RWDI
CONSULTING ENGINEERS & SCIENTISTS

Tel: 519.823.1311
Fax: 519.823.1316

Rowan Williams Davies & Irwin Inc.
650 Woodlawn Road West
Guelph, Ontario, Canada N1K 1B8

December 7, 2010

Ruchu Hsu
Parsons Brinckerhoff (PB)
One Penn Plaza, 250 W. 34th Street
New York, NY 10119
USA

Re: Brent Spence Bridge – stability assessment and design review
<u>RWDI Reference Number: 0940582</u>

Dear Ruchu,

We have assessed the likely aerodynamic performance of the 3 proposed alternates of the Brent Spence Bridge, which spans the Ohio River between Covington, Kentucky and Cincinnati, Ohio. This letter expresses opinions regarding the three alternates, based on our experience with wind tunnel testing and analysis of similar bridge designs.

Information on the proposed bridge layouts, with preliminary dynamic structural properties for each, was provided to RWDI on October 8, 2010. Mass properties were provided in subsequent correspondence on November 17, 2010.

Bridge Descriptions

RWDI were asked to review the aerodynamic performance of the following three alternates, all of which are double-decked with a main span of at least 1000ft:

i. Alternate 1: tied arch
ii. Alternate 3: Two tower cable-stayed
iii. Alternate 6: Single tower cable-stayed

Elevation and sectional views of each bridge are provided in Figures 1 through 3. Mass information used in the assessment is provided in Table 1. Frequencies of vibration for each alternate are provided in Tables 2a through 2c for at least the first 10 modes of vibration. Vertical and torsional modes involving significant deck motions are identified in each table.

Stability Considerations for the Completed Bridge

For the stability assessment of the deck and the towers, there are three types of wind-induced oscillations that need to be considered:

i. Flutter. A self-excited aerodynamic instability that can grow to very large amplitudes in torsion only or coupled torsion and vertical motion, that is to be avoided at all costs.
ii. Galloping. An instability involving across-wind motions similar to flutter that can theoretically grow to unlimited amplitude and is thus to be avoided.
iii. Vortex-induced oscillations. Limited amplitude vibrations caused by alternate and regular vortices shed from both sides of a bluff body, such as the decks. It occurs over limited wind speed ranges. This vibration can be tolerated if the amplitudes are not excessive.

CONSULTING ENGINEERS
& SCIENTISTS

Flutter

Flutter is an instability caused by the deflection of a structure, modifying the aerodynamics in such as way as to alter (increase) the wind loads. Typically, flutter occurs above a threshold wind speed. It is important to ensure that, should a bridge deck cross section exhibit a tendency towards flutter or divergence, the threshold wind speed be well beyond the wind speeds being considered for the ultimate strength design of the bridge.

At this stage in the design process, preliminary screening tools were applied to assess the aerodynamic stability of the bridge deck alternates. In 1961, Selberg[1] introduced simple empirical formulae for the estimation of the onset velocity of flutter. Using Selberg's formulations and the mass, modal and geometric properties of each of the decks, the critical wind speed for the onset of flutter has been estimated for each alternate. Recall from RWDI Wind Climate Analysis Report No. 0940582 that the recommended wind speed at deck height for the 10,000 year return period was equal to a 10-minute mean speed of 86.3 mph.

The flutter speeds estimated using the method of Selberg are well in excess of 86.3 mph for each of the three alternates reviewed by RWDI.

An alternate approach was used to confirm Selberg's method. Using aerodynamic derivatives measured on the Tacoma Narrows bridge deck section (which failed due to torsional flutter response caused by low torsional stiffness and a vortex shedding wind speed near the flutter velocity), torsional flutter velocities were estimated using the mass, modal and geometric properties of the Brent Spence Bridge alternates. This approach, which should yield conservative results, also suggested that the critical wind speeds for the onset of torsional flutter are well beyond the 10-minute mean speed of 86.3 mph for each of the three alternates.

Galloping

Galloping is a self-induced vibration of a flexible structure in an across-wind bending mode. Galloping has been frequently seen in iced transmission line cables, however many non-circular cross sections are prone to gallop. Galloping starts at an onset wind velocity, and normally increases rapidly with increasing wind velocity. The onset wind velocity may be approximately estimated using the Eurocode EN 1991-1-4 standard, as follows:

$$v_{CG} = 2\ Sc \div a_G\ x\ n_{1y}\ x\ b$$

where Sc is the Scruton number, a_G is a factor of galloping instability, n_{1y} is the first vertical mode frequency of vibration, and b is the across-wind deck dimension. The Scruton number is defined in the Eurocode EN 1991-1-4 as

$$Sc = 2\ x\ \delta_s\ x\ m_{i,e} \div \rho_{air} \div b^2$$

where δ_s is the logarithmic decrement structural damping, $m_{i,e}$ is the equivalen mass per unit length of deck in mode i, ρ_{air} is the air density (taken as 1.225 kg/m^3) and b is defined as above.

In the absence of measured data for a_G a value of 10 may be used, and is considered conservative. Assuming a structural damping ratio of δ_s =0.063 (1% of critical), and substituting in the mass, modal and geometric properties of each bridge alternate indicates the following:

[1] Selberg, A., Oscillations and Aerodynamic Stability of Suspension Bridges, Acta Pol. Scandina., Ci 13, 1961

i. Alternate 1 – Tied Arch: v_{CG} >> 86.3 mph
ii. Alternate 3 – Two Tower Cable-stayed: v_{CG} >> 86.3 mph
iii. Alternate 6 – Single Tower Cable-stayed: v_{CG} ~ 80 mph

Although admittedly conservative, the Eurocode approach suggests that Alternate 6, the single tower cable-stayed double-deck bridge, may be susceptible to galloping excitations at a wind speed near the once-in-10,000 year recurrence. This finding suggests that further detailed investigation of the tendency towards galloping of Alternate 6 is warranted, should this be a preferred alternate.

<u>Vortex-Induced Oscillations</u>

The phenomenon of vortex shedding occurs frequently on bluff engineering structures. Based on RWDI's experience, and research publications available in the literature, it is our view that vortex-shedding vibrations in both the vertical and torsional directions may occur for each of the alternates reviewed. However, the magnitude of the vibrations is unlikely to be severe and we are confident that appropriate aerodynamic modifications to the deck cross-sections will mitigate the vibrations.

Early model tests on open-truss suspended bridge decks undertaken for the Firth of Forth bridge indicated excellent performance with regards to vertical vibrations, i.e. fairly benign response. Depending on the aspect ratio of the truss depth and deck width, open truss bridge decks can also exhibit good torsional behaviour. However, it is known that both the torsional and vertical response of truss stiffened suspended decks is sensitive to the number and size of openings between running surfaces on the deck, and studies undertaken for the Tsing-Ma bridge indicated the placement of the openings in the deck surface was critical for mitigating vibrations. Note that this particular bridge incorporated edge fairings into its design to further enhance the wind-induced behaviour.

The magnitudes of vortex-induced vibrations are difficult to estimate precisely at this stage without wind tunnel testing. It would be prudent at this stage to consider countermeasures to mitigate vortex-induced vibrations, in the event that subsequent wind tunnel tests indicate they are necessary. These countermeasures could take the form of:

i. Edge fairings.
ii. Vents in the top and bottom deck surfaces.
iii. Open traffic barriers.
iv. Aerodynamic Damper Plates
v. Turbulence Generators

RWDI can provide sketches of the proposed solutions prior to any wind tunnel tests, to enable the design team to evaluate and rank order the solutions, to facilitate possible trials during the model studies.

Construction Stage Considerations

There are unique construction stage considerations for each alternate reviewed. A brief summary of our conclusions follows.

<u>Tied-Arch Alternate</u>

During construction of a tied-arch bridge type, depending on the selected erection scheme, the following may deserve attention with regards to aerodynamic instability:

i. the free standing arch structures
ii. the suspended double-deck cantilever before closure at mid-span (depends on erection scheme)

The free standing arches may be subject to galloping and/or vortex shedding instability, particularly before they are linked to adjacent arches. While fabrication of the arches may be undertaken off-site and the erection window can be narrow – thereby reducing the risk of an aerodynamic instability – the risk of instability remains. The following stabilizing schemes have been applied in practice:

a) install temporary tie-downs for the arches
b) install temporary link-beams to connect the arches and providing additional stiffness

During erection, should the deck be suspended from the hangers (beginning at the main-span piers and joined at mid-span), the “free” decks may have a low flutter onset speed due to the reduced stiffness and low frequency of vibration. There are erection sequences that are known to have improved performance, which RWDI and the design team will be familiar with. If these scheme are not suitable then similar measures as recommended for the free-standing arch may be applied to eleviate this problem.

<u>Cable-stayed Alternates</u>

During construction of the cable-stayed bridge alternates, there are typically two primary concerns:

i. the free standing tower; and
ii. the suspended double- deck cantilever before closure at mid-span (and/or closure at the main and back spans)

The free standing tower legs may be subject to galloping and/or vortex shedding instabilities themselves. Though some early estimates of instability may be carried out numerically, the best tool for assessing stability and verifying the wind loads and deflections during construction stages is an aero-elastic model test. If any type of instability turns out to be a problem for the towers, the following stabilizing schemes may be applied (and have been used successfully on other bridge developments):

a) install temporary tie-downs to the critical tower elevation
b) install temporary cross-beams to connect tower legs (which will require both legs to be build at the same time)
c) install temporary dampers

Considering the cantilevered double- deck during construction, the principal problem typically is not stability but load demands at the base of the towers and at the deck to tower connections. Although lower wind speeds are normally considered for design during construction, there may be a critical cantilever length where the peak loads during construction could become higher compared to the completed bridge. To reduce wind loads during construction temporary frame supports or temporary ties and/or guides are normally used by the contractors.

Serviceability Considerations

Each of the three bridge alternates have serviceability considerations which are affected by wind loading and aerodynamics. Common to all bridges are issues involving the wind-induced vibration of the stay and hanger cables.

Cables may vibrate due to:

i. Vortex shedding
ii. Rain/wind induced vibrations
iii. Wake galloping of groups of cables

iv. Galloping of cables with ice accumulations
v. Galloping of isolated cables inclined to the wind
vi. Excitations induced from the stay anchors
vii. Motions due to wind buffeting on cables

Vibrations of cables occur due to their low mass and low damping. The expected damping ratio of a stay cable or hanger cable would typically be less than 0.1%, without the use of supplementary damping or energy absorbing bushes. The excitation mechanisms noted above are considered instabilities.

It is well documented that cable-stayed bridges have experienced galloping of dry inclined cables and/or rain/wind-induced vibrations, which have led to peak vibration amplitudes as high as 5 times the diameter of the very longest stay cables. This is significant since these deflections are visible to users of the bridge, and are sufficient to cause alarm - not to mention potential damage due to fatigue of connections. Vibrations of this sort should therefore be suppressed. An effective method for controlling rain/wind induced vibrations would be through the use of helical fillets which spiral along the length of the cable. The pitch of a typical helical fillet is about 2 to 3 times the diameter of the cable. However, in colder climates these may lead to excessive ice accretion which in turn may cause galloping in its classical form. The installation of secondary cross-cables, often referred to as cross-ties or aiguilles, has also been used to suppress rain/wind vibrations. Examples of where this approach has been adopted are the William H. Harsha Bridge in Maysville, Kentucky, and the Second Severn Bridge crossing between Wales and England, to name two.

Vortex shedding is typically not a problem of stay cables, in that the critical wind speeds causing vortex shedding are low, and the magnitudes of vortex-induced vibrations are minimal. However, vortex shedding is common problem on hangers. Countermeasures such as Stockbridge dampers have been applied in such cases.

Vibration induced through the stay anchors, or parametric excitation as it is sometimes referred, occurs when the cables have similar frequencies of vibration to the decks, towers, and/or arches. Any dynamic load such as wind, vehicular or pedestrian traffic could be the origin of the vibration. Small motions of the deck, towers, or arches could result in significant cable vibrations. The most common method for suppressing motion-induced vibrations is through the use of cross-ties, which effectively detune cables' frequencies off the modal frequencies associated with the anchorage motions.

It should be noted that cross-ties are only effective for suppressing motions in the cable plane that are due to vertical deck and along the bridge tower motions. Out-of- plane cable motions are more difficult to control, and can be excited by motions of the towers or arches normal to the plane of the cables where the structure modes of vibration are close to the cable frequencies of vibration. In cases where the modal properties of the bridge tower or arch structures are not sufficiently separated from the cable frequencies, an alternative measure for vibration control could be external supplementary damping.

Conclusions

RWDI have reviewed the three bridge alternates proposed for the Brent Spence Bridge between Covington, Kentucky and Cincinnati, Ohio, and identified any potential aerodynamic instabilities which may affect the strength and safety of the bridge, and any aerodynamic issues that may affect the serviceability.

Regarding aerodynamic instability, it appears that Alternate 1 (Tied Arch) and Alternate 3 (Two-Tower Cable-stayed) will have excellent aerodynamic performance. RWDI estimates that the onset speeds for flutter and galloping are well beyond the recommended wind speed at deck height for the 10,000 year return period (equal to a 10-minute mean speed of 86.3 mph). Preliminary review suggests that Alternate 6 (Single Tower Cable-stayed) may have a galloping onset velocity which almost equal to the 10,000 year return period wind. Although RWDI's estimates are conservative, this is worth noting at this early stage.

With regards to vortex-induced vibrations of the bridge decks, the performance of the bridge decks for each alternate may be enhanced through the use of open vents in the deck surfaces, aerodynamic fairings, or open traffic barriers. Wind tunnel testing is critical to determine which of the above is most impactful.

Regarding the serviceability of these bridge decks, RWDI have identified a number of sources of wind induced cable and hanger vibrations, and suggested possible mitigations. As the designs progress and additional dynamic structural properties and information become available, we suggest that a more detailed review of the issues involving cable vibration be reviewed.

Please do not hesitate to contact us if you have any questions or comments.

Yours very truly,

ROWAN WILLIAMS DAVIES & IRWIN Inc.

John Kilpatrick, PhD, PEng
Technical Director (UK), Senior Associate

Stoyan Stoyanoff, Ph.D., P.Eng., ing.
Project Director/Principal

Table 1. Preliminary Mass Information

Alternate	Mass/Unit Length
1 – Tied Arch	10.5 kip/ft/rib (Arches)
	42.5 kip/ft/deck (Deck)
2 – Two Tower Cable-stayed	33 kip/ft/deck (Main span and Back span)
3 – Single Tower Cable-stayed	14 kip/ft/deck (Main span)
	27 kip/ft/deck (Back span)

Table 2a. Modal Frequencies of Vibration – Tied Arch

Mode	Frequency (Hz)
1	0.379293
2	0.690965
3[a]	0.692145
4	0.822813
5[a]	0.894064
6[a]	1.000086
7	1.200667
8[b]	1.320892
9[b]	1.427788
10	1.453115

a: vertical deck mode
b: torsional deck mode

Table 2b. Modal Frequencies of Vibration – Two Tower Cable-stayed

Mode	Frequency (Hz)
1	0.309716
2[a]	0.319934
3	0.375362
4	0.431133
5	0.431138
6[a]	0.433314
7	0.437484
8	0.438539
9	0.438556
10[b]	0.439909
11	0.440171
12	0.440171
13	0.44024
14	0.44024
15	0.466702
16[b]	0.737653

a: vertical deck mode
b: torsional deck mode

CONSULTING ENGINEERS
& SCIENTISTS

Table 2c. Modal Frequencies of Vibration – Single Tower Cable-stayed

Mode	Frequency (Hz)
1	0.297868
2	0.360090
3[a]	0.564058
4[a]	0.724985
5	0.901072
6	1.159462
7[b]	1.259159
8[a]	1.306169
9	1.393606
10	1.424260

a: vertical deck mode
b: torsional deck mode

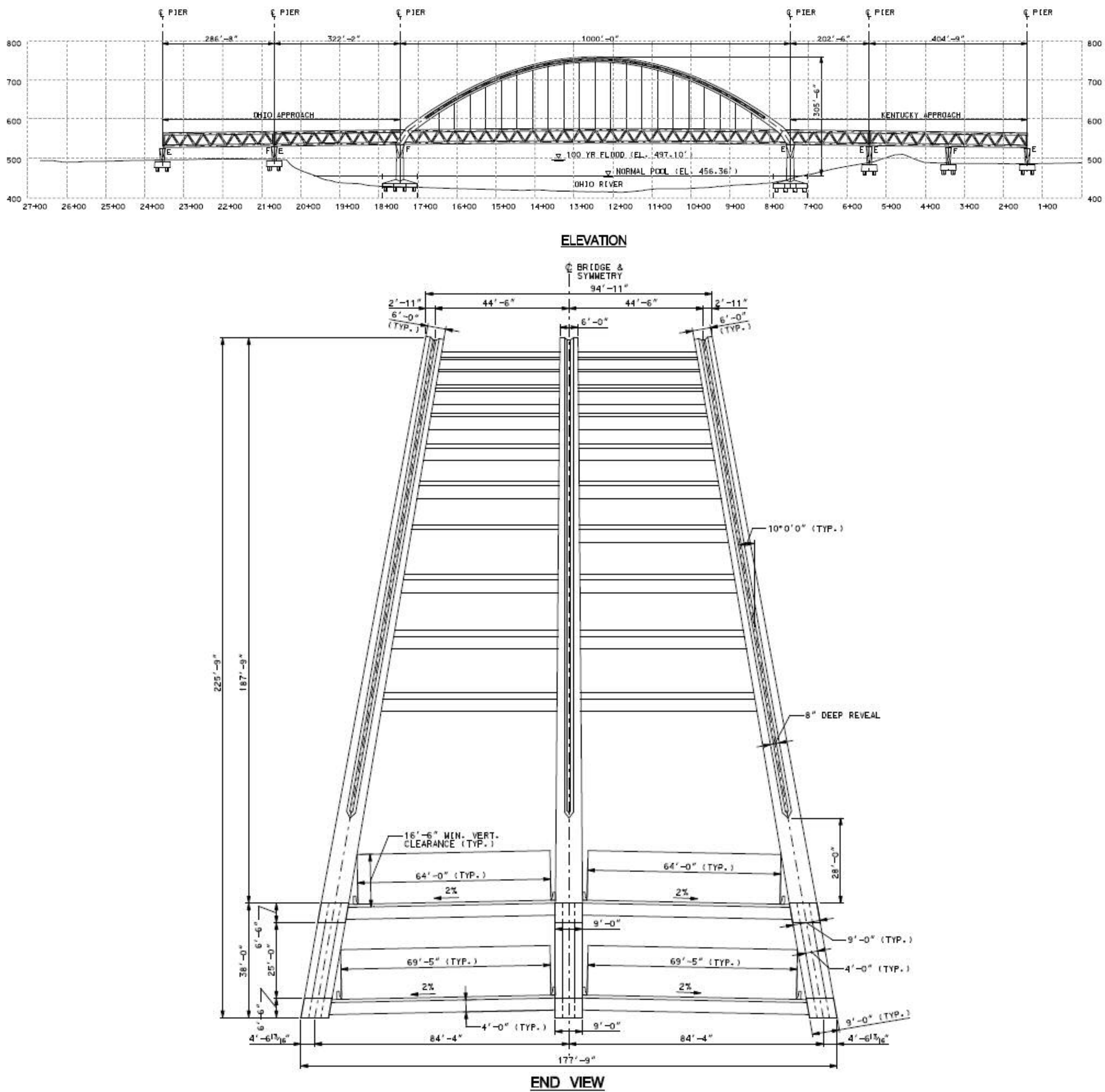

Figure 1. Alternate 1 - Tied Arch Bridge

CONSULTING ENGINEERS
& SCIENTISTS

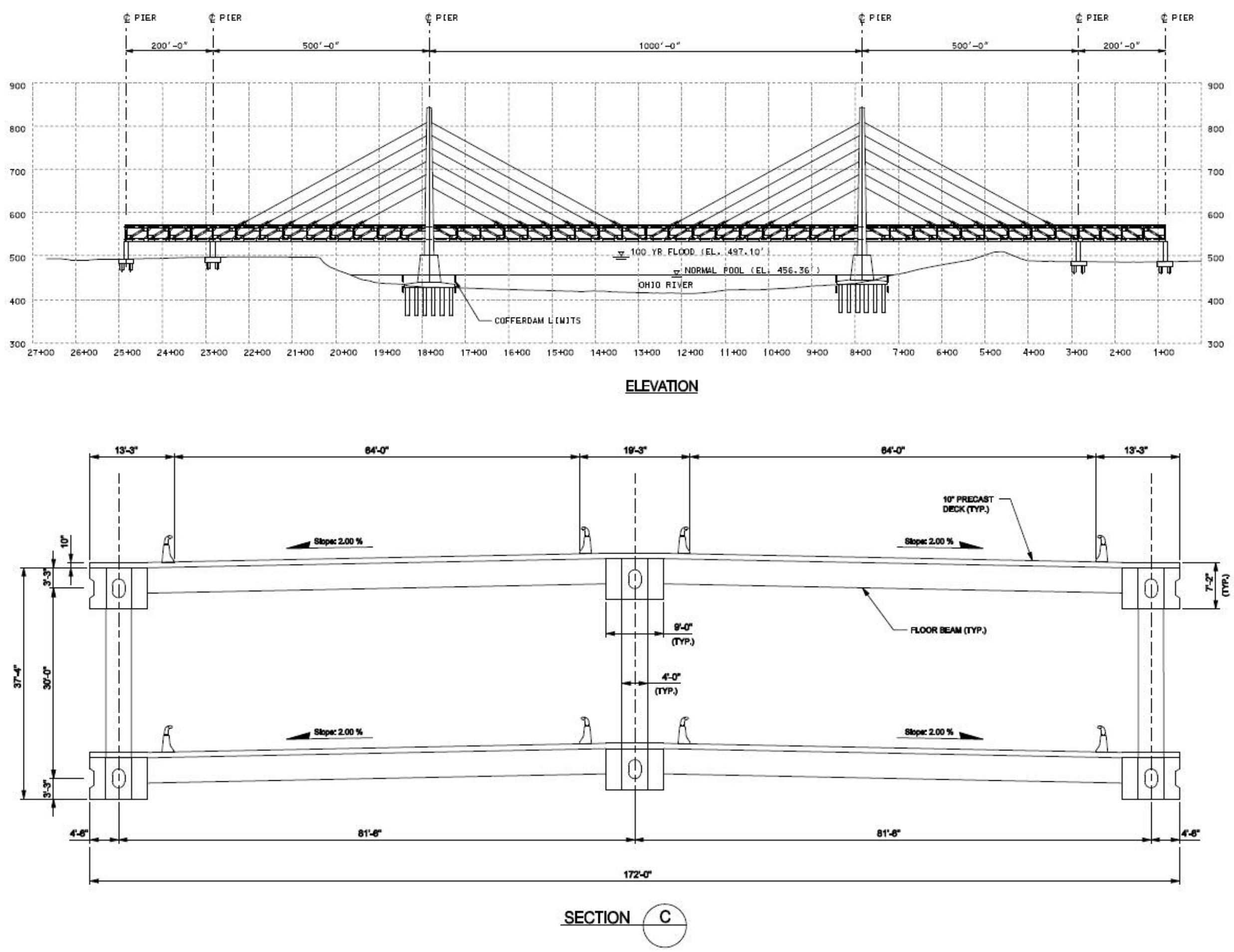

Figure 2. Alternate 3 – Two Tower Cable-stayed Bridge

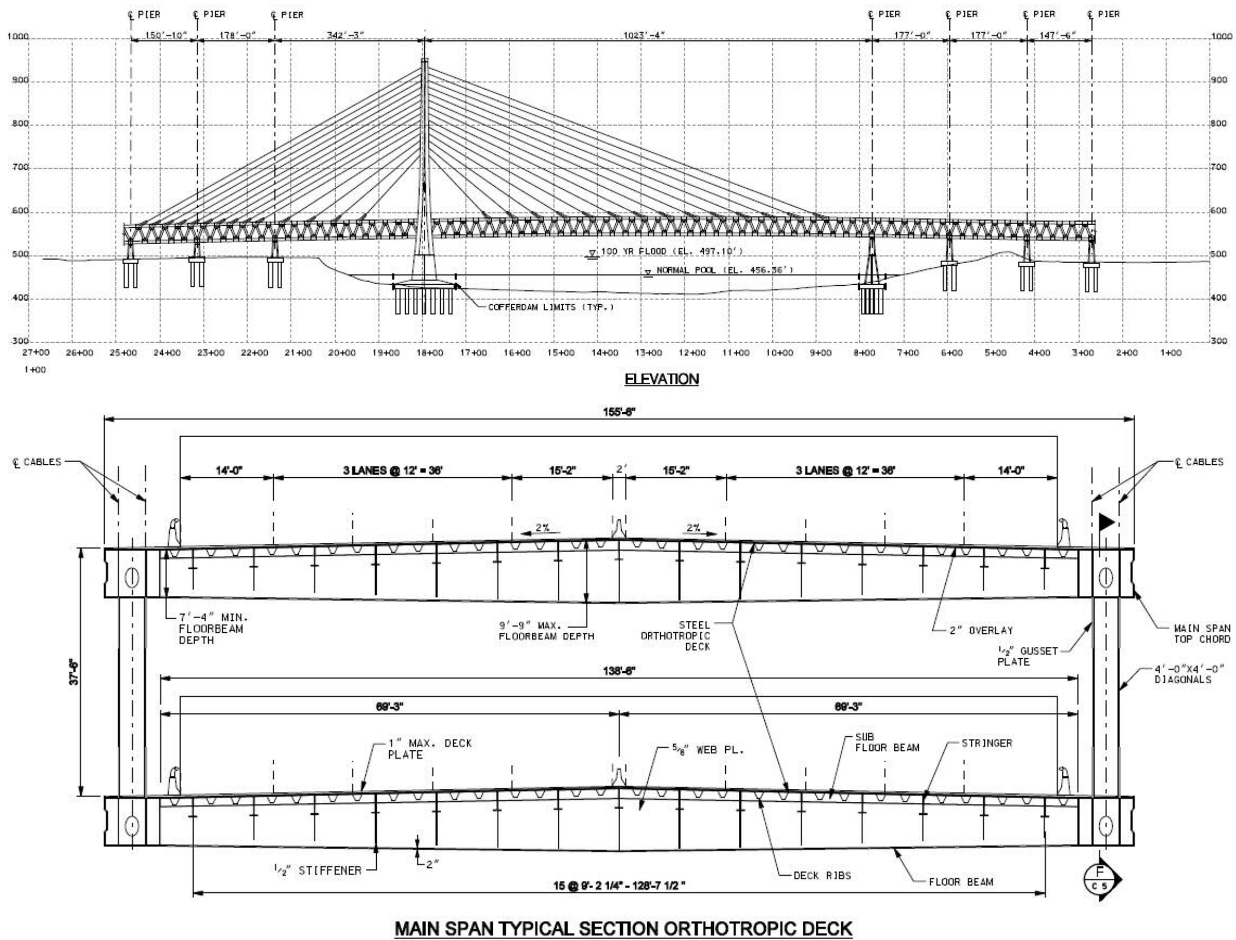

Figure 3. Alternate 6 – Single Tower Cable-stayed Bridge