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A CRITICAL ANALYSIS OF THE LEONARD P. ZAKIM BUNKER HILL BRIDGE, BOSTON, MA, USA

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Abstract: This paper provides a critical analysis of the Leonard P. Zakim Bunker Hill Bridge in Boston USA. It will consider the initial concepts, aesthetics, construction and foundations of the bridge. Furthermore analysis of bridge loading, strength, wind loading and serviceability will be covered under simplified analysis. Loading is done according to BS 5400:2 (2006).

Keywords: Zakim, Charles River Bridge, Cable Stayed, Menn

1 Introduction

The Leonard P. Zakim Bunker Hill Bridge (Fig. 1) crosses the Charles River in Boston MA, linking Charlestown and north downtown Boston. The bridge is the crowning piece to the Central Artery Tunnel (CA/T) project, also known as the 'Big Dig'. The CA/T was the largest highway construction project in the United States, and set out to relieve Boston's traffic problems and reunite the city's commercial district to its historic waterfront.

At 56.4m [1] wide it boasts being the widest bridge in the World, carrying ten lanes of traffic. Two of the ten lanes are cantilevered off the east side. The structure was the first hybrid cable-stay and the first asymmetric bridge built in the US. There are numerous unique aspects to the bridge with many resulting from the complex site constraints.

This is illustrated with the previous Charles River Bridge overlapping the Zakim Bridge through the construction period; which lead to the unusual cable arrangement of the back span cables being anchored to the median of the deck while the main span cables anchored to the sides of the deck. Further issues with the existing Orange Line and Charles River lock & dam system provided problems which are addressed in the construction section of this paper.

The bridge acquires a position of prominence and heritage in Boston. Situated in the area the Battle of Bunker Hill took place, the bridge's name reflects this as well as providing a memorial to the local civil rights leader Leonard P. Zakim.

Figure 1: Eastern view of bridge

2 Initial Ideas

16 initial designs were put forward including three arch bridges, four truss bridges, a suspension bridge and six cable stay bridges, among others [1]. Eventually a concept for a cable stay derived by Dr. Christian Menn was considered most suitable. Piers could not be located within the river due to navigation requirements for sailboats and barges; hence a main span of 227m was needed. Tied arch, suspension or truss bridges would not suit the required span or width of the main deck. An arch or truss structure would need a complex level of wind bracing for a 50m wide deck. Hence the cable stay arrangement was arrived at, to meet the challenging requirements and provide the desired dramatic structure.

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3 Aesthetics

Given the bridge's importance as the visual symbol of the CA/T and its contribution to the city's skyline it was clear that the aesthetics of the bridge were important; moreover, that a landmark structure was to be achieved.

A useful starting point to analyse the aesthetics of the bridge to is Fritz Leonhardt's book '*Bridges: Aesthetics and Design*'. In his book he points to 10 criteria which must be considered during bridge design.

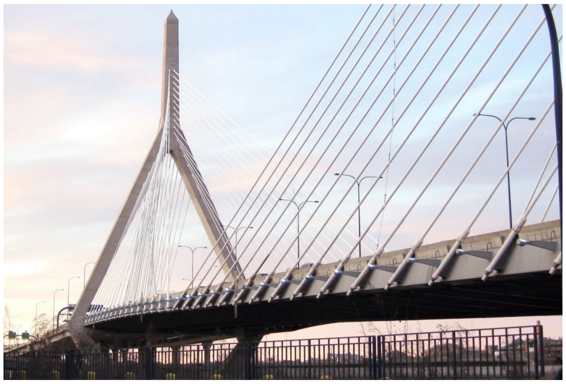

Figure 2: Western view of tower and bridge deck

Firstly fulfilment of function; the designed structure must fulfil its purpose. Furthermore it must be clear how the structure is working and "impart a feeling of confidence" [2]. It is clear the structure works as a cable stay. The towers divert outwards to cradle the deck and this gives a sense of stability. A vertical tower dissecting the deck would seem unstable given the large width of the deck. The action of a component is made more obvious by the appropriate use of materials in accordance to their inherent strengths. For example the use of concrete for the piers which primarily undergo compression and steel for cables in tension. Furthermore the piers increase in dimension at the bases which follows logic given that the moments are larger at these points. The back spans' deck depth is larger than the main span which visually accentuates the counterbalancing action of the back spans.

However, there are aspects of the bridge which seem unconventional, such as the alternating cable arrangement and cantilever deck; but have resulted from the site constraints. The arrangement of the cables makes the load path harder to grasp quickly. Furthermore the depth of the back spans is fairly constant; it would have been nicer to see the deck depth vary with varying moments. The deck was initially considered to be incrementally launched hence this could be a reason for the constant depth.

Conversely, bridges are mainly viewed in perspective from the deck. Considering this view, the arrangement of cables, particularly in the back spans, provides an interesting view for the drivers. In comparison to a twin pylon system, the level of crossing is less with cables anchored to the top of the inverted 'Y' towers. Increasing cable spacing may relieve any issues, but would cause an increase in the required deck depth and hence the proportions of the bridge.

The proportions of the towers look correct with respect to their width to height and inclined legs to pylon height. Importantly the dimensions of the towers were reduced by using steel hollow box section for the towers. It can be common with fan arrangements that the top of the towers look bulky due to the high stresses they must resist and level of cable anchorage they accommodate. However, the Zakim Bridge manages to avoid this by having a fan and harp compromise and using a unique anchorage system. The fan and harp compromise was chosen to reduce moments in the tower, caused by the difference in forces in the eastern and western cables, by allowing some cables to be anchored to the inclined tower legs. A solely harp arrangement may have looked better as there would be less crossing of cables and the voids between the cables would be of constant proportion. However, the cables would have a larger horizontal component therefore apply a greater bending moment on the towers.

The depth of the deck seems in proportion to the bridge. Due to tight navigation requirements the depth of the deck had to be limited. The voids between the steel transverse girders importantly help make the mid span deck look lighter than the back spans.

Leonhardt's third rule stresses the importance of keeping the lines of a bridge simple. "Too many directions of edges, struts, and the like create disquiet, confuse the observer, and arouse disagreeable emotions" [2]. There are no unnecessary sudden changes in depth of deck or cross-section with the Zakim Bridge. Logically, the back span cross-section of the deck is kept constant as it approaches the north viaduct and changes between the back span and mid span. The spacing of the cables and the transverse beams are the same, which reduces the direction of edges. However, an improvement would be to have the fascia run uninterrupted on the west side without the cable anchors attached to it. Moreover the ungainly lighting between the anchors could have been better integrated into the deck.

Given the historic importance of the site one key refinement was made by altering the tip of the tower to echo the Bunker Hill memorial. This small adjustment has proved very successful, especially to residents of Boston. Furthermore the towers are tapered to prevent the illusion that they are wider at this point than the base.

The choice of cable-stay fits well into this urban environment. An arch or truss bridge would seem out dated and a peculiar choice given the proposed width and spans required. Conversely, a suspension bridge would be more suited to larger spans. Careful attention was paid to limit the height of the towers to ensure they did not dominate Boston's skyline.

The use of concrete and steel are appropriate in this urban environment, given the surrounding buildings. The fascia beam on the east side is smooth which helps highlight the line of the deck. The piers are rough and untreated, which gives the towers a more sculptural and honest look.

Understated colours work effectively with the Zakim Bridge. The choice of white cables particularly gives a more elegant feel. Red is often used to highlight a component; however, if this was used for the cables, it would have been too offensive and detracted attention from the towers. The towers are left honest and the concrete makeup easy to see.

The aesthetic success of any bridge relies on how it affects people; and by incorporating a degree of patriotism by mimicking a local historic monument, there is a sense of character with the bridge.

Although keeping a design simple is important, a degree of complexity can bring a great deal of interest. Often hard to achieve, the Zakim Bridge does display elements of this with the unusual cable arrangement and hybrid deck. Hence elements which previously could be considered adverse for function and order, work favourably in providing a more interesting structure. The bridge does not attempt to incorporate any natural aspects and given its location I feel it is an appropriate decision.

3.10 Summary of Aesthetics

It must be noted that a structure does not have to meet all, or any of Leonhardt's 10 criteria to be considered beautiful. Equally we cannot automatically assume a structure will be aesthetically pleasing if all the criteria are met. The assessment of aesthetics is difficult because there is no clear rationale as aesthetics reaches emotion. Visually the Zakim Bridge is relatively clear to understand, which, given the complexity of the project it is a remarkable feat. The designers have followed a form following function approach; not sacrificing on economics or efficiency, but still managed to deliver on aesthetics. The bridge has become iconic structure and with this respect is aesthetically successful.

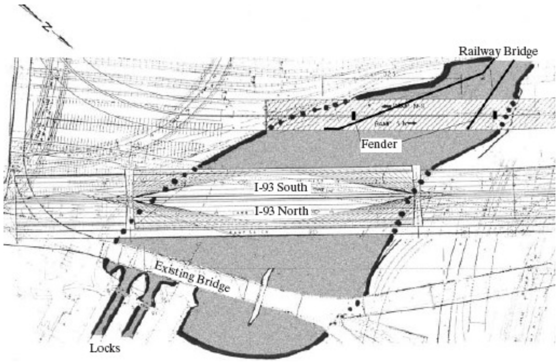

Figure 3: Site conditions [3]

5 Structural Design

The structure is truly unique. Its form is largely dictated by stringent site issues. The choice of a hybrid structure is a sensible choice with the limited space. The back spans were limited in size; hence a relatively light main span was needed to allow the back spans to effectively counterbalance the main span. Furthermore it allows the stays to be more spaced out in the back spans, which has clear aesthetic benefits. Due to the asymmetrical design of the bridge several measures were employed to reduce the moments generated in the towers and foundations. Heavyweight concrete ($4000kg/m^3$) [1] was poured into the south back spans to aid in counterbalancing the main span and reducing longitudinal moments in the towers. Secondly due to the cantilever section in the main span, there was a much higher cable tension under dead load in the eastern cables. This created a large amount of lateral bending and torsion in the tower. Furthermore due to the net transverse force from the cables applied to the deck, the deck would sway to one side during cantilever construction. One solution was to limit the dead load eccentricity by using lightweight concrete ($2000kg/m^3$) [1] in the cantilever lanes. This reduced the difference in forces of the east and west cables to about 30% [1].

Clearly detailing of the towers would be an integral part of this structure. The top of the towers contain a fabricated grade 70 high performance steel anchor box, which acted compositely with the exterior concrete using shear connectors [3]. This was an efficient way to accurately control the complex cable arrangement anchoring into the tower without external cable anchors. The steel box allowed for a more compact cable anchorage which minimised the transverse spacing and hence reduced the torsional leverarm (Fig. 5). It also eradicated the need for post-tensioning of the tower to resist the tension forces from the cables. The steel box acts as reinforcement vertically for the towers and reduces the required dimension of the towers. The dead load torsional moment from the cantilever span was eliminated by offsetting the main span cables by 79.2mm [4] from the tower centreline.

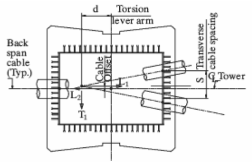

Figure 4: Offset cable anchorage to tower [4]

Anchorage of the cable stays to the main span box edge girders is controlled in a similar fashion. The cables pass through a fabricated anchor pipe which is bolted to outer webs. The fabricated component allows an efficient load transfer between the cables and the girder.

Anchoring the eastern cables in the main span between the Interstate-93 and cantilever section reduces the floorbeams' length and depth. This helped keep the steel costs down and helped meet the navigation height required under the bridge.

As previously stated, the back spans cables were anchored to the median of deck. To accommodate an existing double deck ramp, the deck was cut short and the spline beam was cantilevered out, anchoring the last three cables.

The change in direction of the tower legs creates a thrust which is taken by a beam connecting the two legs. This beam also connects the main span and back span of the bridge. Critically the connection beam has to resist torsion, biaxial bending and shear stress generated in the deck due to uneven loading in the main span and back spans. This required the beam to be post tensioned in stages up to a jacking force of 2513kN [1].

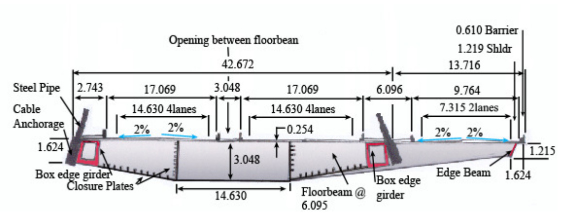

Figure 5: Main span deck cross section (all in m)

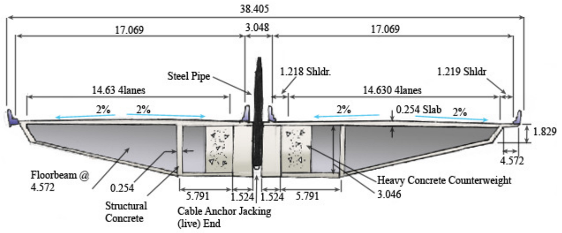

Figure 6: Back span deck cross section (all in m)

6 Loadings

The paper considers the loads on the bridge with respect to BS 5400:2 (2006), however the bridge would have been designed to AASHTO(1995). Partial load factors γ_{fl} and γ_{f3} are applied to the characteristic loads. γ_{f3} is taken as 1.10 for ULS and 1.0 for SLS (Table 1 Ref [5]). The value of γ_{fl} varies with respect to load case and material. The factor γ_{fl} is always higher for superimposed loads given the uncertainty of whether the object would be replaced with something of equal weight. The main deck has mainly been considered as it is the longest span hence is the worst case.

6.1 Dead and Superimposed Dead Loads

An estimation of each element's dead load was undertaken to assess the asymmetric design and the design of the counterbalancing back spans. Due to lack of data, the following assumptions were made. Main span floor beams have a uniform thickness of 0.5m. Box edge girders have a uniform thickness of 0.2m. Asphalt is 0.04m thick. Fill is 0.06m thick. Services are 0.2kN/m^2. The back span transverse girders are rectangular in cross section and are 0.75m thick.

Table 1: Summarised Dead Weight of Deck

Component	Factored UDL (kN/m)	Partial Factors (ULS)
Main Span Deck (without cantilever)	1393.9	Concrete γ_{fl}=1.15 Steel γ_{fl}= 1.05 Super-imposed γ_{fl} =1.75, γ_{f3}= 1.1
Cantilever	301.0	
South Back Span	2398.6	
North Back Span	1872.4	

The weight of the heavy concrete only acts in three bays of the south back span. The total unfactored weight of the heavy concrete is 8382kN. The total factored weight of the main span deck is 384895.4kN, south back span is 209142.6kN and north back span 239665kN.

The post-tension force in the main span cables can be found, assuming they are tensioned to take the dead weight of the deck. The furthest eastern cable from the north tower has been analysed. It should be noted that the span between the north and south cables is 7.62m. 63.98m is the distance from the deck to the highest cable, 24.95 is the transverse distance from the cable to the tower anchorage point and 109.7 is the longitudinal distance from the cable to the tower.

Vertical angle of cable to the deck:

$$\theta = tan^{-1}\left(\frac{63.98}{109.7}\right) = 30.25^\circ \tag{1}$$

Horizontal angle of cable to tower:

$$\theta = \tan^{-1}\left(\frac{24.95}{127}\right) = 11.12^\circ \tag{2}$$

Area western cable carries:

$$(\frac{7.62+6.096}{2})(\frac{42.672}{2}) = 146.32\text{m}^2 \tag{3}$$

Pretension force taken by eastern cable:

$$\frac{\left(\frac{(146.32(1393.9/42.67))+(301\times 6.858)}{sin\,30.25^\circ}\right)}{cos\,11.6^\circ} = 13869\text{kN}$$

The subsequent cables have a lower post tensioned force given they are at a steeper angle.

6.2 Primary Live Loading

The main deck carries 10 lanes of traffic, two of which are cantilever off the east side. The width of the cantilever is 13.7m [1]. There are two carriageways of width 17.069m [1] each and hence with accordance to BS 5400:2 (2006) 3.2.9.3.1 this gives two sets of 5 notional lanes, excluding the cantilever. Hence the notional width of an individual lane is 3.414m. The cantilever has 3 notional lanes, with a carriageway width of 9.764 m [1]. Hence the notional width of an individual lane is 3.255m.

6.2.1 HA Loading

HA loading refers to a uniformly distributed load for all normal vehicles. The load has been increased to account for overloading, impact loads and if more than one vehicle occupies the width of the lane. The total length of the bridge is 428.8m [1]; thus according to 6.2.1 Ref [6], Eq. (1) should be used. The most adverse case would be loading of the just the main span, as loading the back spans would help counterbalance the main span. Thus the length is taken as 227m which yields 20.93kN (unfactored) per metre of notional lane.

$$\mathrm{HA} = 36\left(\frac{1}{\mathrm{L}}\right)^{0.1}. \tag{1}$$

A knife edge load of 120kN (unfactored) is to be added and is to be position within the notional lane to create the most adverse effect.

6.2.2 HB Loading

HB loading represents an exceptional large vehicle load. The overall length of the vehicle can be adjusted to produce the most adverse effect. Full HB loading is 45 units; each wheel represents 2.5kN for every unit. Fig. 7 shows the most adverse sagging case with HA and HB loading. The loads alternative between stay spans. HA and KEL loading is applied to the lane with HB loading, however there must be an unloaded length of 20m either side of the vehicle. The remaining lanes are applied with 0.6 HA and 0.6 KEL as stated in Table 14 (Ref. [5]). The loaded length is taken at the total length of the main span for simplicity; however this should be the worst case from an influence diagram. The load of each axle for the HB vehicle is taken as 643.5kN. All loads were factored by γ_{fl} =1.3, γ_{f3} =1.1 for ULS.

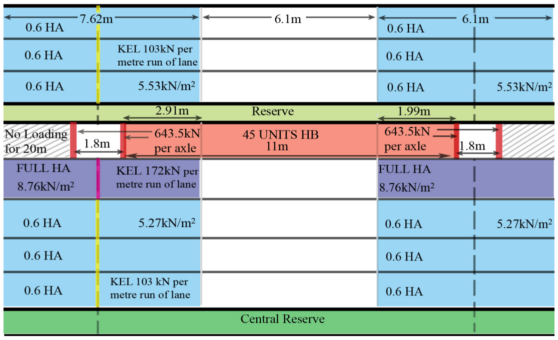

Figure 7: Maximum sagging case with HA and HB loading

6.3 Secondary Live Loading

Concrete barriers are used to protect the exposed cables at the anchorages. Concrete barrier behave in a different action to flexible steel barriers, they tend not to majorly deform but rather absorb the energy through compressive action of the vehicle's suspension system and friction between the tyre and road. Generally vehicles which collide with the barrier tend not to sustain large damage and are less likely to overturn. The AASHTO (1989) states high level barriers (PL-3) must be capable of taking a 22-tonne tractor trailer at 22.22m/s at an angle of 15º. These barriers are designed to take a transverse force of 516kN, a longitudinal force of 173kN and a vertical load of 222kN downwards [6].

7 Strength

7.1 Bending

Bending can be analysed by considering the deck as a continuous beam fixed between pylons. The stays act as elastic supports along the deck and are post tensioned such that the dead load of the deck will induce insignificant moments in the deck compared to the live load moments, hence only live loading is considered. For simplification it is assumed that the stays act as rigid supports on the deck. In reality the cables have some elasticity which causes the whole main deck to sag a certain amount. The moment longitudinally between stays is given by Eq. (2). Eq. (3) gives the stress due to bending in the tensile region of the deck and it can be seen the stress is very modest. Given the large width of the deck the maximum moment occurs transversely between the stays. The worst case live loading for bending was HA and KEL across 10 notional lanes with factors γ_{fl} =1.5 and γ_{f3} =1.1. The deck was taken to be simply supported between the stays which yielded a moment of 102.4MNm.

$$\mathrm{M} = \frac{\mathrm{wl}^2}{12} + \frac{\mathrm{Pl}}{8}. \tag{2}$$

$$\mathrm{M} = \frac{(297)(7.62)^2}{12} + \frac{(1702.8)(7.62)}{8}.$$

$$\mathrm{M} = 3059\mathrm{kNm}.$$

$$\sigma = \frac{\mathrm{My}}{\mathrm{I}} = \frac{(3059)(1.424)}{2.22} = 1962\mathrm{kNm}^{-2}. \tag{3}$$

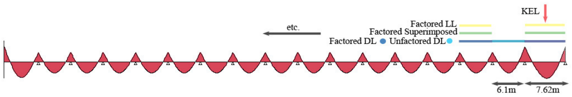

Figure 8: Bending moment of half the main span

7.2 Buckling

The compression in deck caused by cables is found using Eq. (6). This compressive force will cause buckling of the deck which it must resist. The Euler critical buckling load is found using Eq. (4). The effective length of the bridge was taken as 158.9m (0.7 x 227) P_{E1}, and 6.1m P_{E2}; the true buckling limit would lie between these results.

$$P_E = \frac{\pi^2 EI}{L_e^2}. \tag{4}$$

P_{E1}= 173.5MN < 460MN < P_{E2}= 117767MN.

7.3 Cables

An understanding of the size of the largest main span cable is outlined bellowed. The area of deck the cable carries is 240.5m^2. This is the furthest eastern cable from the north tower. Considering factored HA and HB loading, along with factored dead load, the load which the cable must carry is 8879kN. The force in the cable is 17992.5kN. Assuming the tensile strength is 1700MPa and the cable is stressed to 50% of ultimate, the area of cable required is found using Eq. (5). This gives a cable with diameter of 164mm. The total factored live and dead load force in all the main span cables was found to be 638.3MN (γ_{f3} = 1.1, γ_{fl} = 1.5) using a spreadsheet. Only HA and KEL loading was considered as this produced a worse case rather than HA and HB, as with HB loading included, 46m of the deck would be unloaded.

The tension force in each cable was found and the results were processed using a spreadsheet. The total compression in the main deck was found using Eq. (6) and the results are shown in Table 3. The compression stress was found using the furthest set of cables in the main span, shown in Eq. (7).

$$A_{cable} = \frac{17992.5\text{x}10^3}{1700/2} = 21.17\text{x}10^3\text{mm}^2. \tag{5}$$

$$C = T_{cable}\cos\theta. \tag{6}$$

$$\sigma_b = \frac{C}{A_{deck}} = \frac{24153.4\text{x}10^3}{23.67\text{x}10^6} = 1.02\text{N}/\text{mm}^2. \tag{7}$$

Table 3: Compression in Main Deck

	Factored Force (MN)	Factors ULS
Total Compression in Deck (dead)	397	See table 2 for dead load factors γ_{f3} = 1.1 γ_{fl} =1.5
Total Compression in Deck (live)	62.6	
Total Compression in Deck	459.6	

7.5 Torsion

Asymmetrical loading will induce large torsional moments in the deck. A case may occur where one carriageway is closed and the other fully loaded. The most adverse affect for the bridge would be HB loading on the cantilever section, HA loading on the cantilever and the nearest carriageway. Given it is a cantilever, 25 units of HB loading would be applied not 45. If a vehicle of 45 units was to go on the bridge it would go as close to the centre of the deck as possible and other lanes may be closed. Torsion in the main deck is transferred to the connecting beam in the tower. The transverse floor beams and concrete slabs act compositely to stiffen the deck. Torsion in the back span deck is taken through to the abutments, substructure tower and intermediate columns. The arrangement of cables provides torsional rigidity relative to a separated system.

8 Serviceability

Given the large width of the deck, deflection due to transverse bending will be much greater than due to longitudinal bending. A rough understanding of the deflection of the deck transversely is given by Eq. (8); however in reality plate analysis would be used to define the deflection. The deck is assumed to be simply supported between the stays and the deflection is considered for a 1m strip transversely. HA was taken as 5kN/m^2, KEL: 28.7kN/m and dead load: 5kN/m^2 (concrete/steel: γ_{fl} =1.0, superimposed γ_{fl} =1.2 and γ_{f3} =1.0).

$$\delta = \frac{5wl^4}{48EI}. \tag{8}$$

$$\delta = \frac{(38.7)(42.7)^4}{48(200\text{x}10^6)(2.22)} = 0.03\text{m}.$$

8 Construction

Space on site was extremely limited (Fig. 3) and hence was the primary dictation for the construction procedure. The space for the north back span was of particular concern as there was little room for falsework. For this reason the north back span was initially designed to allow for incremental launching from the north tower. Incremental launching raises many problems. Every section of the back span would have to be designed to take very high hogging and sagging moment which would be incurred during launching of the deck. After construction the tendons required for construction would be out of place relative to the in service loading. The contractor instead opted for cast-in-place back spans.

The construction of the back spans coincided with the construction of the towers. The first step was to construct the tower foundation and back span piers. Then work began on the falsework of the back spans, along with construction of the towers. This method of construction was suited to the site given the low height of the back spans, which meant less falsework was required and costs could be minimised. The soffit of the south back span is just 6m [4] above the ground level. As with the north back span, space was limited on the south back span and the deck had to be prematurely stopped to avoid disrupting an existing double deck ramp. A cantilevering spline beam was used to anchor the remaining three stays.

The main span was constructed in a suspended cantilever fashion. This form of construction is best suited for cable stay structures as the towers and cables are used to support the cantilever and reduce moments in the deck. This construction method is most cost effective with cable stay structures as unlike other bridge forms, the towers and cables are permanent. The main deck consisted of 13 segments to be erected; 12 cantilevered from the towers and one formed at mid-span between cantilevered segments. The segment erection began by installing two trapezoidal edge girders (18.3m in length) and field splice. The transverse floor beams were then attached between the girders, three per segment. This followed with the installation of a strut beam between the floor beams and then three cantilever floor beams and an edge beam. Main span and back span cables were installed simultaneously, with the first stage of stressing. Precast deck slabs were installed starting from the previous installed section. The next set of panels was put in place and the second set of cables in the unit was stressed to the first stage. The final set of precast panels was installed and then the last set of cables was stressed to the first stage. Longitudinal post tensioning was installed and transverse concrete closure strips were poured over the floor beams between panels. When the closure strip reached a strength of 24MPa the slabs were longitudinally post-tensioned [3]. Longitudinal closure strips were poured in and then all cables in the segment were stressed to the second stage. This cycle was repeated for all the cantilever segments.

Figure 9: Construction of north back span

The Freyssinet 'Iso-tension' stressing method was used to install the ungrouted stay cables. This was the first the time this method had been used in the USA. The method allowed the contractors to install one cable at a time. The initial strand is installed, stressed according to a pre-calculated force and this is used as a reference strand for subsequent strands.

When the final segment is put in place and the main deck is post tensioned, the falsework in the back spans can be removed. The deck (excluding the cantilever section) is then surfaced, ready for traffic. The next stage was to build the ramp adjacent to the back spans which joined to the cantilever part of the main deck. However, this could not start until the existing Charles River Bridge was removed.

Figure 10: Cantilever construction of main span deck

9 Foundations and Geotechnics

There was limited space for the foundations due to nearby underground infrastructure. Placing the foundation in the Charles River foundation would introduce numerous environmental and navigation issues. The foundations consist of footings on 2.44m bored piles, 14 at the south tower and 16 at the north tower, and each are designed to take 227kN [1]. The south tower foundation slab is 47x12.2x4.6m and the north tower slab is 42.1x16.8x4.6m. There are a greater number of piles on the western side of both foundations. As well as giving greater axial capacity to the foundation, this also provided the foundations with a greater resistance to uplift induced by the cantilever section and asymmetrical loading.

Near to the south tower foundation lays the Orange Line tunnel ventilation building. The closest pile is around 0.6m from the building, and so the effect of lateral forces transmitted by the works to the building had to be evaluated to ensure no damage was done to the structure. The solution was to install the closest piles within a 2.74m diameter steel isolation casing [1]. The gap between the casing and pile was left unfilled. As well as the ventilation building there was a major 0.9m diameter [1] waterline which had to be avoided.

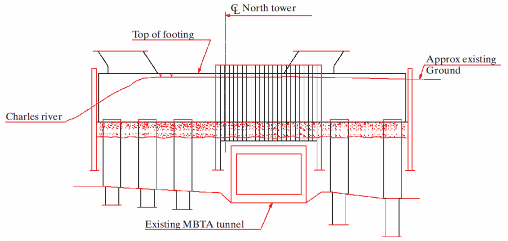

Figure 11: Orange line tunnel under north tower [4]

The north tower straddles the Orange Line Tunnel (Fig. 11). Just like the south tower steel casings were used for the piles closest to the building to minimise lateral forces. The closest piles to the Orange Line are placed outside a 1.52m buffer zone [1].

Asymmetrical loading on the bridge deck and wind loading create transverse moments, torsion in the deck and the connecting beam. The asymmetrical live load causes the forces in the cables to vary which in turn creates longitudinal bending in the tower. Both the transverse and longitudinal moments have to be resisted by the foundations along with axial loads from the pylon and other superstructure. However, the abutments and intermediate columns in the back spans do take some of the axial loads and torsion forces from the deck, thus relieving some forces in the foundations.

Foundation settlement is an important assessment especially for cable stay structures. If one of the towers was to settle this would cause the cables to lose their tension. It is likely that there is some alluvial in the ground given the proximity of the river, which increases the risk of tower settlement.

10 Temperature

Temperature effects are an important consideration in bridge loading. The two temperature effects which will be consider here are, overall temperature increase (effective temperature) and variations in temperature between the top and bottom surfaces of the deck (temperature difference). The Zakim Bridge has a hybrid structure so the stresses developed through temperature difference will be different for the different spans. Furthermore temperature variation in the cables and towers will induce bending moments in the deck, connection beam and the foundations.

A 1 in 120 year minimum and maximum temperature is taken and the deck is assumed to be fully restrained. The thermal expansion for concrete and steel is taken as $12\text{x}10^{-6}$ /ºC.

The American standard specifies that in modern climates metal bridges must be designed for a temperature range of -18ºC to 49ºC and for concrete bridges, a range of -12ºC to 27ºC. Assumed datum temperature during construction is 10ºC. Therefore the effective temperature will be taken at 39ºC.

$$\varepsilon = \alpha \Delta t. \tag{9}$$

$$\delta = \varepsilon l. \tag{10}$$

$$\sigma_c = E\varepsilon. \tag{11}$$

Table 4: Values for Eqs. (9,10,11)

ε	δ	L	σ_c	E
468με	106mm	227m	93.6N/mm^2	200x10^3 MPa

Eq. (10) shows the movement which must be accommodated by the piers if the whole deck increases by 39ºC. The apparent stress in the deck caused by the temperature increase is given by Eq. (11). Multiplying this stress by the cross sectional area of the deck (17.79m^2) gives an apparent buckling force of 1665MN. It can be seen this lies in the buckling resistance range from Eq. (4).

The second effect is a different temperature on the top of the deck to the bottom. Fig. 12 shows the temperature distribution through the deck with accordance to Fig. 9 (Ref. [5]).

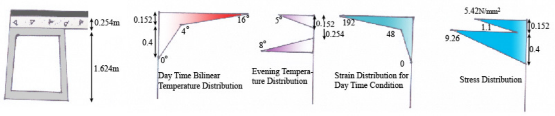

Figure 12: Temperature distribution through the deck

11 Creep

The back spans are post tensioned concrete, hence are susceptible to creep. Over a period of time there will be shortening of the concrete due to creep, especially if the concrete was no given sufficient time to strengthen before prestressing. This will shorten the prestressing tendons, leading to a loss in prestress force. By having a steel main deck the effects of creep are less substantial and have allowed a monolithic connection of the deck to the connecting beam.

12 Wind Loading and Seismic Effects

Wind loading will be carried out with accordance to BS 5400:2 (2006); the bridge would most likely be evaluated to AASHTO LRFD. Analysis through the codes will be conservative given that I am not considering the effect of surrounding structures in reducing the exposure of the structure and hence reducing the wind force on the structure. Furthermore due to the change in deck composition longitudinally across the bridge and asymmetry transversely, the bridge would definitely require wind tunnel testing. Under ASCE 7-95 the basic wind speed maps shows Boston should be analysed to 49.17m/s [7]. This value is a 3-second gust speed at 10m above ground for exposure C category and is associated with an annual probability of 0.02 [7]. The max wind gust speed, without live loads, is given by Eq. (12).

$$V_d = S_g V_s. \tag{12}$$

$$S_g = S_b T_g S_h'. \tag{13}$$

$$V_s = V_b S_p S_a S_d. \tag{14}$$

$$S_p = 1 + 0.001\Delta. \tag{15}$$

Table 5: Values for Eqs. (12,13,14,15)

Vd$_1$	Vd$_2$	Vs	Sg
82.16m/s	84.21m/s	51.35	1.64
Vb	Sp	Sa	Sd
49.17m/s	1.05	1.024	1.00

Table 5 shows both the mean wind gust speed for the whole bridge (Vd$_1$) and just the main deck (Vd$_2$).

The altitude of Charlestown was found to be 24m [8]. Conservatively S_h' and T_g are taken as 1. S_b is taken as 1.6 for the whole bridge and 1.64 for just the main span.

12.1 Normal Transverse Wind Load

$$P_t = qA_1C_d. \tag{16}$$

$$q = 0.613V_d. \tag{17}$$

Table 6: Values for Eqs. (16,17)

$P_{t\,1}$	$P_{t\,2}$	$q_{\,1}$	$Q_{\,2}$
18.9MN = 44.2kN/m	10.53MN = 46.4kN/m	4137.9kN/m^2	4347.4kN/m^2
		$A_{1\text{ total}}$	$A_{1\text{ middle}}$
		1756.9m^2	931.2m^2

The parapet is solid therefore and taken to be 0.8m high; thus d_2 = 4.10m. The value of C_d should not be taken lower than 1.3 for box girders (from 5.3 figure 5 Ref [5]). The ratio of clear span between the box girders and the d_2 is less than 7. Thus according to 5.3.3.2.2 (Ref [5]) C_d shall be taken as the number of box girders, in this case 2, times by the values derived from figure 5 (Ref [5]). Hence C_d was taken as 2.6 for the whole deck and just the main span. When considering the whole bridge deck the cantilever section is not considered to contribute to the overall width as it is not attached to the back spans. A possible 2.9% reduction of C_d could be applied however was not included because the inclination is not constant throughout the deck.

12.2 Longitudinal Wind Load

Longitudinal wind loading must be considered with and without traffic on the bridge deck, P_{LS} and P_{LL} respectively. Only the main deck was considered in this case.

$$P_{LS} = 0.25qA_1C_{d1}. \tag{18}$$

$$P_{LS} = 0.5qA_1C_{d2}. \tag{19}$$

Table 7: Values for Eqs. (18,19)

P_{LS}	C_{d1}	$A_{1\text{ middle}}$	P_{LL}	C_{d2}
0.26MN = 4.67kN/m	1.3	186.2m^2	1.031MN = 18.3KN/m	1.45

When considering the traffic on the deck, the depth of deck must be increased with respect to vehicles on the deck. This value is d_L = 2.5m (from 5.3 Fig 4. Ref [5]) and this gives a total depth of 5.81m.

12.3 Nominal Vertical Wind Load

Only the main span of the deck was consider as it is the most likely section to go in uplift.

$$P_V = qA_3C_L. \tag{20}$$

$$C_L = 0.75\left[1 - \frac{b}{20d}(1 - 0.2\alpha)\right]. \tag{21}$$

Table 8: Values for Eqs. (20,21)

P_V	C_L	A_3	Q
8.35MN = 36.7kN/m	±0.15	12800.1m^2	4347.4kN/m^2

It can be seen than the up force or down force is much less than the deadweight of the deck and hence the deck should take the force.

The worst combination for wind loading would be $P_t + P_v$ which yields 83.1kN/m. Under wind tunnel testing vortex excitation occurred at 35.6m/s and fluttering occurred at 198.6m/s which is higher than the requirement of 58.3m/s [1].

12.4 Cable Vibration

Stay-cable vibrations are common in cable stay structures due to the cables' lateral flexibility and low fundamental frequency. These cause problems in that large vibration in the cables would cause fatigue in the weld connecting the cable to the anchor pipe or even cracking of the pipe. A common case for cable stay bridges is rain-wind induced vibrations. Here a water rivulet would form on the cable and the combination with wind flow causes excitation of the cable. The Zakim Bridge attempts to mitigate rain-wind vibration by altering the surface of the cable to a 'double helix spiral bead formation' [9]. The bridge uses cross ties which work by reducing the effective length of the cables and hence increasing their frequency and uses visco-elastic dampers near the anchorage to minimise vibrations.

13 Natural Frequency

An understanding of a bridge's natural frequency is important in ensuring the bridge is not susceptible to collapse under vibration effects and vibration of the bridge does not yield unpleasant movement for its users. It is generally considered that if a bridge's natural frequency is above 5Hz it will be sufficient against vibration. It is also considered that a natural frequency over 75Hz can cause adverse physiological effects on the bridge users. An approximate value for the natural frequency of the bridge can by found from the Raleigh-Ritz method Eq. (22). The mass does not include live loading but does include superimposed dead loads. Given the bridge is a hybrid structure and the equation does not account from the use of multiple materials, only the main span will be considered. The longitudinal second moment of area of the deck is taken as 2.057m^4. The analysis considers the deck in a 'clamped – clamped' scenario, i.e. that the deck is fixed at the towers. Only the first mode of vibration was considered given this requires less energy than other modes to occur, hence is more likely to occur. One condition is to assume the stays are very rigid to the deck and hence the length of vibration is taken as 6.1m. The other condition is to assume the stays are flexible hence the length is taken as the total length of the main span; the true natural frequency would lie between these values. The transverse floor beams and the concrete slab stiffen the deck against vibration which is not considered in this equation. From the results clearly computer analysis should be undertaken to get a realistic natural frequency.

$$\omega_n = (\beta_n l)^2 \sqrt{\frac{EI}{ml^4}}. \tag{22}$$

Table 9: Values for Eq. (22)

l	ω_n	$(\beta_n l)^2$	E	m
227m	0.676Hz	22.37	$200x10^9$	169495kg/m
6.1m	936Hz			

14 Durability and Vandalism.

Vandalism should not be a major problem with the Zakim Bridge primarily because the bridge is solely a vehicle bridge. For vandalism to occur vandals would either drive onto the bridge and get out of their car or walk along the reservations. It is unlikely either case would occur. However, the tower legs are on land and the area is relatively well hidden hence making it an easy location to target.

Durability is more a key issues in particular durability of the steel cables. Maintenance of the cables has been incorporated into the design. Firstly the anchorage of the cables stays to the deck were prefabricated, without complex connections and made easily accessible. Secondly the towers have a hollow core with ladders to allow for inspection of the cables anchored to the steel box.

The cables are ungrouted and cased in HDPE pipe hence will ensure the cables stay dry. The problem with grouted cables is that when the cable vibrates, through traffic and wind loads, the grout begins to break apart. This releases water which leads to corrosion of this strands in the cable. Hence ungrouted cables avoid these problems.

With the Zakim Bridge the floor beams and the box edge girders are easily visible underneath the main deck and inspection and cleaning of these elements would not require removal of other elements.

There has been a case with cracking of the concrete which occurred on the bridge deck in a localised area. It was deduced that the cracks were most likely due to excess tensile stresses in the deck. A non-destructive sonic test found there to be voids near the cracks.

14 Future Changes

There has not been any provision for expansion of the bridge; it certainly cannot cantilever any more lanes either side of its deck. The site is already very congested and it is hard to see where the bridge can expand. The bridge is designed to take around 200,000 vehicles per day, and current traffic levels are within its capacity. However, the northbound I-93 is frequently congested and I can see the bridge operating close to capacity in the not too distance future.

15 Conclusion

The paper has provided a detail analysis of the Zakim Bridge, with accordance to BS 5400. The bridge is true example of how form following function can produce exceptional results. An appreciation of the construction challenges and demanding brief amplifies the success of the bridge. The length of the structure is relatively modest at 428.8m, vehicles travelling at 50 mph will cross the bridge in just 19 seconds; and with no room for expansion it seems a short sighted endeavour to Boston's long term traffic problems. However, the bridge has become a civic pride in Boston, but its merits extend further. The bridge is a remarkable structural engineering feat and its innovations have strengthened America's bridge technology.

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