University of California, Berkeley

CE 122/123L Bridge Design

Byzantine

Brian Tam
Omar Natouf
Shien Jinn Sam
Yanlong Ma
Yiqiu Zhang

Table of Contents

I. Letter of Transmittal

Davis 502
Berkeley, CA 94720
Yanlong Ma, PhD

April 28th 2017

Professor Stephen Mahin
Professor of Civil Engineering,
University of California, Berkeley
Berkeley, Ca 94720

Dear Professor Mahin,

We are Byzantine, a structural engineering firm specializing in bridge design and construction. It has come to our attention recently, that there is an open design bid for the proposed J St. Bridge. Our team, consisting of professionally licensed structural engineers, is writing to submit our design proposal for the proposed replacement bridge.

Given the geographical conditions of the site and the required bridge span, we are proposing an asymmetric cable stayed bridge. Our extensive design considerations-to be expounded on briefly- captures the combined interests of all stakeholders in this project;environment, safety, constructability, and financing.

Our basic design considerations were to provide a safe, easily constructable and economical design that would have minimal disruption to the existing environment while maintaining a pleasing design aesthetic. These considerations are paramount as societies increasingly value reliability and sustainability. They are also an exercise in practicality. Due to the enormous financing costs of such structures, having an economical and easily implementable design would expedite the financing process and reduce the economic disruption of local communities typically associated with longer construction schedules. The site requires that the proposed structure span 525 feet over the Sacramento River. For this particular geometry, it is determined that a cable stayed design would be most economical. Furthermore, having a suspension based superstructure negates the need for multiple supporting structures and columns within the river below, easing construction and reducing the environmental impact on the existing river ecosystem.

Our design utilizes an asymmetric support system, with two towers on a single bank bearing the load of the entire deck. This design, while unusual has several key structural advantages- having the supporting tower be placed on the bank negates the need for expensive construction equipment and procedures to be carried out to construct the structure in the river below. Cable stayed designs are also much more efficient, having significantly lower weights than their compression arch counterparts, increasing its seismic reliability. We have taken great care to ensure that all parts of our bridge meet a minimum factor of safety of 1.5.
Cables are strung along the tower in parallel to suspend the deck above the water at the required height. The parallel cable arrangement allows for loading to be evenly distributed along the cables and along the tower. Parallel backties from the tower to a ground anchor on the bank help stabilize the tower against the moment generated by the bridge load. The deck in contact with the bank opposite of the towers are attached on rollers to allow it to freely slide and prevent tension in the concrete deck.

When performing the modelling of our structure, we examined two main loading conditions: dead load and moving live load. Under the dead load conditions, we examined the deformations generated on the deck and tower of our SAP model and applied the required pre-stresses. We repeated a simulation of moving load conditions to examine or structure's behavior with moving cars on it. On both accounts, our design passes the stipulated deflection conditions.

Construction will be expedited by using prefabricated steel decks that will be consecutively suspended by a crane, and connected by cables to the supporting tower. Due to the linear nature of this procedure, the same process can be repeated with minimal procedure alterations as the decks get suspended. The estimated cost of our proposal is $53,254,439.48. We believe that this is a justified investment, since the proposed development will increase the transit capacity of the region, while upgrading the existing transportation infrastructure to be more seismically resilient and aesthetically pleasing. Increase capacity will drive economic growth within the region while increasing the productivity of commuters as they spend less time stuck in traffic. Given that no major seismic event has occurred in a while, we believe the improved structural reliability is a timely and necessary upgrade in anticipation and mitigation of future risks. We hope that our design and the technical details it entails in this report will convince you of the viability of our solutions.

Sincerely,
Yanlong Ma, PhD
CEO, Byzantine Structural Solutions

II. Financial Proposal

Overview

For cost estimation, we took considerations for the components of the cables, deck, and tower of the bridge.

Our cables were #37 and back ties were #127 in diameter. The length and diameters of each cable were used to calculate the volume of grad 270 cable steel, and we used the density of grade 270 steel to calculate our total cable weight. Our weight of steel was multiplied by the cost per unit weight for the cables to obtain the net cost of our cable system, approximately $935,567 for the material costs and $1,247,423 for the construction of the cables.

The tower consists of two reinforced concrete pylons, connected by two concrete bracings. The volume of these four components were geometrically estimated. The tower, in combination with the 8” slab in our deck consists of the main reinforced concrete components of our bridge and in total sums up to around 215,890 cubic feet of structural concrete costing $6,476,711.

For our rebar estimations we took the volume of structural concrete and assumed that 2% of the volume to be filled with steel rebar with a density of 490 pcf. In total, our structural rebar costs $12,694,354. The steel components were calculated to be $14,177,947 and $17,722,434 for the furnish and erect costs respectively. A 5% bonus was added to account for steel bolts, connections, and other extra components.

In total, our bridge costs a total of $53,254,439.

Cable Costs

Cable #	Length (ft)	# of Strands	Cable Diameter(mm)	Cable diameter (ft)	Volume of cable (cubic feet)	Density (pcf)	Weight (LB)
1	603	37	109.9	0.3605643	61.5458267	374	23018.13919
2	567	37	109.9	0.3605643	57.9398202	374	21669.49275

3	532	37	109.9	0.3605643	54.33382177	374	20320.84934
4	497	37	109.9	0.3605643	50.72783315	374	18972.2096
5	461	37	109.9	0.3605643	47.12185658	374	17623.57436
6	426	37	109.9	0.3605643	43.51589506	374	16274.94475
7	391	37	109.9	0.3605643	39.90995266	374	14926.32229
8	356	37	109.9	0.3605643	36.30403509	374	13577.70912
9	320	37	109.9	0.3605643	32.69815055	374	12229.10831
10	285	37	109.9	0.3605643	29.09231135	374	10880.52444
11	250	37	109.9	0.3605643	25.4865367	374	9531.964725
12	214	37	109.9	0.3605643	21.88085853	374	8183.441089
13	179	37	109.9	0.3605643	18.27533394	374	6834.974894
14	144	37	109.9	0.3605643	14.67007618	374	5486.608491
15	108	37	109.9	0.3605643	11.06534605	374	4138.439423
16	73	37	109.9	0.3605643	7.461908268	374	2790.753692
17	38	37	109.9	0.3605643	3.863380707	374	1444.904384
18	235	127	204.1	0.6696194	82.75865648	374	30951.73752
19	194.5	127	204.1	0.6696194	68.4959944	374	25617.50191
20	154	127	204.1	0.6696194	54.23333233	374	20283.26629
21	113	127	204.1	0.6696194	39.79458801	374	14883.17592
22	72.5	127	204.1	0.6696194	25.53192593	374	9548.9403
							309188.5828
							Total Weight

Deck Costs

			Total Self Weight(kip)
Deck	8" Slab	Concrete	4462.5
Deck	Pavement , needs data		
Deck	Stringers	Steel W30x 391	1055.7
Note A on dwg	Girder Type 1	Built-up: 18x4 flange, 3 x 32 web	1298.08685

Note B on dwg	Girder Type 2 (Not Used)	Steel W30x261	0
Deck	Bottom Deck (Not Used)	Steel 40 in x 40 in x 2.5 in	0
Note C on dwg	Edge Girder	Box Steel HSS 5 x 4.5 x 0.75 in	1190.7

Tower Costs

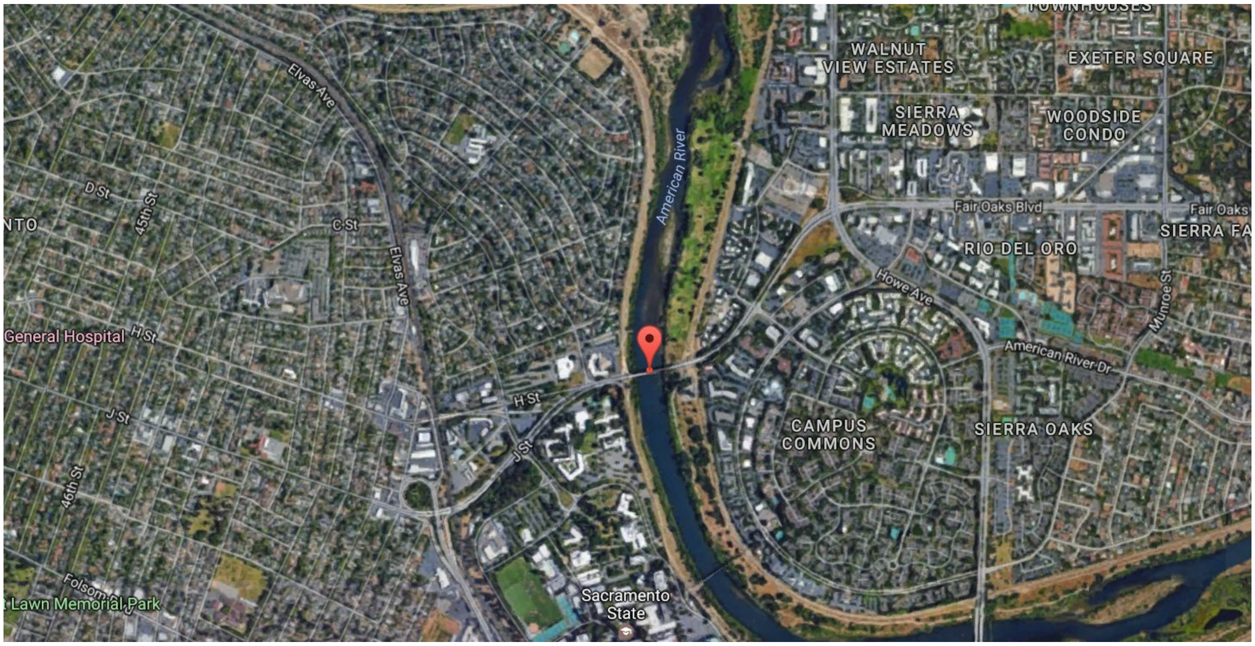
Pylon diameter 1	3	ft
Pylon diameter 2	18	ft
Pylon Height	228	ft
Pylon Volume	92400.44508	ft^3
Tower as a whole	184800.8902	ft^3
Concrete Bracing 1	689.1863063	ft^3
Concrete Bracing 2	650.30913	ft^3

Total Costs per Material

Material	Unit price	In Units Of	Unit (LB or CF)	Material Cost
Furnish Structural Steel	4	$ / LB	3544486.85	$14,177,947.40
Erect Structural Steel	5	$ / LB	3544486.85	$17,722,434.25
Furnish Cable System	4.5	$ / LB	207903.9609	$935,567.82
Erect Cable System	6	$ / LB	207903.9609	$1,247,423.77
Bar Reinforcement Steel (2% volume of structural concrete)	6	$ / LB	2115725.779	$12,694,354.67
Structural Concrete	30	$ / CF	215890.3856	$6,476,711.57
				$53,254,439.48
				Total Cost

III. Technical Proposal

General Project Description



The bridge will replace the existing J St, Bridge near CSU Sacramento over the American River (38.568903, -121.423128). The proposed new bridge will carry 6 lines of traffic connecting J St. and Fair Oaks Blvd.

Structural system

The type of the bridge is **cable stay bridge** with two towers on the east side of the river bank. The two tower are parallel and tilt to the east side. Both the towers are cone shapes and same design features. The Table 3.1 shows some dimensions of the design bridge.

Table 3.1: Design Dimensions

Main Structure	Category	Size	Unit
Length	Deck	525	ft
Width	Deck	89	ft
Height	Tower	228	ft
Tilt Angle	Tower	15	--

Diameter (Bottom)	Tower	18	ft
Diameter (Top)	Tower	4	ft
Height	Abutment	22	ft

Codes and references

The design confirms the:

- ACI 318: Building Code Requirements for Structural Concrete (ACI)
- AISC Steel Construction Manual, 14^{th} Edition (AISC)
- Some details part follow the special guides.

Loading criteria and assumption

The design length of the main bridge is 525 feet and 50 feet above the water level. The service distributed live load is 0.64 kip / ft / lane and two 25 kip axles per lane spaced at 4 feet. The reductor used for live load is 0.65. Follow the load combination in ACI, the Load = 1.2 Dead Load + 1.6 Live Load. The tandem load place on the left end of the bridge which will produce the extreme force effect.

Materials

The Materials used for the bridge design are summarized in the Table 3.2 below.

Table 3.2 Material Properties

Member	Material	Grade	Unit
Deck	Concrete	8	ksi
Deck	Steel Rebar	60	ksi
Stringer	Steel	50	ksi
Girder	Steel	50	ksi
Edge Girder	Steel	50	ksi
Cable	Steel	270	ksi

Tower	Concrete	6	ksi
Tower	Steel Rebar	60	ksi
Abutment	Concrete	6	ksi
Abutment	Steel Rebar	60	ksi
Fundation	Concrete	6	ksi

Geotechnical

The preliminary geotechnical assumption show in the Table 3.3 below.

Table 3.3: Geotechnical Assumption

Limit State	Nominal Bearing Resistance, q_n
Service	20 ksf
Strength	50 ksf

Design of special conditions / elements

Basically, the bridge will carry the dead load of itself, the distributed live load, and a point lived load. The service distributed live load is 0.64 kip / ft / lane and two 25 kip axles per lane spaced at 4 feet.

For both distributed live load and dead load, the loads are transformed into distributed load (kips / ft^2) to do the calculation.

The live load impact factor used for this bridge is 0.65, and the combination factors for live load and dead load are 1.6 and 1.2 ($L = 1.2\ L_D + 1.6\ L_L$).

The calculation of loads for superstructure is shown in the Appendix Excel file (A1).

Design of typical elements

i.Superstructure

There are 4 components for the superstructure: concrete slab, steel stringer, steel girder, and steel edge girder. There properties are shown in the Table below.

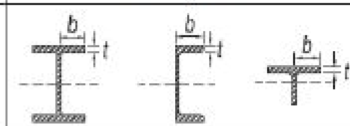
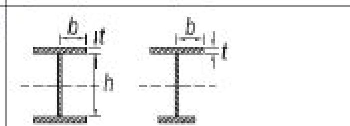
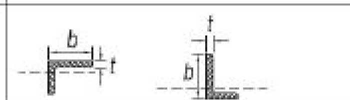
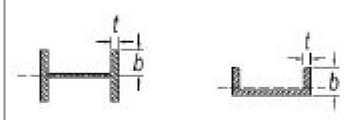
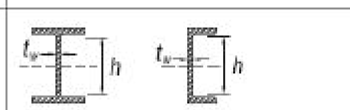
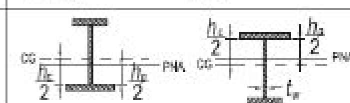
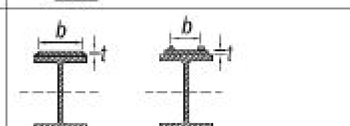
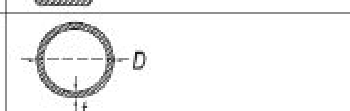
Superstructure Properties

Components	Material	Grade	Notes
Slab	Concrete	8 ksi	Steel ratio in the concrete is 2%
	Steel Rebar	60 ksi	
Stringer	Steel	A572 (50 ksi)	W30 391
Girder	Steel	A572 (50 ksi)	Built up W shape 22”4” Flange, 4”22” Web
Edge Girder	Steel	A572 (50 ksi)	Built up HSS, 12”78”2” for deck #1 - #8, 24”78”2” for deck # 9 - #17

The capacity design for each member basic follow the AISC codes. For stringer and girder, the local buckling and lateral buckling are the mainly checking parts. For Edge girder, the overall buckling is the key factor to determine the capacity. For all the member, the required shears are much smaller than the capacity.

The compression elements checking using the Table below.

TABLE B4.1b
Width-to-Thickness Ratios: Compression Elements Members Subject to Flexure

	Case	Description of Element	Width-to-Thickness Ratio	Limiting Width-to-Thickness Ratio λ_p (compact/noncompact)	Limiting Width-to-Thickness Ratio λ_r (noncompact/slender)	Examples
Unstiffened Elements	10	Flanges of rolled I-shaped sections, channels, and tees	b/t	$0.38\sqrt{\frac{E}{F_y}}$	$1.0\sqrt{\frac{E}{F_y}}$	
Unstiffened Elements	11	Flanges of doubly and singly symmetric I-shaped built-up sections	b/t	$0.38\sqrt{\frac{E}{F_y}}$	[a] [b] $0.95\sqrt{\frac{k_c E}{F_L}}$	
Unstiffened Elements	12	Legs of single angles	b/t	$0.54\sqrt{\frac{E}{F_y}}$	$0.91\sqrt{\frac{E}{F_y}}$	
Unstiffened Elements	13	Flanges of all I-shaped sections and channels in flexure about the weak axis	b/t	$0.38\sqrt{\frac{E}{F_y}}$	$1.0\sqrt{\frac{E}{F_y}}$	
Unstiffened Elements	14	Stems of tees	d/t	$0.84\sqrt{\frac{E}{F_y}}$	$1.03\sqrt{\frac{E}{F_y}}$	
Stiffened Elements	15	Webs of doubly-symmetric I-shaped sections and channels	h/t_w	$3.76\sqrt{\frac{E}{F_y}}$	$5.70\sqrt{\frac{E}{F_y}}$	
Stiffened Elements	16	Webs of singly-symmetric I-shaped sections	h_c/t_w	[c] $\frac{\frac{h_c}{h_p}\sqrt{\frac{E}{F_y}}}{\left(0.54\frac{M_p}{M_y}-0.09\right)^2} \le \lambda_r$	$5.70\sqrt{\frac{E}{F_y}}$	
Stiffened Elements	17	Flanges of rectangular HSS and boxes of uniform thickness	b/t	$1.12\sqrt{\frac{E}{F_y}}$	$1.40\sqrt{\frac{E}{F_y}}$	
Stiffened Elements	18	Flange cover plates and diaphragm plates between lines of fasteners or welds	b/t	$1.12\sqrt{\frac{E}{F_y}}$	$1.40\sqrt{\frac{E}{F_y}}$	
Stiffened Elements	19	Webs of rectangular HSS and boxes	h/t	$2.42\sqrt{\frac{E}{F_y}}$	$5.70\sqrt{\frac{E}{F_y}}$	
Stiffened Elements	20	Round HSS	D/t	$0.07\frac{E}{F_y}$	$0.31\frac{E}{F_y}$	

[a] $k_c = 4/\sqrt{h/t_w}$ but shall not be taken less than 0.35 nor greater than 0.76 for calculation purposes.
[b] $F_L = 0.7F_y$ for major axis bending of compact and noncompact web built-up I-shaped members with $S_{xt}/S_{xc} \ge 0.7$;
$F_L = F_y S_{xt}/S_{xc} \ge 0.5F_y$ for major-axis bending of compact and noncompact web built-up I-shaped members with $S_{xt}/S_{xc} < 0.7$.
[c] M_y is the moment at yielding of the extreme fiber. M_p = plastic bending moment, kip-in. (N-mm)
E = modulus of elasticity of steel = 29,000 ksi (200 000 MPa)
F_y = specified minimum yield stress, ksi (MPa)

Equation used for Lateral buckling

a. When $L_b \leq L_p$:

$$M_n = M_p$$

b. When $L_p < L_b \leq L_r$:

$$M_n = C_b\left[M_p - (M_p - 0.7F_y S_x)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \leq M_p$$

c. When $L_b > L_r$:

$$M_n = F_{cr} S_x = C_b S_x \frac{\pi^2 E}{\left(\frac{L_b}{r_{ts}}\right)^2}\sqrt{1 + 0.078\frac{Jc}{S_x h_o}\left(\frac{L_b}{r_{ts}}\right)^2} \leq M_p$$

Where

$$\mathrm{Lp} = 1.76 * \mathrm{ry} * \left(\frac{\mathrm{E}}{\mathrm{Fy}}\right)^{.5}$$

$$\mathrm{Lr} = 1.95 * \mathrm{rts} * \frac{\mathrm{E}}{0.7 * \mathrm{Fy}} * \left(\frac{\mathrm{J}}{\mathrm{Sx} * \mathrm{h0}} + \left(\left(\frac{\mathrm{J}}{\mathrm{Sx} * \mathrm{h0}}\right)^2 + 6.76 * \left(0.7 * \frac{\mathrm{Fy}}{\mathrm{E}}\right)^2\right)^{.5}\right)^{.5}$$

Shear Design

$$V_n = 0.6 F_y A_w\ C_v$$

where

a. for webs of rolled-shaped members with $h/t_w \leq 2.24\sqrt{E/F_y}$

$$C_v = 1.0 \tag{7.8}$$

b. for webs of all other doubly symmetric and singly symmetric shapes and channels, except round HSS, the web shear coefficient C_v. is established based on "3-segment" curve in Figure 7.8 on the next age:

(a) For $h/t_w \leq 1.10\sqrt{k_v E/F_y}$

$$C_v = 1.0 \tag{7.9a}$$

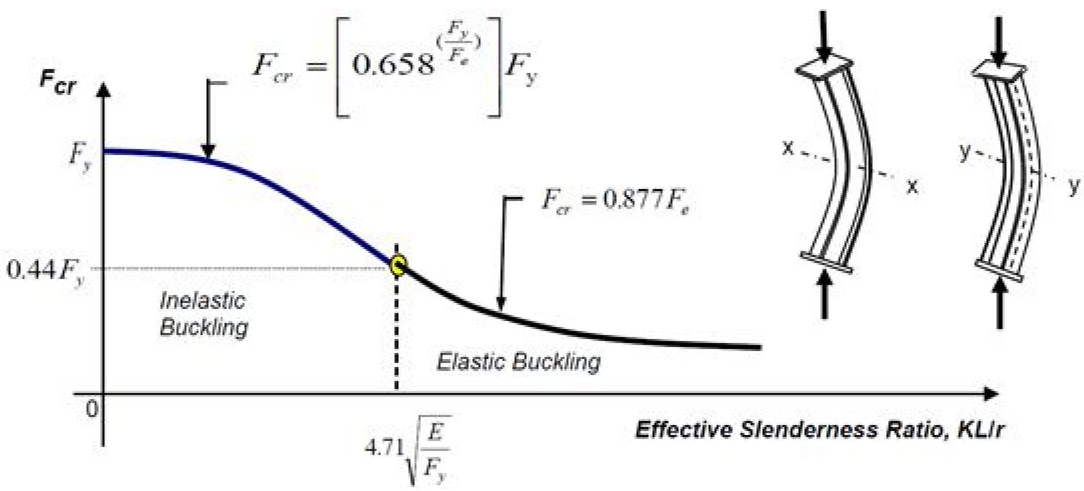
(b) For $1.10\sqrt{k_v E / F_y} < h / t_w \le 1.37\sqrt{k_v E / F_y}$

$$C_v = \frac{1.10\sqrt{k_v E / F_y}}{h / t_w} \tag{7.9b}$$

(c) For $h / t_w > 1.37\sqrt{k_v E / F_y}$

$$C_v = \frac{1.51 E k_v}{(h / t_w)^2 F_y} \tag{7.9c}$$

Equation for Overall buckling



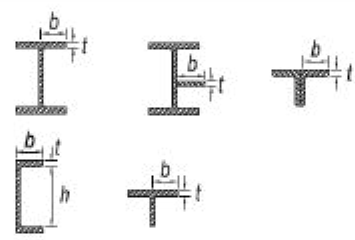
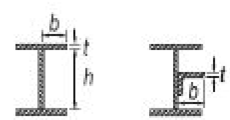
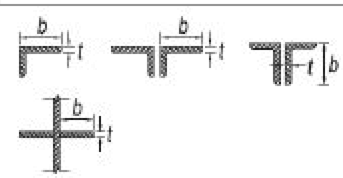
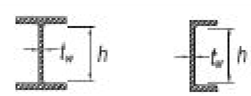
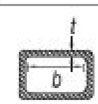
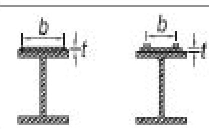
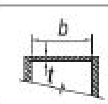
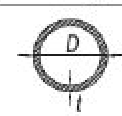
$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2}$$

Where Fe determined by Euler's Buckling Equation

K = 1.0 (pin-pin condition) for edge girder

The table used to check local buckling

TABLE B4.1a
Width-to-Thickness Ratios: Compression Elements
Members Subject to Axial Compression

	Case	Description of Element	Width-to-Thickness Ratio	Limiting Width-to-Thickness Ratio λ_r (nonslender/slender)	Examples
Unstiffened Elements	1	Flanges of rolled I-shaped sections, plates projecting from rolled I-shaped sections; outstanding legs of pairs of angles connected with continuous contact, flanges of channels, and flanges of tees	b/t	$0.56\sqrt{\frac{E}{F_y}}$	
	2	Flanges of built-up I-shaped sections and plates or angle legs projecting from built-up I-shaped sections	b/t	$0.64\sqrt{\frac{k_cE}{F_y}}$ [a]	
	3	Legs of single angles, legs of double angles with separators, and all other unstiffened elements	b/t	$0.45\sqrt{\frac{E}{F_y}}$	
	4	Stems of tees	d/t	$0.75\sqrt{\frac{E}{F_y}}$	
Stiffened Elements	5	Webs of doubly-symmetric I-shaped sections and channels	h/t_w	$1.49\sqrt{\frac{E}{F_y}}$	
	6	Walls of rectangular HSS and boxes of uniform thickness	b/t	$1.40\sqrt{\frac{E}{F_y}}$	
	7	Flange cover plates and diaphragm plates between lines of fasteners or welds	b/t	$1.40\sqrt{\frac{E}{F_y}}$	
	8	All other stiffened elements	b/t	$1.49\sqrt{\frac{E}{F_y}}$	
	9	Round HSS	D/t	$0.11\frac{E}{F_y}$	

[a] $k_c = 4/\sqrt{h/t_w}$ but shall not be taken less than 0.35 nor greater than 0.76 for calculation purposes.

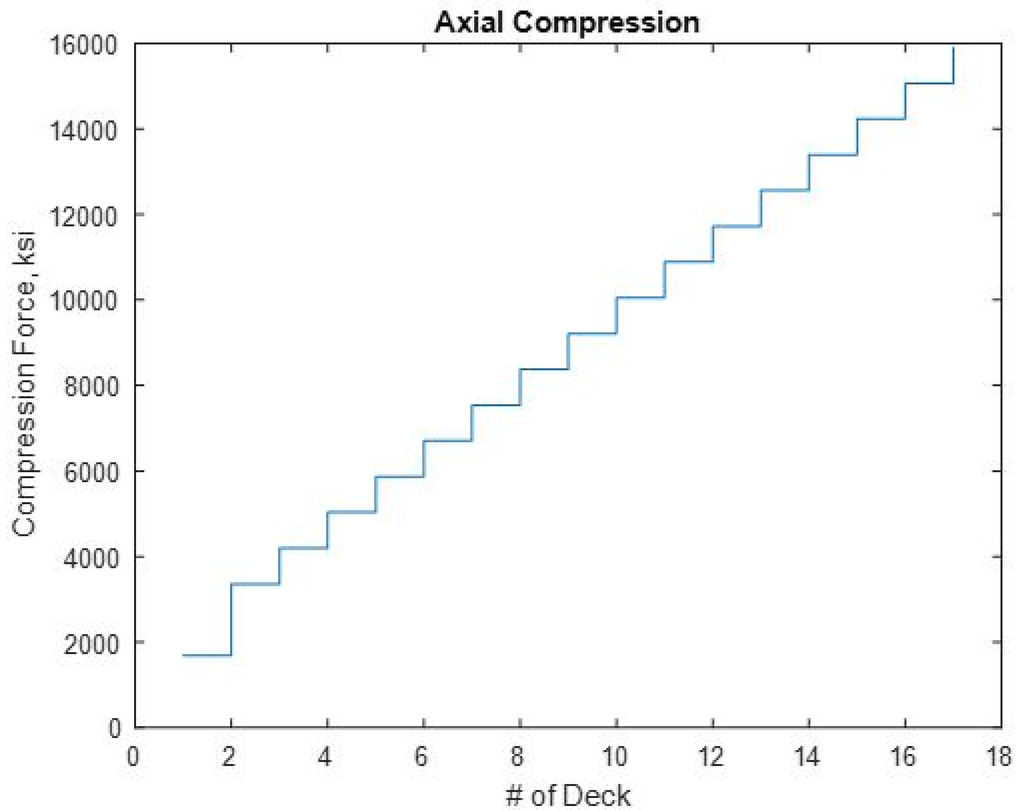
The capacities of the members are summarized in the table below.

	Stringer	Girder	Edge Girder
Local Buckling (k-ft)	Compact	Compact	Compact
Overall bucking (kips)	--	--	16045*
Lateral Buckling (k-ft)	5267	8955	13911
Shear (kips)	1554	4560	4920
*Note: The concrete slab will carry part of compression (474 ksi)			

There are shear studs will connect the concrete slab to the edge girder to resist a part compression in the edge girder.

Because the compression will decrease along the edge girder, there are two type of edge girder. The strong one has a large capacity with dimension 24” * 78” * 2”, and the small one has a dimension 12” * 78” * 2”.

The figure shows the axial compression along the girder.



ii.Front Cable

The cable consists with the seven-wire strand. Some properties of the material are listed below.

Table: Properties of Seven-Wire Strand Cable

Seven-Wire Strand Cable	Properties
Diameter	15.7 mm
Effective Area	150 mm
Grade	270
Tensile Capacity	62.775 ksi

The front cable designed for the bridge consisted with 37 seven-wire strand. The total tensile capacity for the front cable is 2323 ksi.

There are 16 front cables (each side) to carry all the load from the deck and the girders. The ratio of height over length is 1:2.5.

iii.Back Cable

The back cable consisted with the same material as the front one. The number of strand for each back cable is 127, which will create a 7972 ksi tensile capacity.
There are 6 back cables (each side) to balance the moment created by the front cable.

iv.Pylon

There are two parallel pylons both on the east side of the river. The two pylons have the exact same design. The basic dimensions for the pylons are listed in the Table below.

Table: Properties of Pylons

Pylon (each)		Quantity	
Shape		Cone	
Height (ft)		228	
Length (ft)		236.04	
Tilt Angle (Deg)		15	
Bottom Diameter (ft)		18	
Top Diameter (ft)		4	
Materials	Steel (ksi)	60	Steel ratio is 0.02
	Concrete (ksi)	6	
Weight (ksi)		4000	

The capacity design for each member basic are very close to the design method above. The moment of the pylons and the compression of the pylons are the main design parts.

The moment design use the equation . The equation basically use for rectangular box design. Therefore, b = r, d = 2r for this design.

The axial force design follow the same step above (overall buckling), but change the Euler number K to 2 (fix and free).

The cross beam between the pylons only resist the compression force. Therefore, check the overall buckling same as above.
The design capacities are shown below.

Table: Pylons

Variables	Quantity
Moment	4.8*10^6 k-ft
Axial Force	9.6*10^4 kips
Axial Force (Cross Beams)	5584 kips

Foundation

The foundation systems consisted with 2 parts. The abutment for the west side and the deep foundation on the east side of the river.

East side foundation

Because the pylons are on the east side of the bridge, the compression from the edge girder will cause a large shear and compression on the east side bank. Therefore, the east side cannot use the soil to resist the forces. Meanwhile, the back cable need to be anchored on the east side foundation.
The design is to use concrete fill the top 10 feet ground surface to resist the compression force. However, for the back part, the foundation should touch the bedrock and anchored the back cable into the rock and use concrete to enforce the connection part.

Abutment

The left side of the abutment will not resist the horizontal force from the superstructure. Therefore, the design only resist the load from the last segment of the deck and the soil pressure from the bank.

The detail designs are shown below.

The abutment is a reinforced concrete abutment with stem of 20 feet height and and 4 feet thickness. The type of abutment we choose is open end short stem seat. The seat width is thirty six inches wide. . The foundation for the abutment is two feet deep and ten by ten feets.The ratio of steel reinforcement is 0.2 percent.

We design the foundation of the abutment based on the allowable soil pressure at service area. Then we used the $q(net) = Pu/A$. We evaluate the one way and two ways shear and the flexure of the foundation. Based on our calculation we needed 20 #6, more details for the anchorage length and bearing between the stem and footing to be done.

The stem design was based on the soil pressure on the wall. We designed the stem to be adequate for shear and moment at the bottom which is the critical part.

The equations used are below.

Vc=0.75*2*Sqrt(4000)*b*d

Mu=0.9*As*fy*b*d(1-0.59*As*fy/(fc*b*d))

Vu=0.75*Vc

The load analysis we have done is based on AASHTO 2010.

Undeformed Shape

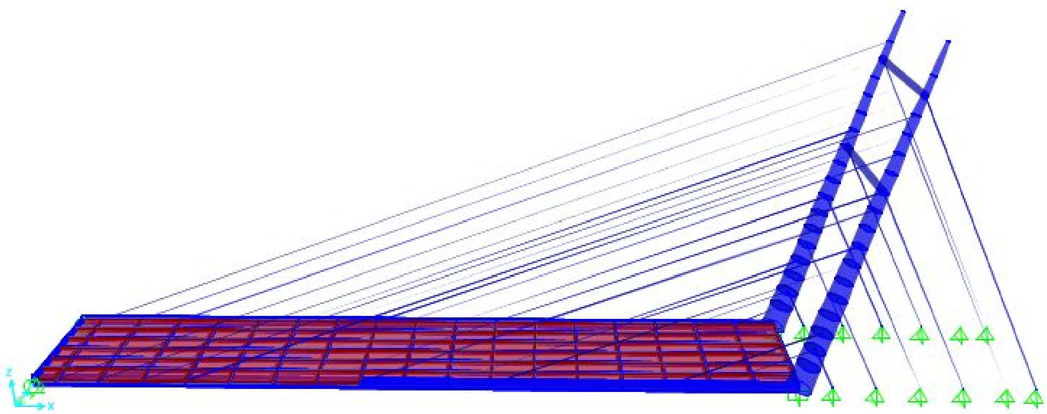


Figure. 1

Load combination one

A live load of 0.64 k/ft/lane and two 25k/lane four feet apart to represent a random cars moving on the bridge. During this load combination we do not account to any wind load.

Load combination two

The dead load with 1.25 factor represent the gravity load of the structure with safety factor.

Load combination three

The other load combination we used is the earthquake load of 0.15 of Deal load.

Results:

Moving load deflection **Figure. A**

Moving load and dead load combination deflection **Figure. B**

Dead load deflection **Figure.C**

The model deflection **Figure. D**

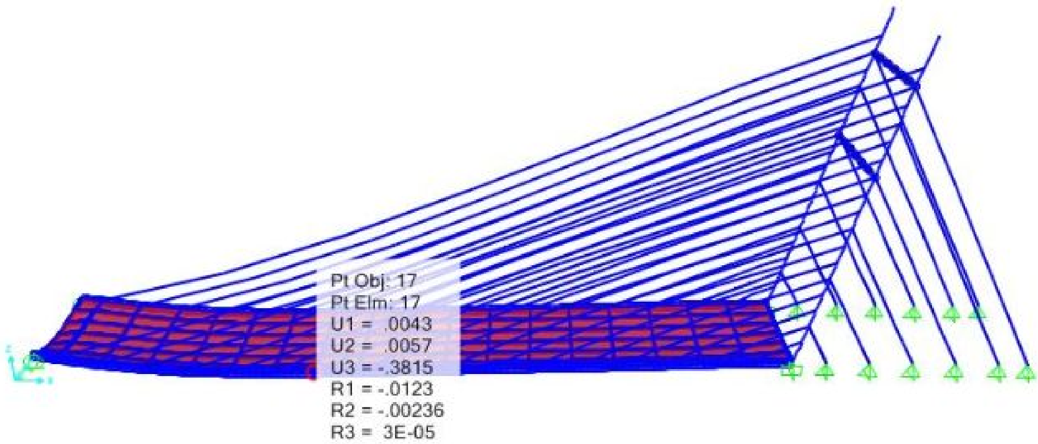


Figure. A

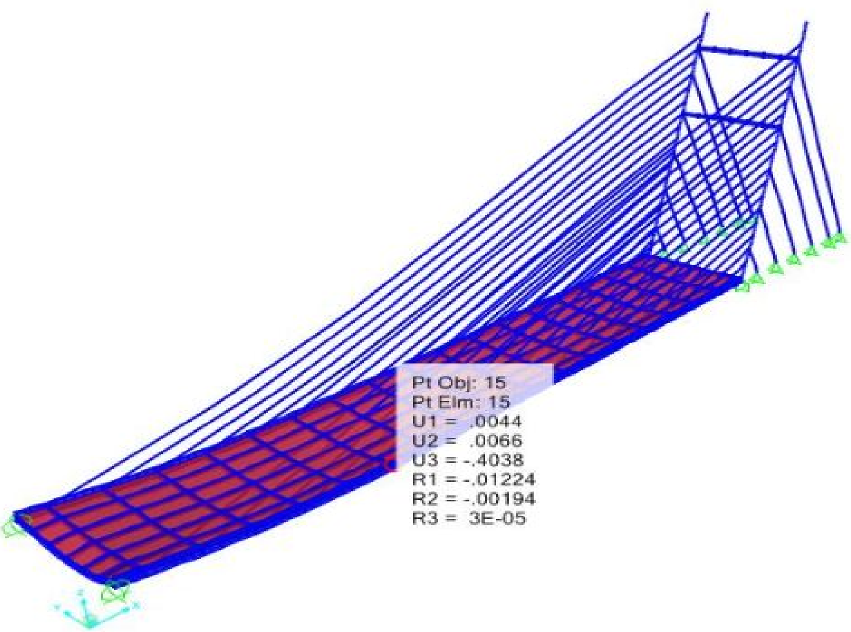


Figure. B

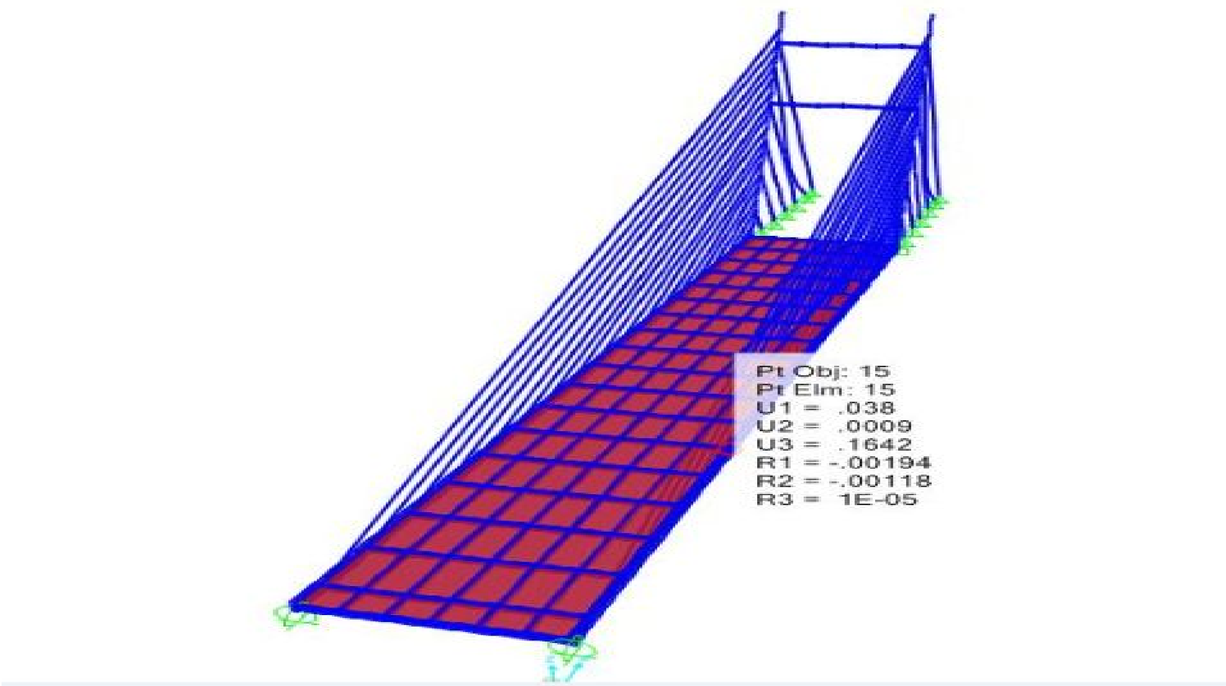


Figure.C

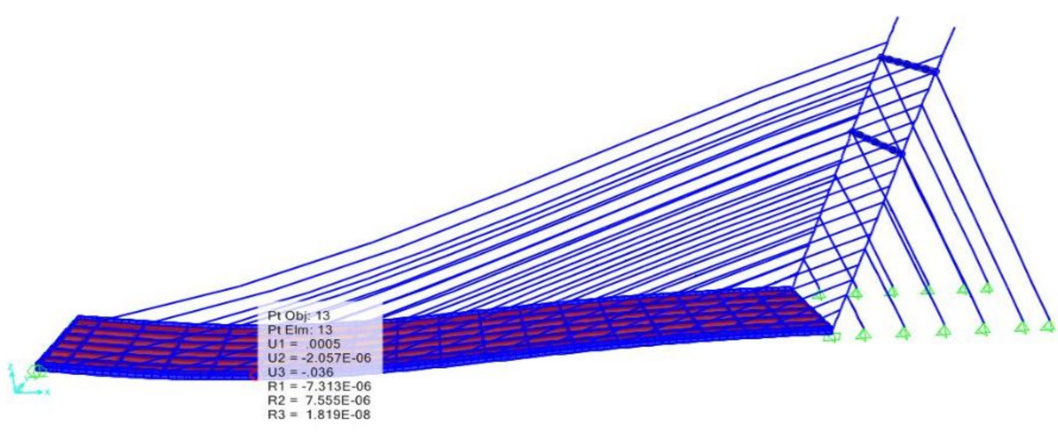


Figure. D

The lane loads **Figure.E**

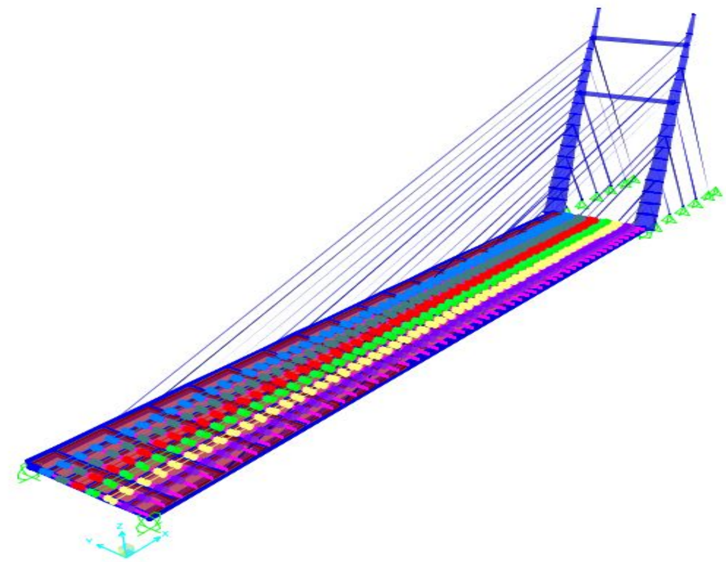


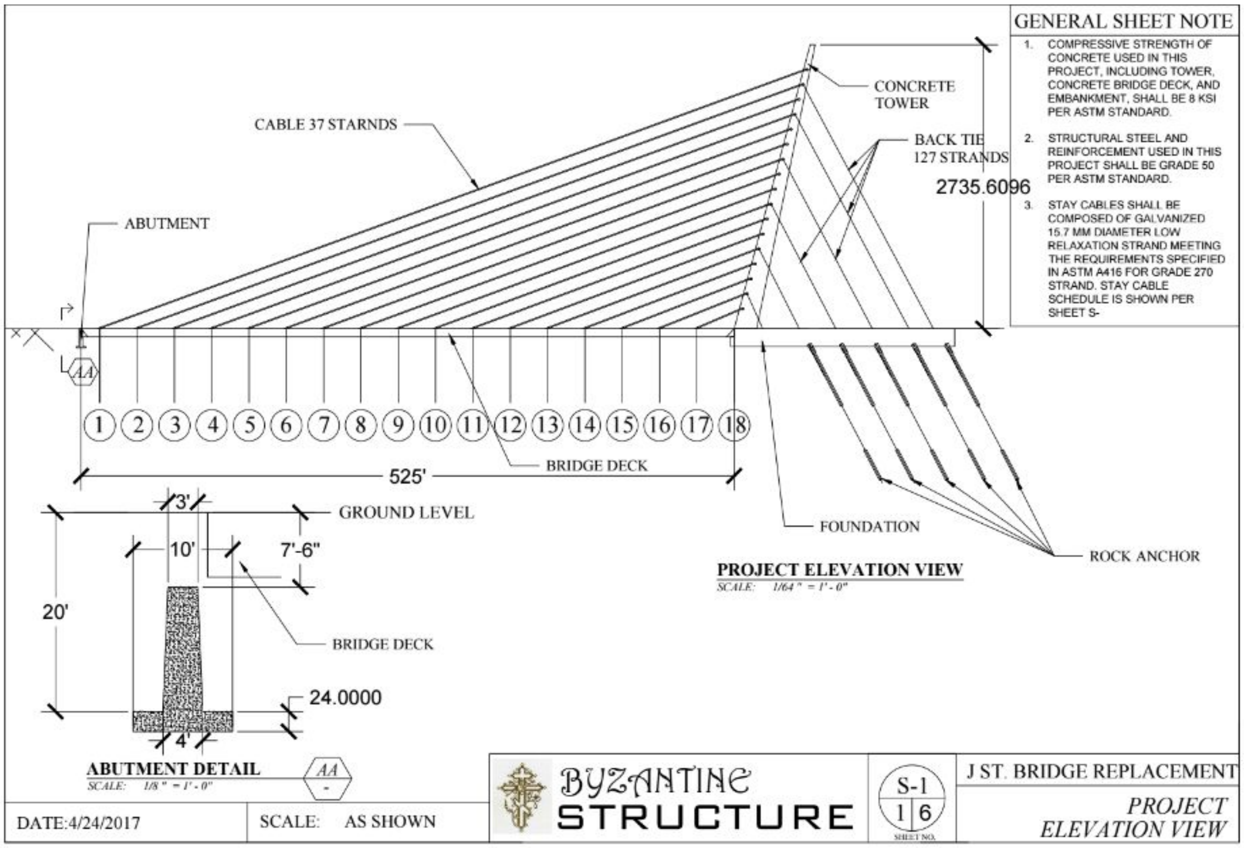
Figure. E

Constructability

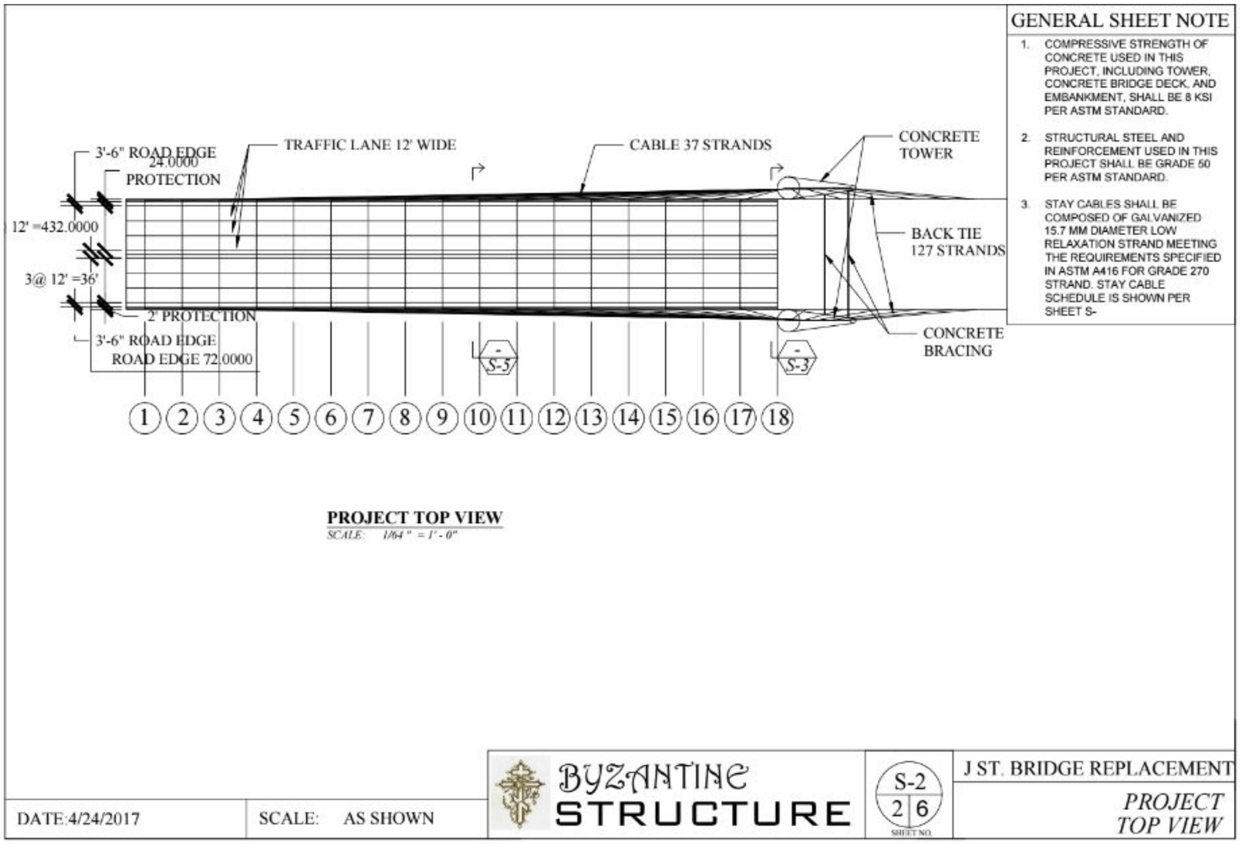
The construction of the cable stay bridge.
Step one:
Construct the adequate foundations and the abutment
Step two:
Construct the pylons
Step three:
Add the cable based on the deck constructions
Step four:
Add the deck sections of 60 ft long
The deck and cable construction will go hand by hand.
We recommended a use of cranes and ships to construct the deck which allow for minimum risk.

Design Drawings

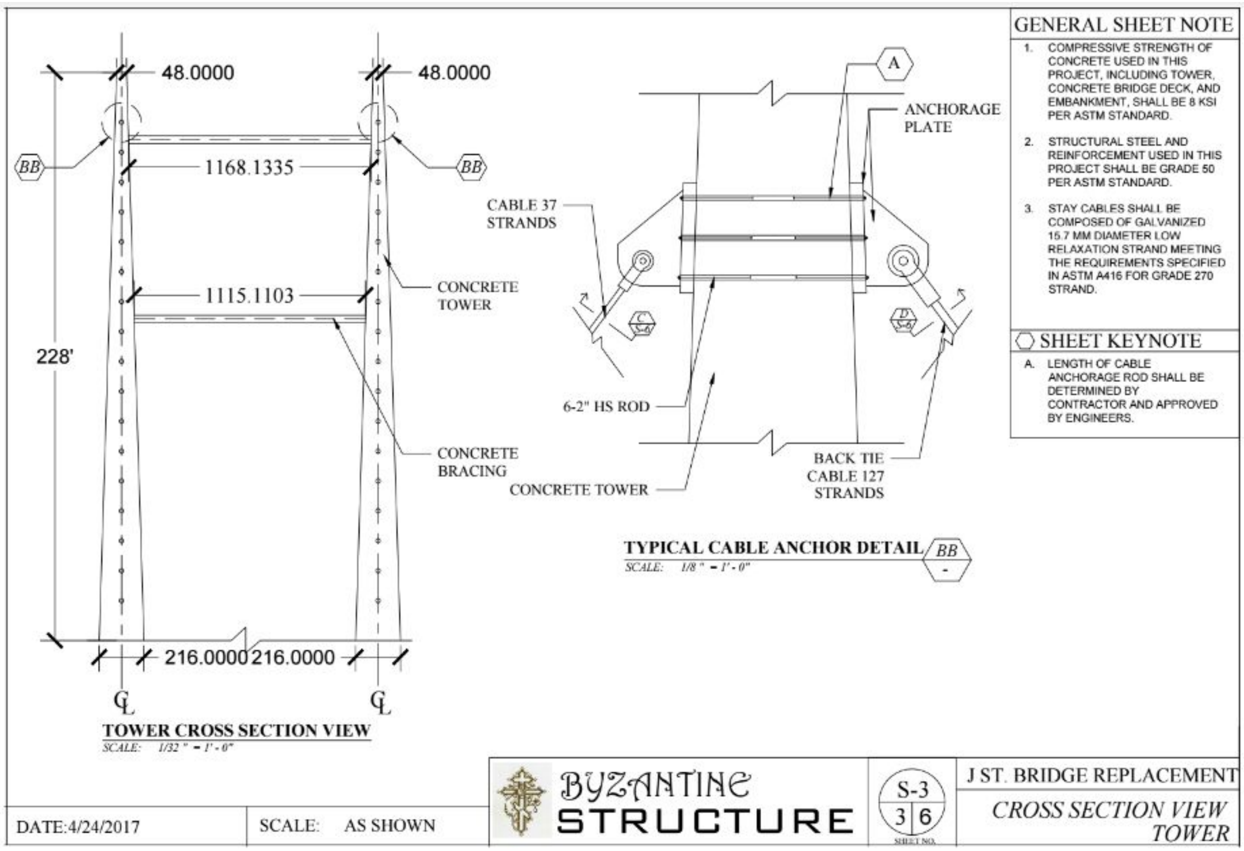
Elevation view of proposed structure



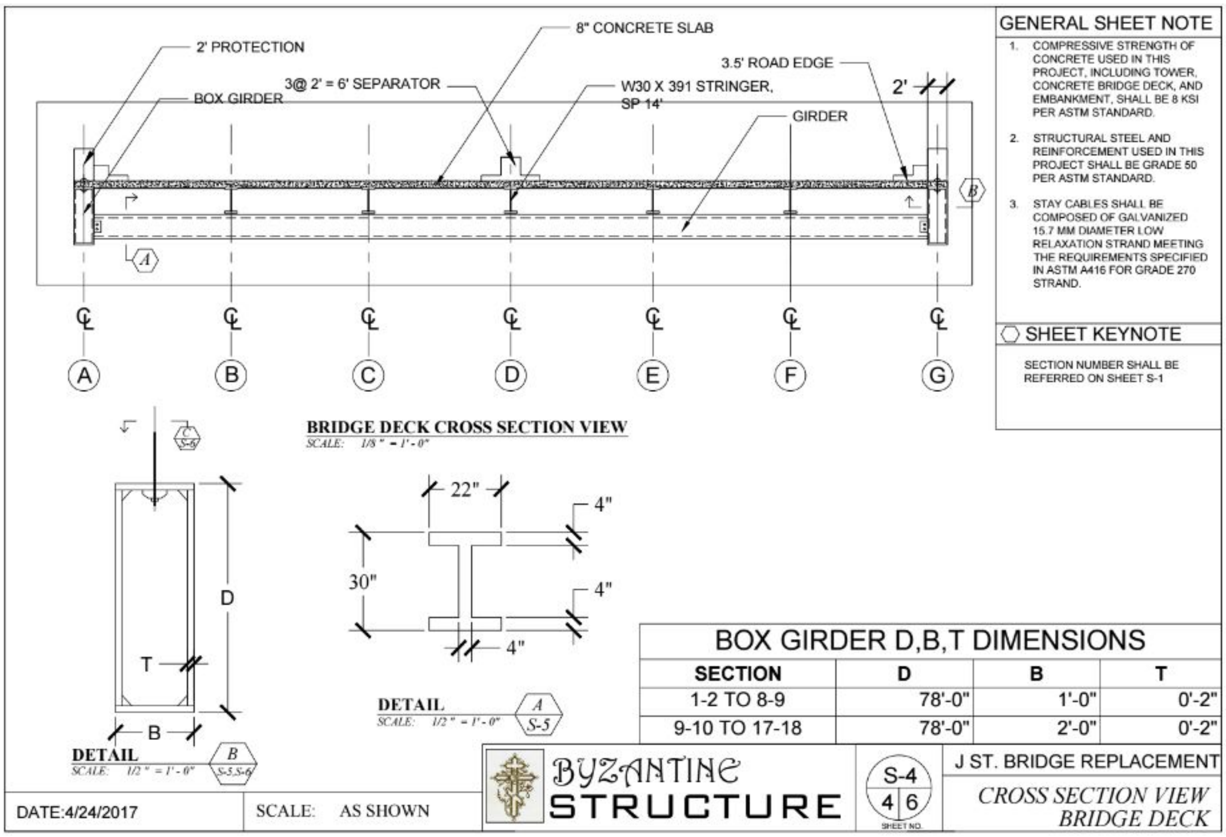
Plan view of proposed structure:



Tower Cross Section

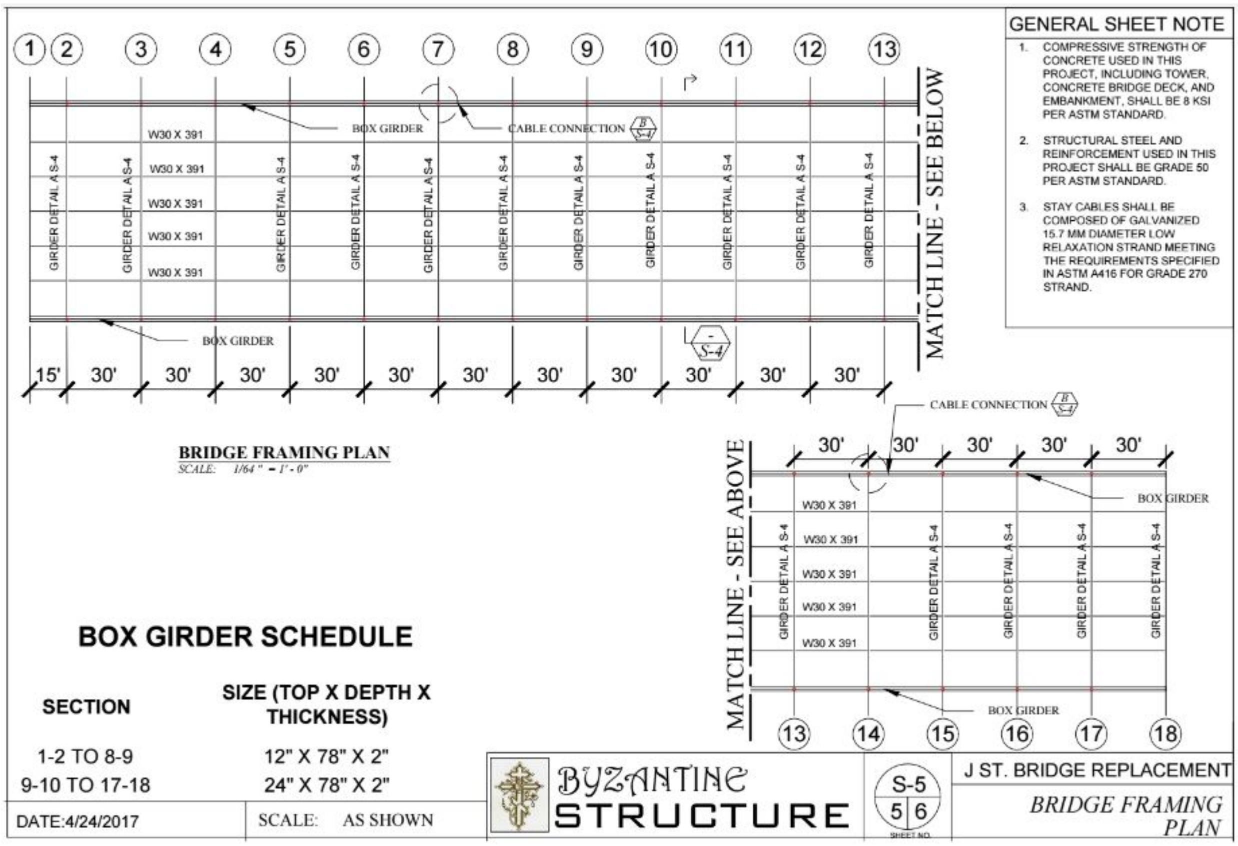


Bridge Deck Cross Section

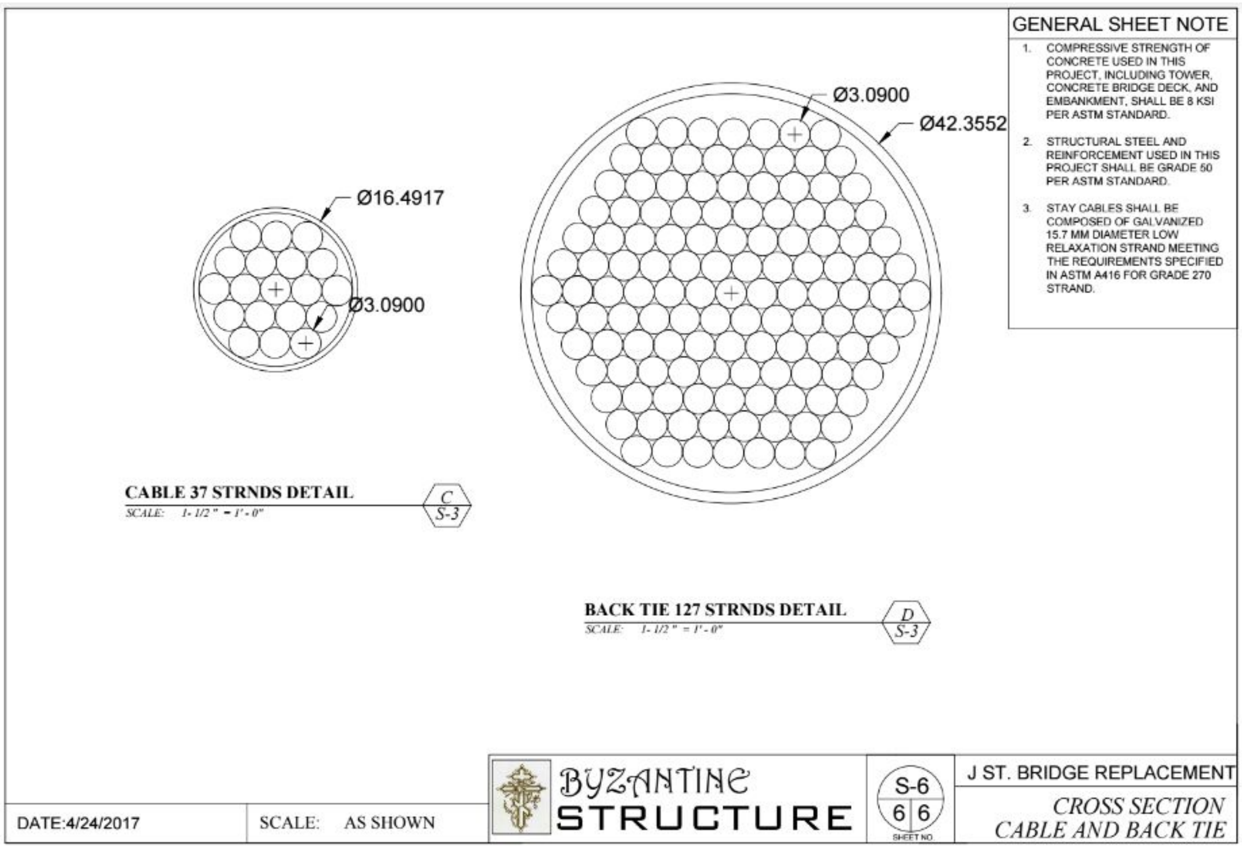


SECTION	D	B	T
1-2 TO 8-9	78'-0"	1'-0"	0'-2"
9-10 TO 17-18	78'-0"	2'-0"	0'-2"

Bridge Framing Plan



Cable


GENERAL SHEET NOTE
1. COMPRESSIVE STRENGTH OF CONCRETE USED IN THIS PROJECT, INCLUDING TOWER, CONCRETE BRIDGE DECK, AND EMBANKMENT, SHALL BE 8 KSI PER ASTM STANDARD.
2. STRUCTURAL STEEL AND REINFORCEMENT USED IN THIS PROJECT SHALL BE GRADE 50 PER ASTM STANDARD.
3. STAY CABLES SHALL BE COMPOSED OF GALVANIZED 15.7 MM DIAMETER LOW RELAXATION STRAND MEETING THE REQUIREMENTS SPECIFIED IN ASTM A416 FOR GRADE 270 STRAND.
Ø16.4917
Ø3.0900
Ø3.0900
Ø42.3552
CABLE 37 STRNDS DETAIL
SCALE: 1-1/2" = 1'-0"
C
S-3
BACK TIE 127 STRNDS DETAIL
SCALE: 1-1/2" = 1'-0"
D
S-3
DATE:4/24/2017
SCALE: AS SHOWN
BYZANTINE
STRUCTURE
S-6
6 6
SHEET NO
J ST. BRIDGE REPLACEMENT
CROSS SECTION
CABLE AND BACK TIE

IV. Appendix

Excel Load Calculations Sheet

			Spacing (ft)	Width(ft)	Length(ft)	Area(sq. ft)	Depth (ft)	concrete(pcf))	Area(kip)	(ksf)	(ksf)	Load (ksf)	(kip)	Location	Load (klf)	Facto
Deck	8" Slab	Concrete	-	85	525	44625	0.666666667	150	4462.5	0.1	0.0294	0.167	-		14.1936	
Deck	Pavement , needs data															
Deck	Stringers	Steel W30x 391	14	14	30	420	* Fill in if built-up section is used	391	11.73	0.1279	0.0294	0.2005	81.25	For 19' spacing, 5*25 kip at middle of joist for maximum moment; at one end for maximum shear	2.806969412	
Note A on dwg	Girder Type 1	Built-up: 22x4 flange, 4x 22 web	30	30	85	2550	* Fill in if built-up section is used	898.33	76.35805	0.1579	0.0294	0.2364	(75,100,75)*0.65	@ 19', 3*25 = 75, @ 38', 4*25 = 100, @ 57' 3*25 = 75 for maximum moment; @19' 4*25 = 100, @38' 3*25 = 75', @57' 3*25 =75, for maximum shear	7.092930454	
Note C on dwg	Edge Girder	Box Steel HSS 5 x 4.5 x 0.75 in	-	42.5	30	1275		1134	34.02	0.1846	0.0294	0.2684	50*0.65	2*25 = 50 at middle for maximum moment, and at one end for maximum shear	11.40911814	

Matlab Codes and Design Tools

<u>Beam Capacity Check:</u>

```
clear all; clc

Lb = 25*12;
E = 29000; Fy = 50;
A = 115; tf = 2.44; bf = 15.6; h = 26.5; tw = 1.36 ; d = 33.2; Cb = 1.14;

ry = 0; %% if do not know, ry = 0
rts = 0;    %% if do not know, rts = 0
J = 0;  %% if do not know, J = 0
Sx = 1250; %% if do not know, Sx = 0
Z = 0;

h0 = d-tf;

if ry==0
   Iy = tf*bf^3/6;
   ry = (Iy/A)^.5;
end

if J == 0;
   J = 1/3*(bf*tf^3*2 + h*tw^3);
end

if Sx == 0
   y = (tf*2+h)/2;
   Ix = (bf*tf^3*2 + tw*h^3)/12+ bf*tf*(y-tf/2)^2*2;
   Sx = Ix / y;
end

if rts ==0
   Iy = tf*bf^3/6;
   rts = (Iy*h0/2/Sx)^.5;
end

if Z == 0
   y = (tf*2+h)/2;
   Z = tf*bf*(y-tf/2)*2 + h/2*tw*h/4*2;
end

Mp = Z * Fy;
```

```
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
Lp = 1.76*ry*(E/Fy)^.5;
Lr = 1.95*rts*E/(0.7*Fy)* (J/(Sx*h0)+((J/(Sx*h0))^2+6.76*(0.7*Fy/E)^2)^.5)^.5;

if Lb <= Lp
   disp('case 1');
   Mn = Mp;
elseif Lb <= Lr
   Mn = Cb*(Mp - (Mp-0.7*Fy*Sx)*((Lb-Lp)/(Lr-Lp)))
   if Mn >= Mp
      Mn = Mp;
   end
   disp('case 2');
else
   Mn = Cb * Sx*pi^2*E/(Lb/rts)^2* (1+0.078*J/Sx/h0*(Lb/rts)^2)^.5
   if Mn >= Mp
      Mn = Mp;
   end
   disp('case 3');
end

disp('Mn in feet');
Mn = Mn /12
Mu = 0.9 * Mn
```

Lateral Buckling

```
clear all; clc
% require 7371
Lb = 85*12;
E = 29000; Fy = 50;
A = 376; tf = 2; bf = 24; tw = 4; d = 78; Cb = 1.14;
h = d - 2*tf;
ry = 0; %% if do not know, ry = 0
rts = 0;    %% if do not know, rts = 0
J = 0;  %% if do not know, J = 0
Sx = 0; %% if do not know, Sx = 0
Z = 0;

h0 = d-tf;

if ry==0
```

```
    Iy = tf*bf^3/6;
    ry = (Iy/A)^.5;
end

if J == 0;
    J = 1/3*(bf*tf^3*2 + h*tw^3);
end

if Sx == 0
    y = (tf*2+h)/2;
    Ix = (bf*tf^3*2 + tw*h^3)/12+ bf*tf*(y-tf/2)^2*2;
    Sx = Ix / y;
end

if rts ==0
    Iy = tf*bf^3/6;
    rts = (Iy*h0/2/Sx)^.5;
end

if Z == 0
    y = (tf*2+h)/2;
    Z = tf*bf*(y-tf/2)*2 + h/2*tw*h/4*2;
end

Mp = Z * Fy;

%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
Lp = 1.76*ry*(E/Fy)^.5;
Lr = 1.95*rts*E/(0.7*Fy)* (J/(Sx*h0)+((J/(Sx*h0))^2+6.76*(0.7*Fy/E)^2)^.5)^.5;

if Lb <= Lp
    disp('case 1');
    Mn = Mp;
elseif Lb <= Lr
    Mn = Cb*(Mp - (Mp-0.7*Fy*Sx)*((Lb-Lp)/(Lr-Lp)))
    if Mn >= Mp
        Mn = Mp;
    end
    disp('case 2');
else
    Mn = Cb * Sx*pi^2*E/(Lb/rts)^2* (1+0.078*J/Sx/h0*(Lb/rts)^2)^.5
    if Mn >= Mp
        Mn = Mp;
```

```
    end
    disp('case 3');
end

disp('Mn in feet');
Mn = Mn /12
Mu = 0.9 * Mn
```

<u>Local Buckling Check</u>

```
bf = 11.6; tf = 1.91;
z = 490; Fy = 50; Sx = 419;  h = 15.5; tw = 1.06; E = 29000;

lam_pf = 0.38*(E/Fy)^0.5; lam_rf = (E/Fy)^0.5;

Mp = z* Fy
lamda = bf/2/tf;

if lamda <= lam_pf
    Mn = Mp / 12
elseif lamda <= lam_rf
    Mn = Mp - (Mp - 0.7*Fy*Sx)*((lamda-lam_pf)/(lam_rf-lam_pf)) / 12
else
    kc = 4 / (h/tw)^0.5;
    if kc < 0.35
        kc = 0.35;
    elseif kc> 0.76
        kc = 0.76;
    end
    Mn = 0.9*E*kc*Sx / lamda^2 / 12
end
```

Overall Buckling

```
clear all; clc;

Kx = [1.0];
Ky = [1.0];
L = [89*12];
rx = 12; ry = 12;
E = 3600; Fy = 6; Ag = 12.57*144;

KL_r = max([Kx.*L/rx; Ky.*L/ry]);
```

```
Fe = pi^2*E./(KL_r).^2;

for i = 1:numel(KL_r)
   if KL_r(i) <= 4.71*(E/Fy)^.5
      Fcr(i) = 0.658^(Fy/Fe(i)) * Fy;
   else
      Fcr(i) = .877*Fe(i);
   end
end

Pu = 0.9 * Fcr * Ag
```

<u>Buckling Check</u>

```
clear all; clc;


Kx = [1.2 2 2 2];
Ky = [.8 1 1 1];
L = [144 17*12+6 14*12+6 144];
rx = 5.51; ry = 3.13;
E = 29000; Fy = 50; Ag = 35.2;

KL_r = max([Kx.*L/rx; Ky.*L/ry])
Fe = pi^2*E./(KL_r).^2;

for i = 1:numel(KL_r)
   if KL_r(i) <= 4.71*(E/Fy)^.5
      Fcr(i) = 0.658^(Fy/Fe(i)) * Fy;
   else
      Fcr(i) = .877*Fe(i);
   end
end
```

<u>Box girder flexural iterative design tool</u>

```
Pu = 0.9 * Fcr * Ag
 E=29000; %Specify Elastic Modulus
Fy=50; %Specify Fy
d=40;%specify depth in inches
tf=2.5;%specify flange thickness in inches
bf=40;%specify breadth of flange in inches
tw=2.5;%specify thickness of web in inches
h=d-tf*2;
d1=d-tf/2;
```

```
d2=bf/2;
A1=2*bf*tf;
A2=4*(h/2)*tw;
%Check Local buckling of Compression Flange
lambda=bf/tf;
lambdapf=1.12*sqrt((E/Fy));
lambdarf=1.4*sqrt(E/Fy);

% Variables
Ixx=2*bf*(tf^3)/12 +bf*tf*(bf/2-tf)+2*tw*(h^3)/12 +h*tw*(h/2-tw); %Check the d with yanlong for Ad^2
beff=1.92*tf*sqrt(E/Fy)*(1-(0.38/(bf/tf))*sqrt(E/Fy));
Seff=Ixx/beff;
Zx=0.25*(bf*((h+2*tw)^2)-(bf-2*tf)*h^2); %Z outer rectangle- Z inner rectangle
Mp=Fy*Zx; %Mp=Fy*Zx
Sx=Ixx/(h/2-tf);
%Check for local buckling
if lambda<lambdapf
   result='Section is Compact, No Local Buckling'
   Mn=Mp;
elseif lambda>lambdapf && lambda<lambdarf
   result='Inelastic Flange Local Buckling'
   Mn=Mp-(Mp-Fy*Sx)*(3.57*bf/tf)*(sqrt(Fy/E)-4.0);
else
   result='Inelastic Flange Local Buckling'
   Mn=Fy*Seff;
end
Mn=num2str(0.9*Mn/12); %Mu in Kip foot, multiplied by safety factor 0.9
Mu=['Mu is ',Mn,' kipfoot']
```

Shear Checks

```
I_roll = 0; %% 1 if rolled I shape, 0  others
stiffen = 0; %% 0 if unstiffened, other 1
E = 29000; Fy = 50;
Cv = 0; %% initial
h = 26.5; tw = 1.36; bf = 15.6; tf = 2.44; %a = 25; % spacing

if h/tw <= 2.24*(E/Fy)^.5
   phi = 1;
   Cv = 1;
else
   phi = 0.9;
end
```

```
if h/tw < 260 && stiffen == 0
   kv = 5;
else
   kv = 5 + 5/(a/h)^2;
   if a/h >3 || a/h > (260/(h/tw))^2
      kv = 5;
   end
end

if Cv == 1 || h/tw <= 1.1*(kv*E/Fy)^.5
   Cv = 1;
elseif h/tw <= 1.37 * (kv*E/Fy)^.5
   Cv = 1.1*(kv*E/Fy)^.5 / (h/tw);
else
   Cv = 1.51*E*kv / ((h/tw)^2*Fy);
end

Aw = (h + 2*tf)*tw;
Vn = 0.6*Fy*Aw*Cv;
Vu = phi*Vn

%% check stiffened
J = 2.5 / (a/h)^2 -2;
Iww = a * tw^3* J
```