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A CRITICAL ANAYLYSIS OF THE FRANJO TUDMAN BRIDGE IN DUBROVNIK, CROATIA

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Abstract: This conference paper examines the aesthetics, structure and socio-economic impact of the Franjo Tudman Bridge. Structural loading and analysis has been conducted to BS5400-2 (2006). An explanation of how the bridge was constructed is provided, and the author makes an attempt to suggest any future changes that the bridge may benefit from.

Keywords: *Dubrovnik, Asymmetrical Cable Stayed, Composite Deck, Box Girder, War of Independence*

Figure 1: Franjo Tudman Bridge, Dubrovnik

1 Introduction

The Franjo Tudman Bridge is an asymmetrical cable-stayed bridge with a total span between its abutments of 490.9m [1].

The bridge provides a vital traffic link between the coastal towns of Dubrovnik (to the east), and Split (to the west). Spanning the mouth of the Ombla Bay, known in Croatian as *Rijeka Dubrovacke*, the bridge reduces the route traffic would have to take overland in order to navigate around this bay by more than 10km. This has the obvious environmental benefits as well as encouraging the development of tourism on the Adriatic coast.

Commissioned and owned by Croatia's National Road Authority, *Hrvatske Ceste*, their tender had certain criteria that needed to be satisfied [2]:

- Piers could only be located on land, avoiding the costs associated with underwater construction.
- The bridge had to have a vertical clearance of 50m, due to navigation constraints.
- The bridge had to be at least 180m from the entrance of the port Cap Kantafig.

Several designs were submitted, including arched and suspension bridge designs, however, Zlatko Savor's cable stayed bridge was chosen. As an accomplished engineer by right, Savor has notable works, including the arched Krka Bridge near the small town of Skradin, between Split and Zagreb. Work began on the bridge in 1990, when the west approach roads and the west-bank abutments were constructed. However, construction was halted with the onset of the Croatian War of Independence, with work not starting again until 1999.

It was upon the restart of the project that the contractors, a joint venture between Walter Bau-AG and Konstruktor Split, proposed some alterations to the design (seen in figure 2), simplifying the construction procedure and reducing maintenance costs [2].

The final design for the bridge is in fact two separate structures. To the Split side, there is a post-tensioned, pre-cast concrete box girder, while the Dubrovnik side consists of a composite deck superstructure which is cable stayed.

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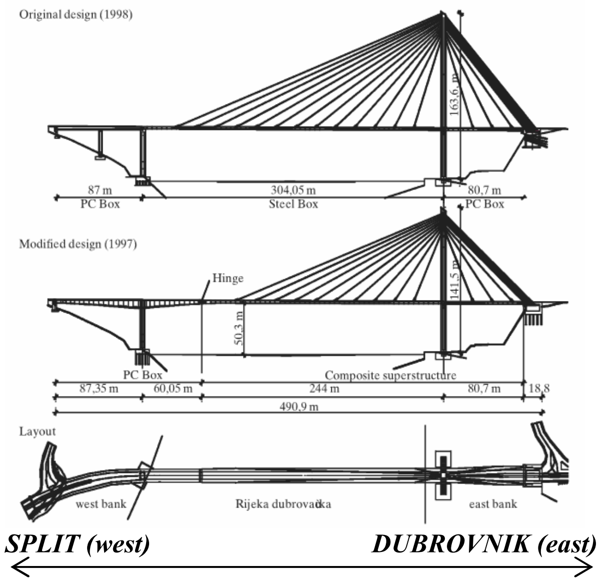

Figure 2: Old and new design of Bridge [1]

The bridge opened in May 2002, at a cost of approximately €25million [1]. While there is no method for the actual estimation of how many vehicles cross the bridge, *Hrvatske Ceste* published figures in 2009 for the approach road to the bridge showing that the Annual Average Daily Traffic (AADT) is 7,669 vehicles, while the Average Summer Daily Traffic (ASDT) peaks at approximately 13,085, demonstrating the benefit this bridge provides, particularly in peak season [3].

2 Aesthetics

While some structures in Civil Engineering fail to reveal their form, a bridge is a structure that demonstrates it for all those to see. To the general public, it is the visual aesthetics that show how a bridge functions and is the decisive factor as to whether a bridge will be accepted in a community. However, opinions vary and while not everything can be agreed on, Fritz Leonhardt's 10 rules are generally regarded as a good starting point in analysing bridge aesthetics [4].

At a glance, the bridge appears to reveal its *function* with ease. The pylon, cables and deck all clearly show their purpose and it is evident for all those to see how the structure is withstanding the loads imposed on it. However, to the trained eye, the honestly of the structure can be questioned. The bridge consists of two sections that aren't immediately noticed and one could argue that this could instigate a sense of instability and questions of safety.

Aspects of the bridge appear to be elegantly in *proportion*. The deck appears slender and the cables don't appear too thin. While the bridge may not provide an impression of balance, due to its asymmetrical nature, the void below the deck is in pleasant proportion, while fulfilling the design criteria of providing a minimum clearance of 50m. One could argue that the height of the pylon in relation to the distance the main span seems a little 'off'. But due to the asymmetry of the bridge, in my opinion, this height appears proportional and comfortably compliments the distance between the abutments.

The bridge has evident *order,* with an equal spacing of 20m between stays and clear lines and edges. A major criticism of cable stayed bridges with two or more planes of stays comes when they are viewed from an oblique angle. While this isn't noticed by those traversing the bridge, Franjo Tudman Bridge is located next to a port and the first impression tourists will get of the bridge is when they arrive by ferry. It's difficult for Savor to avoid the criss-crossing of stays, however the sheer size of the structure means, this is only noticed at particularly oblique angles.

The tapered shape of the inverted Y pier provides sophisticated *refinement* to the design. Some may say that the bridge is unrefined due to the sharp edges of the pylon, creating shadows and making it appear more slender than it really is. This could have been avoided by designing a circular sectioned pylon to reduce shadows; however, I believe this would have reduced the consistency of the bridge, removing order from the bridge with its clearly defined edges. The increase in depth of the box girder as it approaches the column shows clear refinement and style while serving a purpose of resisting the bending in the deck at the top of the column.

In my opinion, one of the makings of the Franjo Tudman Bridge is its *integration* with its environment. The asymmetry of the design fits with the context of the landscape. A typical cable stayed bridge with two pylons physically wouldn't have suited the landscape due to the tight radius of the approach road found on the Split side. The white *colour* and matt *texture* of the bridge compliments the rock and buildings around it, making it fair to say that Savor has taken inspiration form the *nature* around the bridge.

It is important to demonstrate *complexity* without delivering chaos. While the bridge seems simple in its form, complex seismic structural systems are present, as are huge dampers to reduce oscillations in the cables stays. Fortunately these complex features don't draw away from the simplicity of the overall design.

While *character* is one of Leonhardt's most subjective and difficult rules to define, in my opinion, this bridge has plenty of it, dwarfing the buildings below. The bridge has shown defiance. While construction stopped during the Croatia's war of independence, the project was never abandoned and the bridge stands today, as a symbol for a new, modern day Croatia, acting as a vital artery into the heart of Dubrovnik, bringing life to the city.

3 Structural Overview

As revealed earlier, the bridge has two structural elements, the first being the main cable stayed bridge and the second being the post-tensioned, pre-cast box girder. Both elements will be explained separately initially, but an explanation of how they integrate will be provided.

Cable stayed bridges display a direct load path from deck to pylon through stay cables, stiffening the bridge considerably [5]. Loads on the deck are transferred through bending and compression, putting the cable stays in tension. These forces are conveyed through steel anchorages embedded at the top of the pylon, compressing the pylon and transferring the loads into the foundations. Back-stays are then used to reduce the deflection of the pylon and the deck.

The composite deck is made up of 20m long steel grillage sections, with shear studs connecting a 25mm concrete slab to the steel.

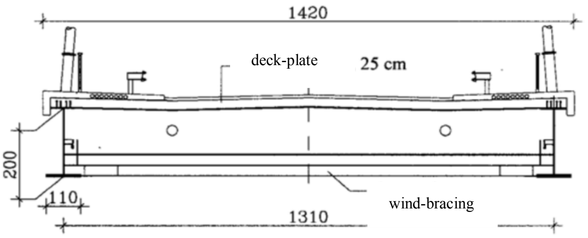

Figure 3: Composite deck section [2]

As is the case with the majority of cable stayed bridges, the further they span, the more their behavior becomes dictated by the actions of live loads. When live loads (particularly due to wind) become fairly significant compared to dead loads, vibrations in cables and the deck have to be considered. Wind tunnel tests in Aachen, Germany, tested whether it was possible to have an open composite deck cross section with two exposed, longitudinal steel supports [6]. However, the tests revealed that fluttering may occur at low speeds, resulting in the need for continuous bottom bracing in order to enlarge torsional rigidity of the deck (see figure 4).

Figure 4: Bridge deck from below

Nineteen pairs of cable stays support the bridge. They are spaced at 20m intervals (note that they have the same spacing as the composite deck, for ease of construction), apart from those closest to the pylon, which come out by 30m. Meeting at the top of the pylon, the cable splits into two planes, providing more torsional rigidity and allowing for a slimmer deck. While modified fan cables aren't as visually appealing from certain angles, this method reduces the horizontal forces exerted in the deck by the stays.

The cables transfer large forces to the top of the pylon, and consequently it is prestressed longitudinally and transversally [1]. The inverted Y pylon, along with the stays, behaves like a rigid closed section in bending, which considerably reduces possible rotation of the running surface (deck) [7].

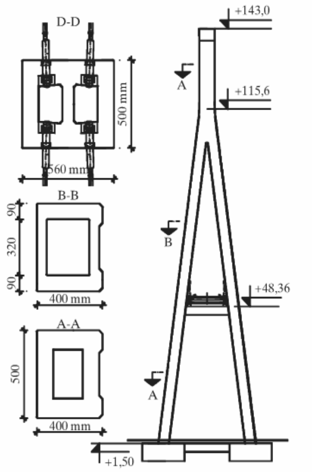

Figure 5: Pylon Details [1]

The connection between the pylon and the composite deck section isn't fixed. The deck is assumed to be continuous over the pylon all the way to the Dubrovnik side abutment. The advantage of assuming that the deck is continuous is that when considering loading cases, the composite deck section to the Dubrovnik side of the pylon can resist the sagging moments of the deck on the Split side of the pylon.

The box girder section, to the Split side of the cable stayed bridge is a hollow structure made of post-tensioned concrete. Made up of 5m sections, it is curved in plan at a radius of 212.5m and the deck cantilevers on both sides of the pier. The section of the bridge varies in depth from 3m at the abutment, to a maximum of 8m at the fixed connection at the pier and back down to 3.2m at the end of the cantilever where the two structures meet.

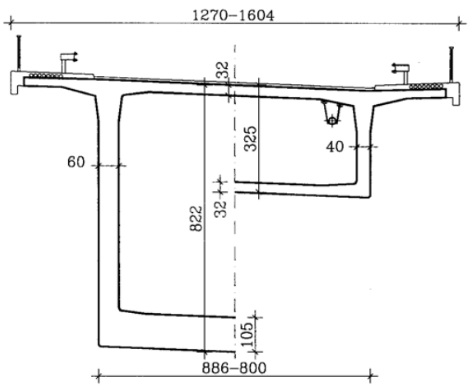

Figure 6: Box Girder Section [2]

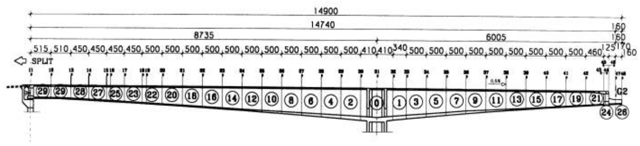

Figure 7: Box Girder Elevation [2]

Hydraulic dampers are located at the Split abutment. With a capacity of 200t, they provide a vital role in reducing earthquake actions on the bridge.

The two structures meet at a hinge. As figure 8 shows below, the box girder supports the composite deck of the cable stayed section. The composite deck sits on bearing pads, which allow the transfer of shear, but critically, don't allow the transfer of any moments.

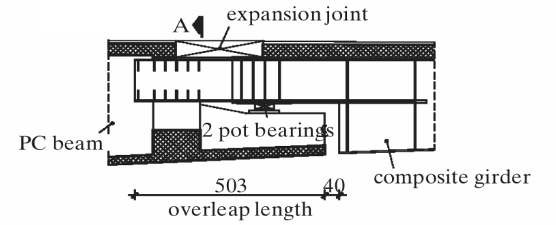

Figure 8: Connection between the two decks [1]

The fundamental reason why the composite cable stayed section sits on the box girder section is because the box girder is much stiffer than the long, flexible cable stayed section. This way, no matter how much the cable stayed deck wants to deflect, it will always be supported at the hinge by the stiff box girder section that is never likely to deflect as much as the long, cable stayed section.

4 Loading

The Franjo Tudman Bridge was designed to German DIN regulations for design [2], probably due to the involvement of the German contractors. The most up to date standards for loading can be found in the Eurocodes. However, having looked at the Eurocodes and seen how much they refer back to British Standards, for the sake of consistency, I have decided to use BS5400-2:2006 to critically analyse this bridge. Looking at Ultimate Limit State (to prevent collapse) and Serviceability Limit State (ensuring the structure is serviceable), I'll assume the inaccuracy factor for a composite bridge is γ_{f3}=1.10 (ULS) and γ_{f3}=1.0 (SLS) and the partial load factors, γ_{fl}, stated below:

Table 1: Partial Load Factors

Load	ULS/SLS
Dead Concrete	1.15/1.00
Dead Composite	1.10/1.00
Superimposed Dead	1.75/1.20
Wind	1.40/1.00
Temperature	1.30/1.00
HA alone	1.50/1.20
HA with HB or HB alone	1.30/1.10

I have decided to analyse according to load combination 1: all permanent loads plus primary live loads (vertical traffic loads).

4.1 Dead Loads

We can estimate the dead loads on the bridge from the information in the table below, where LHS refers to the box girder section and RHS refers to the cable-stayed deck:

Table 2: Materials used in bridge [2]

Material	Bridge Section	
	LHS	RHS
Concrete	1.08m³/m²	0.3m³/m²
Steel	290kg/m²	439kg/m²
Width	14.37m	14.2m

Assuming a concrete density of 2400kg/m³, one can calculate that the total dead weight for the box girder section is approximately 406.3kN/m and the cable-stayed deck has a dead weight of 161.5kN/m. consequently, the factored loads are:

Table 3: Factored Dead Loads

	ULS	SLS
Dead LHS	514kN/m	406.3kN/m
Dead RHS	195.4kN/m	161.5kN/m

4.2 Super-Imposed Dead Load

One of the main super-imposed loads on a bridge is asphalt. The average road width is 14.2m, with an average asphalt thickness of 8cm. Knowing that asphalt has a density of 2300kg/m³, we can calculate its super-imposed load:

Asphalt = (14.2*.08)*2300*9.81 = 25.6kN/m

Parapets, lighting, signposts and pavement can all be classed as super-imposed loads. If the actual value isn't know, they can be difficult to estimate as they are likely to change over time. Consequently, I have conservatively chosen a load of 10kN/m for the remaining super-imposed dead loads. This gives a total superimposed load of 35.6kN/m. Therefore:

Table 4: Factored Superimposed Loads

	ULS	SLS
Superimposed Dead	68.5kN/m	42.7kN/m

4.3 HA & KEL Load

There is no central reservation on the Franjo Tudman. The road deck is 7.7m wide, accommodating for a single carriageway in each direction with a 1.2m raised pedestrian walkway on either side. A carriageway of 7.7-m has 3 notional lanes, each approximately 2.6m wide.

For a bridge with a deck that is 50m<L<1600m, when L = 490.9m:

$$W = 36\left(\frac{1}{L}\right)^{0.1}$$

W = 19.4KN
W = 7.5kN/m (per notional lane)

Table 5: Factored HA Loads

	ULS	SLS
HA	12.4kN/m	9kN/m

In addition to this, there is a knife edge load (KEL). The KEL is taken as 120kN across one lane.

4.4 HB Load

This loading type accounts for abnormally large trucks, transporting long, wide and heavy loads. Full HB loading is considered to be 45 units, with each wheel carrying a load of 112.5kN, 450kN per vehicle axle, which equates to 1800kN in total.

Table 6: Factored HB Loads

	ULS	SLS
HB	2340kN	1980kN

As a HB vehicle has a width greater than that of the notional lane, it is considered that it straddles two lanes. A police escort ensures that there is an unloaded distance of 25m in front of and behind the vehicle. Beyond this, HA loading can be applied, where for a bridge with L > 112m & No. Notional Lanes < 6, β_1 = 1.00, β_2 = 0.67 and β_3 = 0.6.

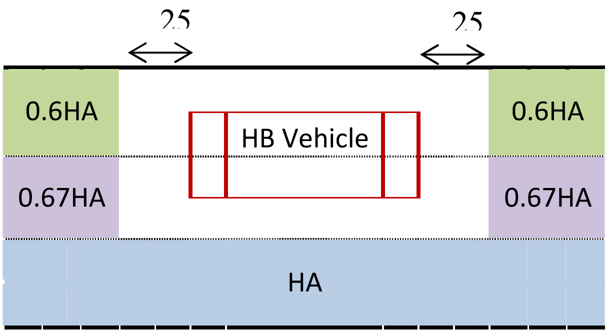

Figure 9: HB Loading Diagram, each axle as 450kN

4.5 Centrifugal Loading

The approach bridge is curved at a radius of approximately 212.5m. This introduces some centrifugal loading into the bridge, increasing torsion and bending in the deck and pier. BS 5400 states that:

$$F_c = \frac{40000}{r + 150}$$

This results in a centrifugal force of approximately 110kN, assumed to be acting in the centre of the curved span.

4.6 Pedestrian Loading

The Franjo Tudman Bridge only has a pedestrian walkway of 1.2m on either side of the bridge. Knowing that the total length of the bridge is 490.9m, this load can be calculated using the following equation:

$$k = \frac{HA\ UDL \times 10}{L + 270}$$

k = (19.4*10)/(490.9+270) = 0.255

For loaded lengths in excess of 36m, we know that:

$$W = k \times \frac{5kN}{m^2} = \frac{1.28kN}{m^2}$$

W = 1.28 *1.2 = 1.54kN/m

Part of the reason why such a high load of 5kN/m² is used is because Dubrovnik is such a popular tourist destination, it is not unlikely that the bridge may find that its pedestrian paths become full, with spectators watching fireworks from the bridge during the town's numerous summer festivals.

4.7 Wind Loading

Due to the length of the Franjo Tudman Bridge and its relatively flexible deck one has to assume that wind loading is dynamic, as the bridge will be prone to oscillations due to the force of the wind. There are three types of wind loading to be considered, which are all dependent on the maximum wind gust speed:

$$Vd = SgVs$$

Where the guest factor is equal to:

$$Sg = Sb'KfTgSh'$$

Table 7: Wind Factors

Factor	Description	Value
S_b'	Bridge & Terrain Factor	1.75
K_f	Fetch Correction Factor	1.00
T_g	Town Reduction Factor	1.00
S_h'	Topography Factor	1.00

S_g = 1.75*1.00*1.00*1.00 = 1.75

Where the site hourly mean wind speed is equal to:

$$Vs = VbSpSaSd$$

Table 8: Hourly mean wind speed variables

Factor	Description	Value
V_b	Basic Hourly Mean Wind Speed	30m/s
S_p	Probability Factor	1.05
S_a	Altitude Factor	1.05

S_d	Direction Factor	1.00

V_s = 30*1.05*1.05*1.00 = 33.075m/s

V_d = 1.75*33.075 = 57.9m/s

4.7.1 Nominal transverse Wind Load (horizontal)

This is a load that is taken to act on the centroid of a bridge deck in the transverse direction. Consequently, the wind force is summed across the whole span of the bridge where:

$$Pt = qA^1Cd$$

Where the dynamic pressure head is:

$$q = 0.613Vd^2 = 0.613 \times 57.9^2 = \frac{2055N}{mm^2}$$

A_1 = solid area of bridge = 1227.3m^2
C_D = drag coefficient = 1.8
P_t = 2055*1227.3*1.8 = 4.54MN
= 4.54MN/490.9m = 9.2kN/m

4.7.2 Nominal Vertical Wind Load

As wind passes over a bridge deck, it can either increase the load on the bridge (due to wind pressure), or relieve the load, by means of suction and uplift. This loading can be calculated using the following equation:

$$Pv = qA_3C_L$$

A_3 = Plan Deck Area = 14.3*490.9 = 7020m²
C_L = Lift coefficient = 0.5355

P_v = 2055*7020*0.5355 = 772576N
= 772576N/490.9m = ±15.7kN/m

4.7.3 Longitudinal Wind Load

$$= P_{LS} + P_{LL} = \frac{6.94kN}{m}$$

Where:
$P_{LS} = 0.25qA_1C_D$
$P_{LL} = 0.5qA_1C_D$

4.7.4 Wind Loading Combinations

The three wind load equations can be combined to provide the worst case scenario under wind loading:

Table 9: Materials used in bridge

Scenario	Load
a) P_t	9.2kN/m
b) $P_t \pm P_v$	24.9kN/m
c) P_L	6.94kN/m
d) $0.5P_t + P_L \pm 0.5P_v$	19.4kN/m

4.8 Temperature

Daily and seasonal fluctuations in shaded air temperature and solar radiation can influence the way construction materials behave. Expansion joints are used in all bridges, however, should they fail, large stresses will be induced within the material that need to be taken into account. Expansion can cause significant problems in cable-stayed bridges. Even a slight extension in a cable stay can dramatically alter the moment distribution in a deck, so it must be accounted for.

I am going to make the assumption that the average temperature at which the bridge was built was approximately 15°C. Therefore I'm going to assume a temperature variation of ±30°C, allowing for a peak summertime temperature of 45°C and a winter low of -15°C. Looking at the cable stayed structure, which has a length of 324.7m and knowing that the coefficient of thermal expansion is for steel and concrete is 12x10^{-6}, we can calculate thermal expansion:

$$\delta = \alpha L \Delta T = 117mm$$

The expansion easily accomodated by the expansion joints, but should they fail, this allows us to calculate the strain in the bridge:

$$\varepsilon = \frac{\Delta L}{L} = 3.6 \times 10^{-4} = 360\mu\varepsilon$$

Assuming that E_{steel} = 205,000N/mm^2 and $E_{cocncrete}$ = 30,000N/mm^2, one can calculate the additional stress in the deck due to the effective temperature:

$$\sigma = E\varepsilon$$

σ_{steel} = 73.87N/mm^2
$\sigma_{concrete}$ = 10.8N/mm^2

5 Strength

5.1 Deck

I have chosen to look at the cable stayed portion of the deck. Analysing the deck as a continuous beam, it is possible to calculate the bending the deck will experience, allowing one to identify whether the compressive forces due to the bending will cause the deck to buckle. The bending moment diagram takes the form of a 'jelly-mould', with sagging between the cable stays.

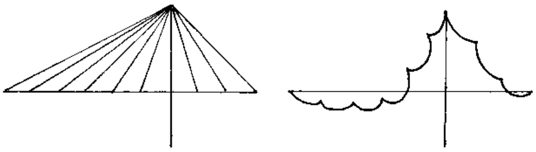

Figure 10: 'Jelly-mould' bending moment diagram

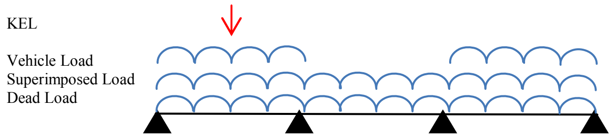

Figure 11: Maximum sagging loading arrangement between stays

Looking at loading combination one, we are left with the load case seen above, which gives us the maximum sagging between stays:

$$Mmax = \frac{Wl^2}{12} + \frac{Pl}{8}$$

Mmax = (278*20²/12) + (2340*20/8) = 15117kNm

$$\sigma = \frac{My}{I}$$

$\sigma = (15117*10^6*2000)/4.076*10^{11} = 74N/mm^2$

One must now check to see that the compressive force that the deck experiences is less than the compressive force it will buckle at, where buckling is:

$$P_E = \frac{\pi^2 EI}{l^2}$$

$P_E = (\pi^2*205000*4.076*10^{11})/20000^2 = 2062MN$

Knowing that the cross sectional area of the deck is 646400mm², buckling occurs at 3189N/mm², which is greater than the buckling experienced in the deck due to bending. In reality, this buckling stress is huge, and it is likely that calculations are incorrect.

The method I have chosen doesn't truly represent the case of the Franjo Tudman Bridge. The bending in the deck is in fact far more complicated than modelled. Cables stretch when a tensile load is applied to them. Some of the stays on the bridge are greater than 200m in length. Due to the large loads experienced by the bridge, and the sheer length of some of the cable stays, should a 200m cable stretch by even 0.5%, an extension of 1m will occur. This extension will completely redistribute the moments within the deck, effectively causing the support to move down, decreasing hogging moments, but increasing sagging moments. Consequently, advanced computer software analysis will have to be done in order to obtain more reliable and accurate results.

5.2 Force in Cables

19 pairs of DYNA Bond® cables are in place on the bridge. 7 individual cables of 15.7mm make up a strand, with each stay grouping 27 or 61 strands, creating a total cable area of 6704mm² [8].

Figure 12: DYNA Bond® cables [8]

If we assume that each stay pair carries a 20m deck section, one can calculate the force in each stay:

Force = 278*20 = 5560kN = 2780kN/stay

Assuming that each cable has a strength of 1700N/mm² and that the stays are loaded to 50% of their ultimate limit state, we can calculate the theoretical area of cable required:

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$$A = \frac{F}{\sigma}$$

$A = 2780*10^3/(1700*0.5) = 3270.6mm^2$

This shows that according to my calculations, the actual area of cable is comfortably greater than the theoretically required area in order to bear the tension due to the weight and loading on the bridge deck.

5.3 Deck Compression due to stays

Cables induce compressive forces in the bridge deck. These forces can be summed and compared to that of the buckling force in order to prevent compressive buckling in the deck. With cables varying from angles of 22-63°, the force in each cable is resolved to work out what the horizontal compressive component is. Summing these components, one can calculate that the stays induce a compressive force of approximately 44.4MN in the bridge deck, which is still below the theoretical force at which the deck will buckle, at 2062MN.

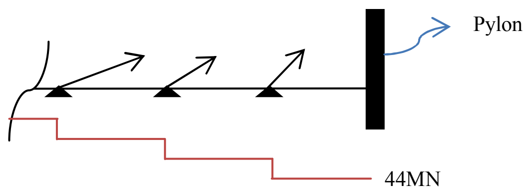

Figure 13: Compression in deck

5.4 Pylon

Large compressive forces are present in the pylon. While it’s difficult to analyse the stresses in the pylon due to its varying cross section, it’s easy to see that an efficient pylon is in equilibrium, with the backstay forces balancing the force due to the main span. This reduces the moments at the base of the pylon. Further wind and seismic analysis should be conducted to gain further understanding of the forces in the pylon.

6 Serviceability

6.1 Cable Oscillations

In 2006, a review was held over the behavior and performance of the cable stays. A natural phenomenon was observed, where during periods of high winds and snow, the cables vibrated violently. Photos showed an accumulation of snow on the windward side of the stays, changing their aerodynamic characteristics, causing considerable damage to the bridge as shown below [8].

Figure 14: Cable damage [9]

As a result of tests and further wind tunnel analysis, adaptive dampers were installed, significantly reducing the oscillation amplitudes, increasing both the service life of the stay cables and improving traffic safety [11].

6.2 Deflections

Designing the bridge to Service Limit State, we can check that the bridge is behaving not only in a structurally acceptable manner, but also one that is acceptable for users. This can be done by looking at deflection between cable stays:

$$\delta = \frac{5Wl^4}{384EI} + \frac{Pl^3}{48EI} = 11.6mm$$

Over a span of 20m between stays, this is an acceptable deflection and shouldn’t be noticed by bridge users. In reality, this value is likely to be different due to the added stiffness associated with the asphalt road surface.

7 Construction

Construction of the bridge restarted in April 1999. Work started on the foundations, with particular care being taken to stabilising the slopes where the abutments were to be constructed. Amendments were also made to the abutments due to the change in design after the war of independence.

The Split side pier was constructed using climbing formwork. Once the pier was complete, the bridge deck was erected using the balanced cantilevering construction method. Two construction cars were used simultaneously to piece the bridge deck together in 5m sections, maintaining balance over the pier. The depth of the deck section had to be greatly increased to a depth of 8m to withstand the enormous hogging moments at the top of the pier. Precast sections were lifted in by crane and fixed into position using placing equipment. Two temporary supports were in place between the pier and the abutment in order to maintain balance in the pier due to the greater span to the Split side of the pier towards the abutment. This also meant that the pier could be more slender. Each section was then post-tensioned, increasing compression in the deck, holding them in position.

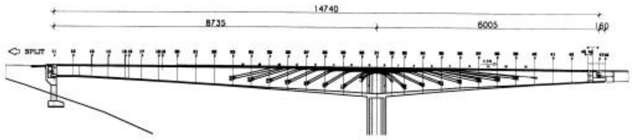

Figure 15: Deck elevation showing post-tensioning [10]

Two constructions methods were used for the construction of the cable stayed deck. Once the pylon was erected, again using traditional climbing formwork, the deck was incrementally launched from the Dubrovnik abutment. This was done because it wasn’t possible to erect temporary supports in the populated area below the bridge on the Dubrovnik side.

A total of 508.2 ton of steel deck was incrementally launched, through and beyond the pier by a distance of 33m. During launching, the deck was suspended by cabling attached at the top of the pylon to reduce the sagging in the deck.

Figure 16: Bridge construction, incremental launching [2]

The remaining, main span of the bridge was then constructed using the free cantilevering method. A Derrick crane would lift 20m, 80 ton sections of steel deck from a barge below [1]. Cables pairs were then attached to support each section. A travelling car would then lay the monolithic concrete deck segments two sections behind the crane to form the composite deck. As the cantilever length increased, assembly was compromised due to deflections in the deck due to high wind loads. Consequently, construction of the road surface was started earlier. The road was laid symmetrically, either side of the pier and improved the horizontal and torsional rigidity of the bridge and reducing the deflections due to wind. Construction was completed in March 2002.

Figure 17: Bridge construction, free cantilevering method [2]

8 Foundations & Geotechnics

The Franjo Tudman Bridge is located in an area with challenging ground and geotechnical conditions. The area experiences occasional seismic activity, considerably affecting the design. The base of the pier of the box girder section is designed to act as a plastic hinge in the event of an earthquake, dampening the actions of the structure.

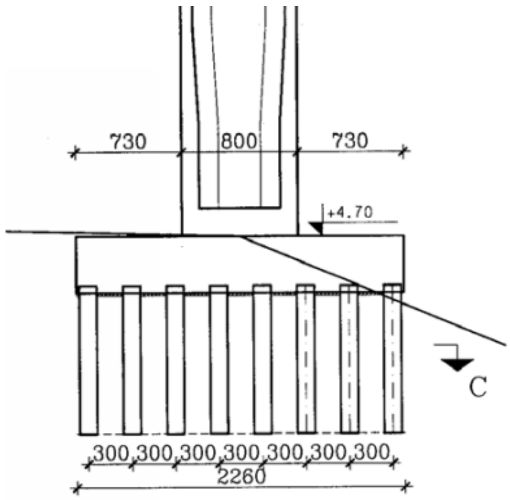

Figure 18: Box Girder Pier Foundations [2]

The east bank abutment on the Dubrovnik side of the bridge has huge uplifting forces due to the six cable stay anchors. Constructed as a huge box structure with vertically prestressed walls, the abutment is filled with gravel for extra weight. A total of 45 anchors are installed under the Dubrovnik abutment for increased slope stability [11].

The pylon legs are connected together at the base for increased stiffness. 20 geotechnical anchors restrain the foundation against differential movement due to adverse inclination of rock layers.

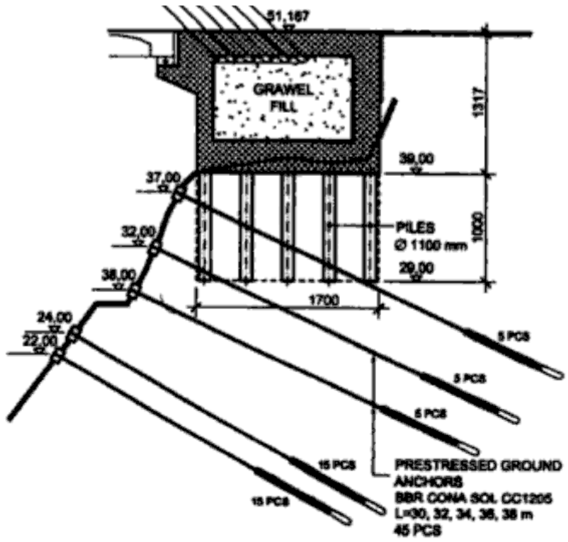

Figure 19: East bank abutment [11]

8. Durability

Creep can have a huge effect on the durability of a structure. While the composite deck, cable stayed structure is unlikely to experience much creep, the box girder will. Problems arise with creep because as the concrete cures, it contracts and there can be a loss in prestress in the tendons within the deck. As the box segments are pre-cast, one can assume that the will have been given at least 30 days to cure. However, creep is still likely to occur and expansion joints located on the bridge will accommodate for any shrinkage, ensuring stresses aren't induced in the bridge.

As stated in the serviceability section, the stays have had issues with their durability and had to be replaced. Naturally, routine maintenance should be conducted on the bridge, as well as conducting checks after incidents of extreme weather. Simple tests, including the Schmidt Hammer, Ultrasound and measurement of cover can be conducted to ensure the durability of the bridge.

9. Vandalism

Vandalism has been considered, partly due to its pedestrian access. Cable stays are protected with an outer casing. Not only does this provide permanent corrosion protection and fire resistance, but it also protects against vandalism.

8. Natural Frequency

Natural frequency is an issue in all bridges, particularly long span bridges due to the effect wind vibrations can have. The natural frequency of bridges must be between 5Hz-75Hz. Frequencies that fall out of this range can match the frequencies of wind, pedestrians and vibrations from vehicles. Should these frequencies match that of the bridge (particularly in the case of wind), displacement amplitudes can dramatically increase, with vibrations resonating through the structure, potentially causing galloping and flutter. Below I have estimated the natural frequency of the Franjo Tudman Bridge:

$$\omega_n = (\beta_n l)^2 \sqrt{\frac{EI}{ml^4}}$$

Table 10: Natural Frequency Calculations

$(\beta_n l)^2$	22.37
E (N/m^2)	$200x10^9$
I (m^4)	4076
m (kg/m)	16458
l (m)	244
ω_n (H_z)	83.6

While this value doesn't fall between 5-75Hz, it must be acknowledged that this is just a rough estimate of natural frequency and doesn't take into account the stiffness of the concrete bridge deck, cross bracing and the effect dampers have on the bridge.

9. Future Changes

Bearing in mind that this is still a young bridge, little other than the replacement of cable stays has changed since its construction. However, with an expected life of 120 years, change is likely to occur. Inevitably, there will be the need for expansion as the region develops, requiring additional lanes due to increased traffic demand. However, the two way bridge only has 3 notional lanes and therefore expansion is limited. Further widening of the bridge deck is not an option, seeing as the bridge deck passes through the pylon also unlikely because

10. Suggested Improvements

While little can be done about it now, a suggestion I would make would be to revise the hinge that is located at the middle of the two structures. Having the hinge in the middle, rather than at one of the piers makes it difficult to access and will increase maintenance costs.

Another suggestion I would make would be to widen the pedestrian walkways. While it may not be a particular route to take for pedestrians, 1.2m seems a little narrow and should a cyclist chose to take the path rather than the road, more room will be available for both users.

11. Conclusion

This paper analyses the bridge from an aesthetic, structural and socio-economical point of view. While it's difficult to prove, one can't deny that the bridge has benefited the area. Dubrovnik continues to grow as it relishes of its booming tourism industry. The bridge was designed to stimulate and accommodate the development of tourism on the Adriatic coast, however it can be argued that little foresight was given to further growth and expansion that the region is likely to experience. Nevertheless, the bridge symbolises the ambitions of the region and of Croatia as a developing nation. The bridge in itself brought jobs and economy into the region and continues to provide since its completion and established Zlatko Savor as an innovative bridge engineer.

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