding architecture of predominantly medieval appearance. Once the decision was made to span between plateaus rather than tunnel through the effect the bridge would have on the landscape would have been top of the agenda. The solution provided when viewing perpendicular to the bridge gives the impression of slenderness and delicacy with regards to the pier dimensions relative to the open spans between. This helps to reflect the idea that the surrounding landscape is very much untouched and itself delicate.

2.6 Surface Texture

The Millau viaduct follows the general rule whereby the piers have been given a rough finish and the deck and pylons a smooth one. The reasons for this have been touched upon in the previous integration into the environment section. The texture also helps to make it clear the materials used with the concrete left untouched and the steel all given a glossy white finish.

The rougher appearance of the concrete helps to give the piers a slightly more organic appearance as well as giving a sense of strength and rigidity to assure users.

The smooth texture of the steel deck, cables and pylons help it to seemingly float above the mist and clouds.

2.7 Colour of Components

As mentioned previously the colour plays an important role depending on the viewpoint from which the bridge is observed. When viewing perpendicular to the bridge the white of the cables blend into sky behind. The pylons, also being white in colour remain visible as the thickness means that some part is always in shadow.

For the piers it is their geometry which is the key factor determining the appearance and not the colour.

They are shaped such that different areas are in shadow depending on the viewpoint. The colour plays very little part here except for it being light such to exaggerate this effect when contrasted against dark shadow.

2.8 Character

The design itself is fairly unique where the norm is that each pylon has stays anchoring the pylons to the ground on one side and supporting the deck on the other. This is not the case with Millau with cables either side of the pylons acting to support the deck.

Despite potentially complicating design and construction, the in plane curvature (on a radius of 20km) adds character. When driving over the bridge you can see each of the pylons as you progress helping you to see exactly how the bridge works. This would not be the case had the bridge been straight.

Holding several world records and being known for this also adds character as there is no bridge higher/longer etc.

The most important feature of the bridges is its height, obviously it is this which gives the bridge most of its character. When viewed from Millau the bridge stands in the distance, imposing itself on the landscape, which acts as a constant reminder to the residents of the effort involved in alleviating the congestion problem in their town. At the same time the slender piers and pylons and sometimes invisible cables reflect sensitivity and delicacy associated with the surrounding environment.

2.9 Complexity in Variety

Complexity has very much been kept to a minimum in the Millau bridge with it obvious exactly how the bridge works. There is no structural confusion and it is clear what role each component has to play. Concrete piers support the deck at 342m intervals (204m at end spans). Additional support is provided by cables attached to steel pylons located above the concrete piers. The cables are of a semi fan arrangement whereby they are fixed at equal distance over a certain area of the pylon, neither all at the top nor equally spaced over the entire height of the pylon.

The slight curve gives some apparent complexity when crossing the bridge but is measured to perfection as none of the cables appear to cross therefore avoiding unnecessary confusion.

As mentioned the piers are of hexagonal shape. This mildly introduces a subtle complexity as from some viewpoints the piers appear to twist as the light catches the different faces together with the taper in the longitudinal direction.

2.10 Incorporation of Nature

In my opinion the bridge does have a slight organic feel about it. The slight taper of the columns is

Figure 2 – The bridge seemingly floating above the clouds.
almost tree like with the much organised cables acting as branches picking up the deck.

With the exception of the above I do not feel any other deliberate or otherwise incorporation of nature exists. There have been associations made with the cables being like a spiders cobweb but personally I don’t see this and this could be vaguely associated with any bridge with multiple cables.

3 Loadings

In 1990 the initial study for the bridge was undertaken using French standards. The final structure was also designed to French standards as specified within the contract. The temporary steel supports and steel deck were designed and checked for instability according to Eurocode 3. The loadings used in the actual design process are therefore likely to be different to those about to be considered. The loadings will be discussed according to BS 5400.

As well as the basic loads applied to all bridges the geometry and design of the bridge leads to other loads and effects which need to be considered. The constant curvature introduces horizontal centrifugal loading and the single plane of cables requires consideration to be given to torsion effects.

3.1 Dead Loads

The dead load is primarily just the steel deck. The cornice and wind screen can also be considered as dead load as removing these will seriously affect the aerodynamics of the deck so will never happen. The fixings for the cables and also the cables themselves may be considered as dead loads as well as the pylons.

3.2 Super Imposed Dead Loads

The black top surfacing (a surface developed especially for this bridge), concrete and steel crash barriers, handrails and all drainage can be considered as super imposed dead loads. These are all considered permanent but can potentially be removed. When the bridge was constructed the loads just mentioned were added after the main structure (dead load) had been completed.

3.3 Live Traffic Loads

The bridge currently has two lanes of traffic and a narrow hard shoulder in each direction. The total width of carriageway is 23.3 metres including the steel crash barriers on the outside. It is therefore appropriate to take the number of notional lanes to be six where the total carriageway width has to be between 19.0m and 22.8m.

The two types of loading, HA and HB will be placed at there most adverse locations discussed later in this section.

During the construction of the bridge the pylons were wheeled out into location on crawlers with the weight of the convoy reaching 8MN. With a crawler located at each end of the pylon this represents a load of 4MN per crawler. According to BS5400 the total loading per HB vehicle is 1.8MN spread over 4 axles each consisting of 4 wheels i.e. 112.5kN per wheel. The crawlers used had multiple axles so would have generated a greater UDL than HB loading but a lesser point load at the wheels. This condition would have been considered separately especially considering that there were no cable stays in place when the vehicle passed over.

HA and HB loading is considered to act vertically only in the form of UDL’s, Knife Edge Loads (KEL) and point loads. There are other secondary traffic loads which would have been considered for Millau. Below are two possible load combinations when trying to determine the most adverse torsional effects. The HA loading would be factored accordingly but as advantageous, the dead load would not. The possibilities show a continuous unbalanced live load acting the entire length of the structure and the other shows it alternating between spans. According to the British Standards the applied loading for a span of 9.5kN/m.

![Figure 3 – Two torsional loading possibilities](image)

The curvature of the bridge is on a radius of 20,000m so the horizontal force associated with this is 1.49kN. This is an extremely low value and would not have been considered in design.

Braking from trucks can cause horizontal loading on the deck with a force of 8kN/m being assumed as acting along one notional lane. This is together with a point load of 200kN and all associated HA and HB loading. For HB loading 25% of the nominal load is applied over 2 axles.

Accidental skidding from vehicles is considered as causing a point load of 250kN. This can act in any direction in one of the notional lanes whilst also applying the associated HA loading.
Collision with the steel parapets on the outside and concrete ones on the inside will have been considered. The effect the collision has on the substructure will have to be minimal such that only localised damage to the parapet itself occurs. With the parapets present it is extremely unlikely that vehicles will impact the pylons and the cable-deck connections. However due to the size and cost of the project this may have been considered.

The locations of each of the seven piers within the valley are isolated enough such that collision loading from road vehicles will not have to be considered.

The height of the bridge introduces another collision loading in the form of impact from aircraft. There is potential for this to occur on any part of the structure. This is something obviously not covered in the British Standards but will be of similar principle to vehicle collision with a single horizontal load being designed for. This would have been a hot topic around the time construction began in January 2002.

3.4 Wind Loading

The British standards obviously only apply to the British Isles and also to bridges spanning up to 200m. Designing to these or probably to any other standards is unlikely for a bridge of this size. The deck of the bridge relies on aerodynamics to resist the wind loads. Comprehensive wind tunnel testing was carried out to gain an understanding of the decks response to the applied wind loads.

Being located in a valley special consideration should be given to funnelling effects acting to increase the wind speed and ultimately the wind load. Increases wind speeds and in particular gusting may occur at the height the bridge is constructed to.

The standards may be of some use when considering the effects on the piers. Standard drag coefficients apply to various cross sections, for an octagon (the closest thing to a hexagon) the drag coefficient would be 1.3. These may have been used for an initial analysis before using an advanced computer model in conjunction with wind tunnel results.

The importance of wind tunnel testing is crucial as in terms of dynamics it may prove impossible to successfully model the interaction of the whole structure.

3.5 Temperature Loading

As with wind loading the British Standards are unlikely to be of much use as all maps and data apply to the British Isles only. With the deck 2460m in length temperature effects are extremely important. The design process would have into account the stresses induced with the expansion joints clogged. With the effective temperature range for the design process taken to be from -35°C to 45°C these stresses will be substantial and will considerably increase compression in the deck.

Another issue is the temperature difference between the upper and lower surface of the deck. This will introduce bending into the deck for which the effect will vary depending on the time of day.

3.6 Other Load Effects

With substantial amounts of concrete involved in the design one of the most important loads to be considered is that associated with creep of concrete. For Millau the highest bridge pier in the world was being constructed so any changes in height, particularly if uneven across the 7 different piers would lead to adverse effects as well as potentially aesthetic problems.

The construction technique used probably generated worse loading as the deck continuously spanned 171m between piers and temporary piers unsupported by any cables as it is in its final state.

Figure 3 – Bridge steel deck section
deck is likely to have experienced more adverse tension and compression than can be expected from the various load combinations during its serviceability lifetime. When looking at pictures of the bridge during its construction the undulations caused from these forces are obvious.

4 Structural Assessment

The bridge takes the form of a multi-span cable stayed bridge. Having multiple spans there are no back stays as with most cables stayed bridges to anchor the pylons to a rigid support. Instead adverse loads on one span directly interact with the next as the pylons bend to accommodate this.

Due to the height of bridge it is important that the pylons have a relatively low bending stiffness compared to the piers. If this is not the case and large bending moments may be transferred to the pier, huge bending moments would result at the base of the piers. Considering the poor bedrock of limestone containing significant cavities the piers are founded on, this would potentially cause problems.

The shapes of the pylons seem to be significant in reducing the bending moment transferred to the piers. The longitudinal A frame appears to encourage the resolution of moments into vertical forces. With the cables inducing a bending moment in the pylon, one ‘leg’ of the pylon will go into tension and the other compression. These forces can be transferred to the ground by the split piers.

The steel deck is placed into compression by the cable stays. The expectancy here would be to use a pre-stressed concrete deck due to its good compressive strength. However the chosen launching method dictated that the deck is of steel. During the launch effective spans where 171m so the ductility of steel was taken advantage of. A concrete deck may have been susceptible to cracking under its own weight which may have lead problems during its serviceability lifetime. Preventing such cracking during the launch would mean pre-stressing the deck in advance using tendons and also completely erecting the pylons and cables prior to launch, effectively pre-stressing the deck superstructure. This would prove time consuming and the steel deck was considered the more efficient option. The steel deck was seen to undulate during construction but due to its high ductility this did not result in any lasting structural problems. As a result the deck needs to be able resist any associated buckling with the anticipated compressive loads which may not have been such an issue as with a concrete deck.

As previously mentioned the single plane of cable stays introduces potential problems associated with torsion. Adverse live loading on one side of the cables and no live loading on the other side will result in torsion. Using an A frame or other similar pier design fixed to the deck itself would provide torsional restraint limiting the torsion effects to in between spans.

![Figure 4 - Steel pylon elevation and cross sections.](image1)

![Figure 5 - Post tensioned concrete piers](image2)
However the pier design used leaves cantilevered edges at the supports. It is therefore necessary to provide torsional restraint in the deck itself. This is provided in the form of triangulated cross beams spaced at 4.16m longitudinally supporting the continuous steel box section. Obviously with cables in two planes supporting the bridge, less consideration would have to be given to the torsional characteristics of the bridge. However given the location I feel that this would introduce too much complexity when viewing the bridge from any location other than square on.

When analysed, the deck would have been considered continuous over a series of fixed cable supports. Although technically not the case as the cables are elastic, this allows for a rough idea of the tension in each cable to be obtained using the worst load case. When analysed computationally an iterative process will be used to obtain the optimum values of tensions in the cables and compression and bending in the deck. All the previously mentioned factors will vary according with the location of the cable anchors on the deck. This is because the arrangement of cables is not a harp arrangement i.e. has varying inclinations so varying horizontal and vertical components. Computational analysis will be required to accurately model this effect. Bending of the deck between the cables can be roughly calculated using the standard results for a fixed beam taking into consideration the effect of axial loads induced by the cables. Depending on the location along the deck there will be different amounts of compression resulting from the tension in the cables. Hogging will act as the reduce compression in the top of the deck and increase compression towards the base; sagging will obviously be vice-versa.

The change in length of the cables and the pre-tensioning is important as extension and contraction can affect the moment in the deck. Without taking extension of the cables into consideration the moment due to uniform vertical load on the deck will generate equal hogging and sagging moments in the effective spans between cables. With this considered the so called ‘jelly effect’ will occur. As cables anchored closer to the main mid-span of the deck are likely to extend more so there will be predominantly sagging moment. The cables closer to the pylon have a greater vertical component so hogging will be more apparent here. With a semi fan/harp arrangement you would expect the cables to be spaced closer at the mid-span. This is where the cables will have a smaller vertical component due to their inclination. However with Millau this has not been done so these cables are likely to experience greater tension under live load and cause increased bending to the pylon whilst acting to hold the bridge up. Again this is an issue to do with aesthetics with the all main spans being equal; it makes sense to make the cable spacing equal to show good order. The areas of the deck closest to the pylons will experience the most compression as the cables acting over the rest of the span will incrementally increase the compression up to this point. There is a ‘window’ in the cable stays here as the increased compression effectively acts as a pre-stress improving bending stiffness.

As mentioned the bending moments will alter the compression force in the deck. Ideally the deck will be acting solely in compression with the bending being considered and how it will affect the compression across the section. A careful balance is required taking into account the various load conditions ensuring the deck remains predominantly in compression but to avoid buckling. Having a steel deck means that if adverse loading causes the deck to go into tension at some point, this is not a problem due to steels ductile properties.

Consideration would have been given to the effects at the abutments. The deck goes from being elastically supported by the cables to being rigidly supported by a concrete foundation. This potentially can introduce large hogging moments in the deck. A solution of this would be to gradually decrease the depth of any approach making it more flexible at the interchange. As mentioned the temperature effects are going to be very important considering the temperature range and the length of the deck. The most important temperature effect will be due to temperature difference at different times of day where bending of the deck will result. The effective temperature is unlikely to pose such a problem as the structure will expand/contract as one given the thermal coefficient of expansion is the same for concrete and steel. However with the cables being of different length there size will vary accordingly with most adverse affects being seen in the longer cables. They will extend/sag more in extremely hot temperatures and contract more in extremely cold temperatures.

As well as for aesthetic purposes the likely reason the piers are the shape they are is to increase flexibility longitudinally to handle expansion of the deck due to temperature.

As mentioned the temperature difference is likely to be more of a problem. This effect is likely to be greater in the morning due to the surface finish of the deck. With the road surface being black asphalt and the underside having a white finish the effect of the sun heating the deck may be exaggerated. This will cause a sagging moment as greater compression will be induced on the upper surface of the deck as it tries to extend more. The design of the bearing above the piers is very important here. The deck must be allowed to expand as necessary and excessive restraint at the piers.
can cause increased compression. A transfer of moment to the pier will occur if the connection is stiff which will then in turn have to be resisted by the foundations as previously mentioned. The likelihood is that the bearings allow a limited amount of rotation to prevent this transfer of moment.

5 Construction

Constructing the world's tallest road bridge was always going to be extremely difficult. There are traditionally two methods used for constructing cable stayed bridges, incremental launching and cantilever construction. Working at such height poses significant risks as well as the cost involved in lifting sections of the deck over 200m. The design of the bridge also deems this method inappropriate as a single pier and pylon cantilevering deck from either side would be very unstable and susceptible to wind.

The decision was therefore made to launch the deck incrementally which itself posed many risks. A launch of this size had never been undertaken before and new technologies had to be developed to slide the deck out into position.

Firstly the foundations for the piers had to be constructed. The concrete for these and the piers was produced in newly built plants close to site to minimize transportation costs. This was important as a recently developed concrete was used so anything which could potential impair the quality had to be eliminated.

The foundations for each pier consisted of four bored piles ranging in depths of 9m to 16m. The piers were then constructed on top of the pile cap. In order to satisfy aesthetic requirements these pile caps were buried and hidden from view after construction.

In order to reduce construction time emphasis was initially placed on the piers closest to the abutments so the launch of the deck could commence whilst the remaining piers were still being constructed. The formwork for the piers was a revolutionary self climbing device using hydraulics. This removed such a need for manual work where the only input required was slightly changing the alignment of the formwork after every four metre rise. This system was only used on the outside with a more traditional formwork system used to form the inside of the piers. This was lifted by crane as required and adjusted accordingly for each new section. Obviously self standing cranes could not be used so the pier itself acted as a support for the cranes as they grew in height. When the piers split 90m below the deck and continue effectively as two separate structures additional self climbing formwork was required. The same process of pouring four metre sections at a time then continued as before.

With the first piers nearing completion the temporary intermediate supports were constructed. These were of tubular steel and with the exception of the first support erected, were all telescopic also contributing to a speedy erection. Again a hydraulic system was used, once one support was complete, the machinery was moved onto the next. These supports reduced the span of the launching deck.

The steel deck was fabricated offsite by one of Eiffage’s subsidiary groups Eiffel. The deck was transported to site in sections by road. This resulted in high cost as over 2000 police escorted convoys were required to transport them. When considering that concrete plants where required onsite for the piers and foundations it seems that the sensible thing to do would have been to use a concrete deck. However various other factors are likely to have been taken into account when deciding this. Considering environmental impacts the emissions caused using either material are high. This is taking into account the general rule of thumb that for every tonne of concrete produced a tonne of carbon dioxide is released into the atmosphere. There are obvious effects associated with the long distance transportation of the steel deck. The high cost associated with the transportation of the steel deck would also somewhat correspond to the cost of pre-stressing the steel deck prior to launch. The overall time taken to construct a concrete deck is likely to be longer than for a steel deck. This is because the concrete will need to be at construction grade i.e. cured for 28 days prior to launching the deck. With a steel deck the prefabricated deck sections can be welded together relatively quickly. However time was unlikely to be an important factor when considering the launch speed of the deck with weather permitting. It is fairly unlikely that the slenderess of the deck sought by Norman Foster would have been met with the use of a concrete deck. For this reason and probably ultimately the most important fact that the leading contractor Eiffage has a world famous steel fabricating subsidiary company, Eiffel, I believe the steel deck was chosen.

Sections of the deck were launched from both plateaus and met above the river Tarn, where it was impossible to construct a temporary support.

The launching of the deck was undertaken essentially using a lift and push system which occurred all in one motion. This system was especially

![Figure 7 - The undulating steel deck prior to the erection of the steel pylons.](image-url)
Each lifting machine consisted of two hydraulically operated wedges and is shown below:

![Diagram](image.png)

**Figure 8 – Hydraulic deck launching system**

In order to prevent excessive hogging in the deck during launch a steel trussed nose was attached to the front. This also contained a hydraulic recovery system. As to be expected the cantilevered deck sagged and obviously this was at its worse as the deck approached its next support. The system acted as to hydraulically raise the front of the deck to above the support as it approached. Without this the deck would have simply impacted with each support. To help reduce this sag the first pylon on each launching deck were constructed and six out of the 11 cables attached.

As there had not been any chance to extensively test the launching device prior to the launch, problems ensued. The Teflon acting to lubricated the two wedges as they slid over each other deteriorated quicker than anticipated. This unforeseen problem halted the launch with the deck precariously hanging mid-launch. Without replacement parts, the other machines not yet in use were stripped so that launching could continue.

The slight curve of the deck didn’t really provide any additional construction problems as the approach roads were also curved on the same radius. This enabled sections to be covered where the already fabricated steel sections could be assembled ready for launch.

With such large dimensions involved in all structural members, accuracy would have been extremely important as consistently small inaccuracies can lead to huge margins of error. These may have resulted in the top of a pier being in completely wrong location or the deck missing its bearings as it is launched. To monitor the progress of the bridge some 300 reflecting prisms were attached externally so total stations could be used. A GPS receiver was also attached to the launch noses so the path of the launching deck could be monitored. This strict monitoring resulted in the deck alignment being only 2cm when the two joined above the Tarn.

With the deck complete the remaining pylons were erected. These were each transported onto the deck in a horizontal position using crawlers. Once in position they were raised and connected to the deck. The method for doing this was inspired by the Ancient Egyptian means of raising obelisks. Temporary towers were erected, themselves cable stayed. The pylons were raised about a pivot which meant that they slowly rotated to a vertical position the higher the pivot got.

These steel pylons were then connected to the deck probably by a method of high strength welding. The pylons are of adequate size for welding to be carried out inside the pylon as well as outside to create a good joint.

Whilst the pylons were being erected the cables would have started to be attached to the already erected pylons including the two in place for the launch. Several steel wires making up the cable would have passed through protective tubing and then anchored onto the prefabricated locations on the deck and pylons. The cables would have been probably been tensioned at this stage taking into consideration additional super-imposed dead loads still to be applied. These would include the 10,000 tonnes of specially developed bituminous material for the road surface.

The final stage of construction was to dismantle the temporary supports although the bridge opened with some of these still in place.

6 Susceptibility to Intentional Damage and Repair

By constructing a bridge of such size there is a chance that someone will want to intentionally damage for some reason or another. It needs to be considered how the bridge will behave should impacts result in serious damage of certain structural members.

Firstly the highest members, the pylons should in theory be the most susceptible member to impact from aircraft. The structural capabilities of the one of the pylons could be completely removed and the likelihood is that the bridge will remain standing. This of course was demonstrated during the launch. With a pylon removed the effective span of the deck would be almost the same for the launch, from the midpoint between piers to the pier itself. This is obviously governed but the way in which the pylon fails under impact. This needs to be such that the sudden increase in tension in the cables caused by an impact does not go on to cause further structural damage. This may be damage to the deck for example caused by the cable anchorages breaking free. It is likely that failure mode of the cable anchorages be designed so they fail before significantly damaging the deck. With a pylon damaged or destroyed the same procedures could be used to repair the bridge as initially used to construct it. Temporary supports would be erected to sure up the now span and the pylon can be replaced using the same method as in the construction.

Removing a pier however is likely to be a different story to removing a pylon. In my opinion at least the two spans directly connected would fail. Then as mentioned previously the extent of the damage would rely on the failure of the cable anchorages and the ease of which the unsupported deck detaches from the deck.
which could remain still standing. Considering this the easier and probably cheaper option would be to design the piers to withstand impact and even blast forces up to a certain level determined using probability. This may prove fairly expensive but so would calculating and designing all the relevant failure modes so the impact doesn’t result in a complete failure.

An impact from an aircraft on the deck is likely to be very damaging to a localised area however as with damage to a pylon, will probably remain localised with no or little effect on the rest of the structure. Due to the nature of the construction, replacing damaged sections of the deck could be difficult. As the sections were welded together onsite, the only way to remove the damaged ones will be to cut them out. A replacement section would then have to be welded in place whilst in a precariously cantilevered position. Obviously with the heights involved and associated wind speeds etc there is a significant level of danger should this need to be done. In order to fit the replacement section in, the tension in the cable stays could be jacked up providing there is adequate buckling resistance.

7. References


