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Seismic Design of a Prestressed Concrete Bridge

Alperen Ozel
aozel@uno.edu

Seismic Design of a Prestressed Concrete Bridge

A Thesis

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Abstract

Latest advancements in software have become a crucial part of engineering, more so, an indispensable tool in structural analysis. The main goal of this thesis is to examine dynamic responses of bridges in seismic activity with the help of such tool as CSi Bridge. Therefore, throughout this study, a prestressed concrete bridge model will be thoroughly designed.

In the first section, the required materials and structural components will be introduced. The following section (2) will cover calculations required for modeling. The actual modeling of the structure will be carried out in Section 3. In Section 4, the set of required analysis for seismic design such as modal, pushover and response spectrum will be conducted. Lastly, in Section 5, analysis results from Section 4 will be evaluated. Thus, by examining the entire model, its construction, materials used, provided properties and conditions such as location on earth, seismic magnitude, it will be determined whether the design is sufficient and acceptable or not.

Keywords: bridges, seismic design, prestressed concrete, structural analysis, dynamic responses, earthquake, 3D modeling, bridge model

1. INTRODUCTION

In this section, the proposed bridge model that is considered for seismic design will be presented with its material properties and structural components.

Design of a prestressed concrete bridge:

1.1. Material Properties

Concrete Slab (t_s= 8.0 in.)

- Concrete Strength at 28-days, fc' = 4,000 psi (4 ksi)
- Unit weight of concrete = 150 pcf
- Modulus of Elasticity, Ec (ksi) = 3605 ksi

Asphalt wearing surface, t_w = 1.5 in. (including any future wearing surfaces)

- Unit weight of asphalt wearing surface = 140 pcf

Precast beams: AASHTO Type- IV

- Concrete Strength at 28 days, f'c = 5000 psi, High Strength Concrete - Normal Weight
- Concrete Strength at transfer, f'ci = 4000 psi (Assumed the 80 percent of the cylinder strength, f'ci = 0.8xf'c)
- Concrete unit weight = 150 pcf
- Modulus of Elasticity, Ec (ksi) = 57,000 x $\sqrt{5000}$ / 1000 = 4031 ksi
- Poisson's Ratio (U) = 0.2

Prestressing Strands (½ in. diameter - seven wire low relaxation)

- Area of one strand = 0.153 in.2
- Ultimate Stress, fpu = 270,000 psi
- Yield Strength, fpy = 0.9 fpu = 243,000 psi (LRFD)

Stress limits for prestressing strands: (LRFD)

fpi ≤ 0.75 fpu = 202,500 psi

fpe ≤ 0.80 fpy = 194,400 psi

Modulus of Elasticity, Ep = 28,500 ksi (LRFD)

Bents (6000 psi concrete)

- Concrete Strength at 28-days, fc' = 6,000 psi (6 ksi)
- Unit weight of concrete = 150 pcf
- Modulus of Elasticity, Ec = 4415 ksi

Reinforcing Steel (A615 - Grade 60)

- Minimum Yield Strength, fy = 60,000 psi
- Modulus of Elasticity, Es = 29,000 ksi (ASSHTO)
- Poisson : 0.3

1.2. General Information of Bridge

1.2.1. Roadway - Traffic Lanes and Shoulders

Below are the width-standards for various types of roadways according to Interstate Highways Standards of United States (defined by ASSHTO) and Federal Highway Administration. Based on these standards the total width of the bridge roadway is calculated to be 384 in., including two 12-foot lanes and two 4-foot shoulder lanes on either side.

Type of Roadway	Rural		Urban	
	US (feet)	Metric (meters)	US (feet)	Metric (meters)
Freeway	12	3.6	12	3.6
Ramps (1-lane)	12-30	3.6-9.2	12-30	3.6-9.2
Arterial	11-12	3.3-3.6	10-12	3.0-3.6
Collector	10-12	3.0-3.6	10-12	3.0-3.6
Local	9-12	2.7-3.6	9-12	2.7-3.6

Table 1: Width standards for interstate highways

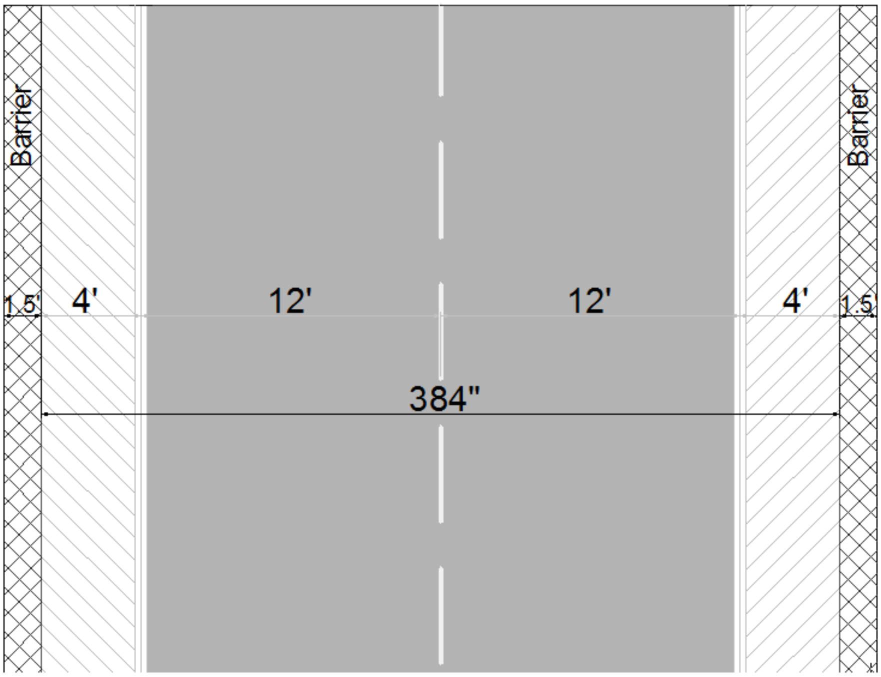

Figure 1: Bridge plan

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1.2.2. Bridge Profile

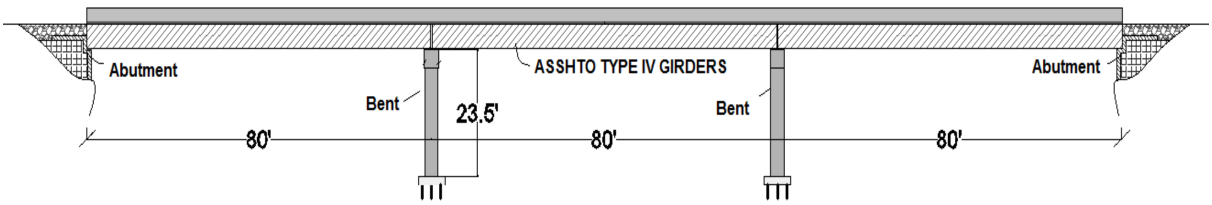

Figure 2: Bridge profile

- *Total Length* : 240 feet
- *Total Width*: 35 feet
- *Total Height* : 29 feet

1.2.3. Structural Components

1.2.3.1. Bridge Superstructure and Deck

The bridge being studied has three 80-foot spans (c/c distance) with no skew and a total width of 35 feet. Its superstructure consists of six AASHTO Type IV Precast Beams that are

spaced 6 feet center to center - designed to act compositely with 8-inch thick cast-in-place concrete deck. Lastly, the asphalt wearing surface has a thickness of 1.5 inches. (Figure 3)

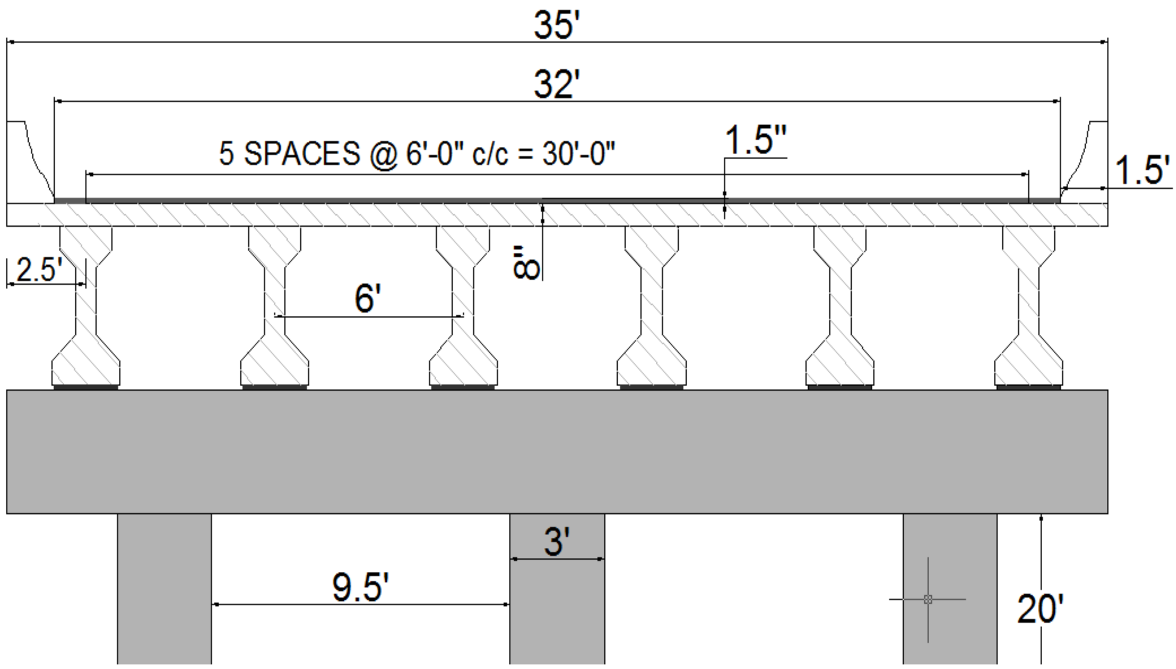

Figure 3: Bridge cross section

1.2.3.2. Substructure - Bents and Abutments

Vertical substructure elements such as bents and abutments are used to support superstructures and transfer loads down to earth through the foundation. In this bridge model, the superstructure is supported by two interior bents, spaced 80 feet (center to center) and by two abutments placed at the each end of the bridge (Figure 2). Each interior bent consists of a bent cap and three 36-inch diameter columns with a height of 20 feet. The foundation of the bridge has a 4-foot thick pile caps and 14-inch diameter concrete piles with steel casing (Figure 4).

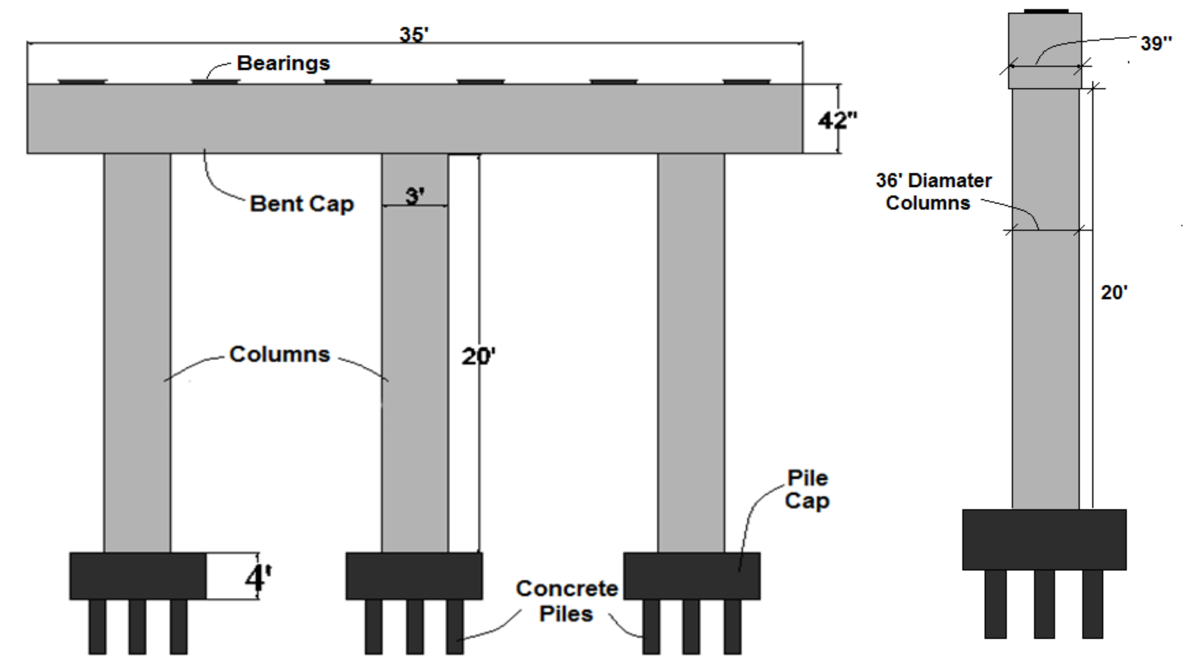

Figure 4: Details of interior bents (YZ and XZ view)

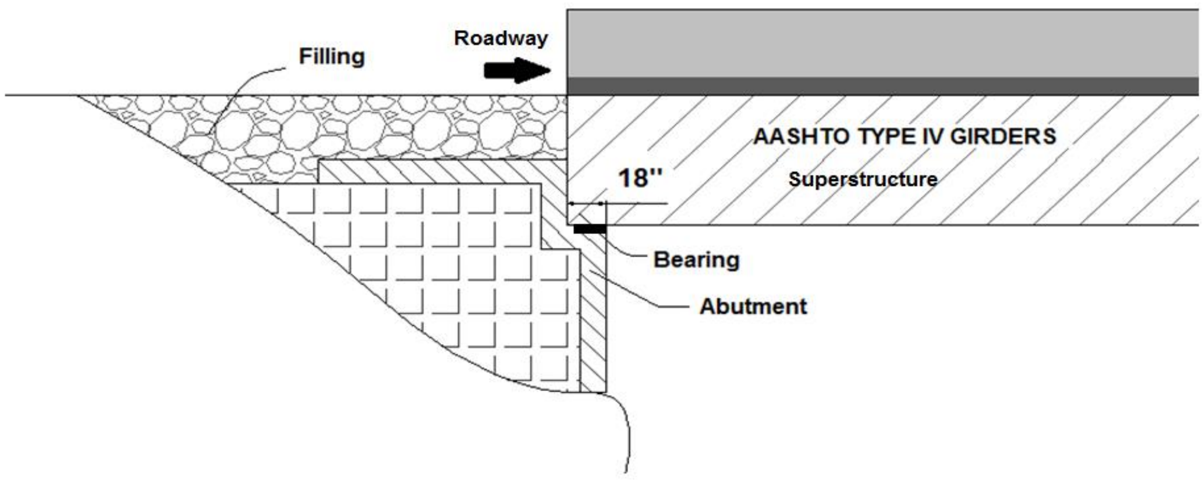

Figure 5: Details of abutments

1.2.3.3. Bridge Bearings

Consisting of two different types, there are total of 24 bearings placed between the girders and the bent caps (also abutments) – specifically located underneath the girders. The bridge bearings are shown in the following diagrams (Figure 6, and Figure 7), however further information about the bearings will be discussed in Section 3.4.6.1.

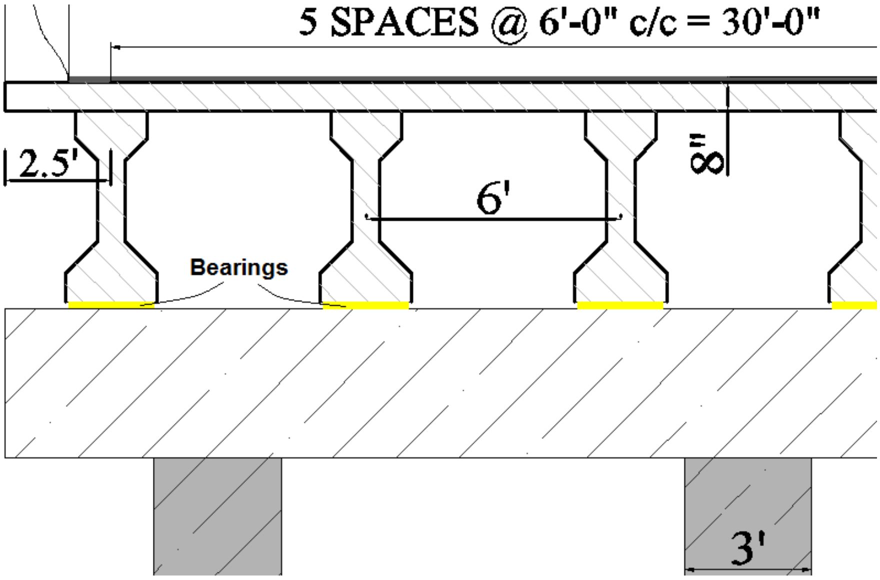

Figure 6: Bent bearings and girders (YZ view)

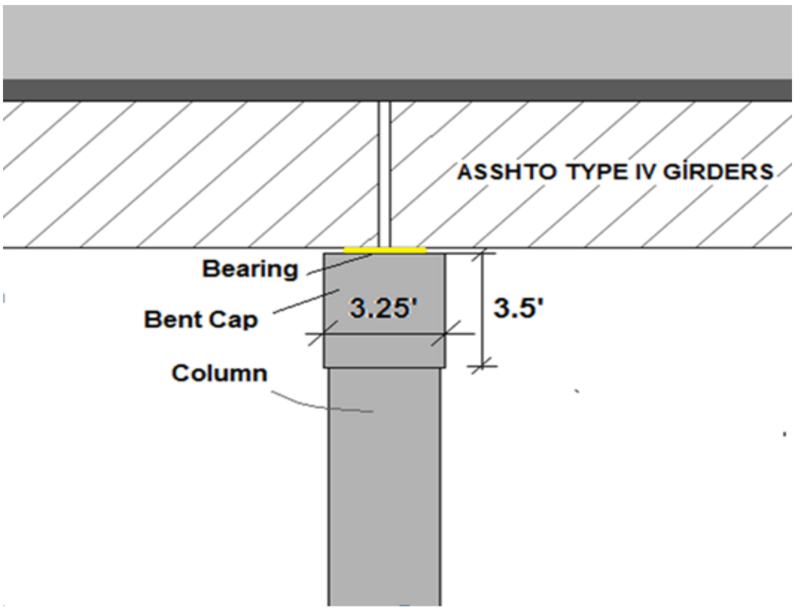

Figure 7: Bent bearings and girders (XY View)

2. PRELIMINARY CALCULATIONS AND CONTROLS BEFORE 3D MODELING

Today's structural analysis software programs like CSi Bridge can automatically calculate most of the loads that affect any structure. Nonetheless, before modeling, dead and live loads are determined manually beforehand in order to obtain prestressing force and number of strands. In this section, these required preliminary data which is used during modeling will be calculated based on AASHTO LRFD Bridge Design Specifications (AASHTO, 2012) and PCI Bridge Design Manuel, 3rd Edition.

The first step of prestress calculations is to simply check the girder depth and spacing that were previously assumed, and calculate the coefficients used to distribute dead and live loads to each beam - also known as distribution factors. With the help of these coefficients, bending moments and shear forces at critical sections due to only dead and live loads will be determined.

Before bending moments and shear forces are determined, for the sake of calculation simplicity, it is assumed that the bridge superstructure consists of three groups of simply supported beams as shown in Figure 8. Therefore, taking only one interior beam in the calculations will be sufficient because of their constant span length.

In the 3D Seismic Design/Analysis (Section 3 - and 4), a more complex model which consists of superstructure connecting to its substructure with the help of bearings will be examined instead of the simplified structure.

Series of calculations mentioned above are demonstrated in this section:

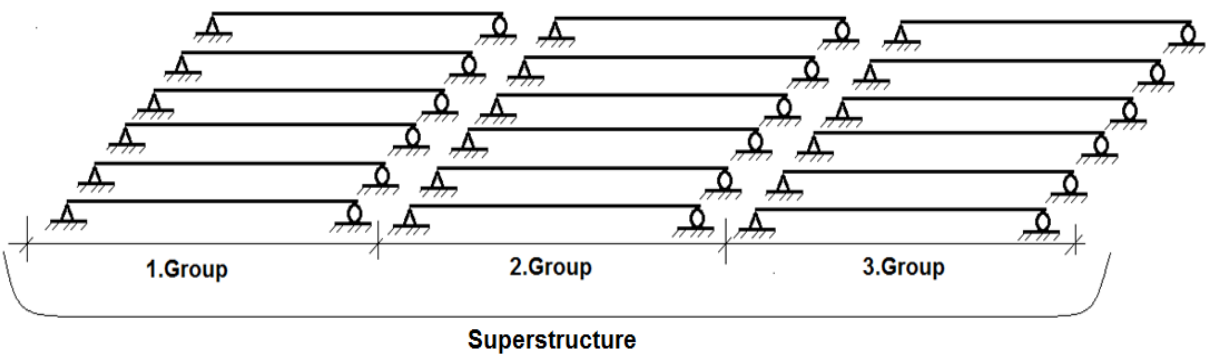

Figure 8: Simplified superstructure groups

2.1. Determining Girder Depth, Girder Spacing and Deck Thickness

The depth and spacing of girders will be determined based on CDOT Bridge Design Manual 2015, as demonstrated below:

Girder Depth:

- The Span length, L = 80ft.

Assuming:

$$\frac{\text{Structure Depth, Ds}}{\text{Span Length}} = 0.055$$

The minimum depth, Ds = 0.055 (80) = 4.4ft.

Because deck thickness is calculated based on girder spacing, and girder spacing is calculated based on structure depth, the concrete slab thickness must be initially assumed.
(8-inch slab previously assumed.)

Therefore, the minimum girder height must be: Ds = 4.4 (12) – 8 = 44.5in.

ASSHTO Type IV girder which has a height of 54in. is sufficient hence selected.

The structure depth, Ds = 54 + 8 = 62in. (5.16 ft.)

$$\frac{\text{Ds}}{\text{L}} = \frac{5.16}{80} = 0.065 > 0.055 \quad (OK)$$

The center-center girder spacing is determined as follows:

Maximum girder spacing, S = 1.5xDs = 1.5 (5.16ft.) = 7.74ft.

Total bridge width = 35ft.

Try girder spacing, S = 6ft.

$$\text{Overhang Length} = \frac{35 - 6\ (5\ \text{Spacing})}{2\ \text{Overhangs}} = \frac{5}{2} = 2.5\text{ft.}$$

According to MTD 10-20, Attachment 1 (Caltrans, 2013), overhangs should be less than half the girder spacing (S/2 = 3), (6ft. maximum)

Overhang length = 2.5 ft. (OK)

Therefore, 6 feet girder spacing (OK)

Slab thickness:

From Deck Slab Thickness and Reinforcement Schedule (MTD, Table 10-20.1-b), for girders that have centerline-to-centerline spacing of 6 feet, the required slab thickness should be minimum 7 inches. Therefore, a slab thickness of 8 inches would be sufficient for the bridge.

2.2. Design Span & Composite Section Effective Flange Width

Design Span and Overall Beam Length:

The design span and overall beam length is calculated as demonstrated below. (Dimension in Figure 9 and 10 should be used in the following calculations.)

- Span length (c/c Piers) = 80'-0)"
- Overall beam length = 80' – 2(2") = 79'8"
- Design span, Lc = 80' – 2(8") = 78'-8" = 78.6' (c/c of bearing)

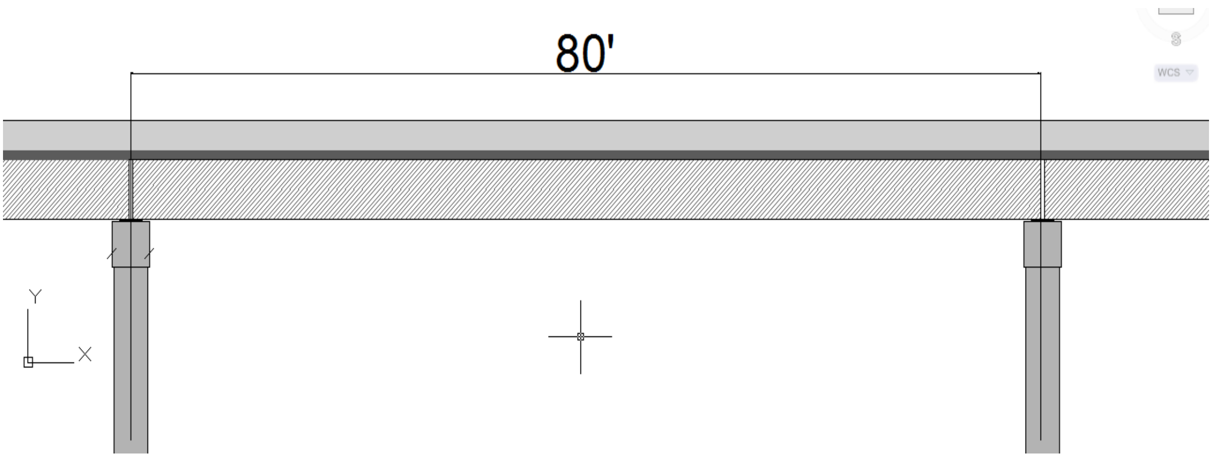

Figure 9: Side view & span length

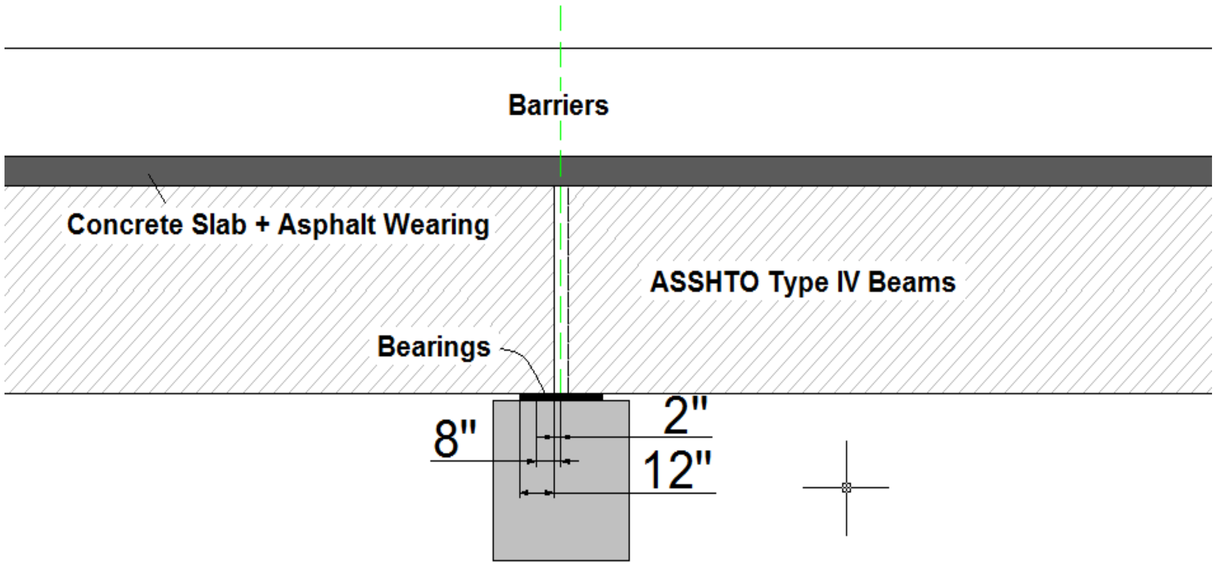

Figure 10: Details of girders - bent bearing

<u>Effective Flange Width:</u>

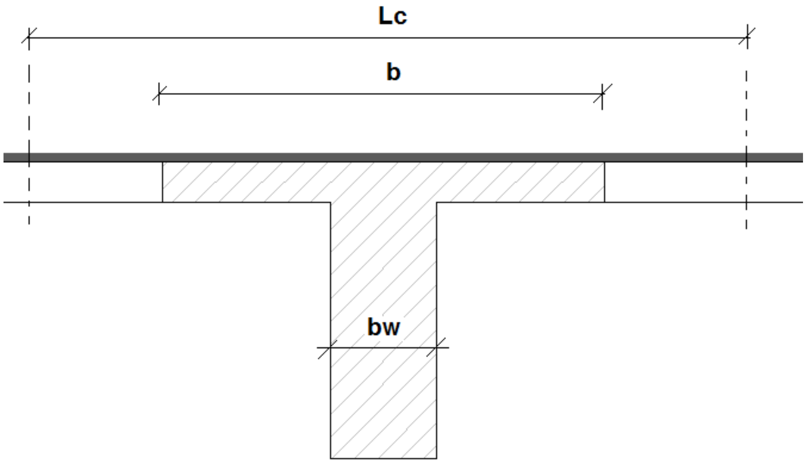

Figure 11: Effective flange width of composite section

- bw = greater of web thickness or ½ beam top flange width
- Lc = Distance center to center of beams , and b= Effective flange width

	End Beam	Intermediate Beam
ACI	bw + 6hf	bw + 6hf
	bw + 0.5Lc	bw + Lc
	bw + (Lc/12)	Lc/4
ASSHTO	bw + 6hf	bw + 12hf
	bw + 0.5Lc	bw + Lc
	bw + (Lc/12)	Lc/4

Table 2: ASSHTO requirements for effective flange width

Based on ASSHTO requirements shown in Table 2, effective flange width (b) is lesser of:

- 1/4 span length: $\frac{78.6\ x\ (12\ in./ft)}{4}$ = 235.8 in.
- Distance center to center of beams: 6 x (12 in./ft.) = 72.00 in. (controls)
- 12 (effective slab thickness) + greater of web thickness or ½ beam top flange width: 12x (8.0) + 1/2(20.0) = 106.00 in.,

Therefore, the effective flange width (b) = 72 inches (6 feet)

2.3. Composite Section Properties

$$n = \frac{Ec\ (for\ slab)}{Ec\ (for\ beam)} = \frac{57{,}000\sqrt{4000}}{57{,}000\sqrt{5000}} = 0.9$$

According to LRFD Bridge Design Specifications the value of n can be considered to be 1. Therefore,

Effective flange width = Modified flange width = 72 in.

$$Cb = \frac{(72x9.5)(58.75)+(789x24.73)}{(9.5x72)+789} = 40.53 \text{ in.}$$

$$Ic' = 260{,}741 + 789(40.53-24.73)^2 + \frac{72(9.5)^2}{12} + 72 \times 9.5(22.97)^2 = 819{,}141 \text{ in.}^4$$

$r^2 = I/A = 819{,}141 / [789 + (72 \times 9.5)] = 556.1$ in.^{2}

$S_{cb} = I/c_b = 819{,}141/40.53 = 20{,}210.73$ in.^{3}

(composite section modulus for extreme bottom fiber of the precast beam)

$S_c^t = I/c_t = 819{,}141/22.97 = 35{,}661.34$ in.^{3}

(composite section modulus for top fiber of the slab)

	I	Area in.^{2}	y in.	A x y^2 in.^{3}	I + Ay^2
Beam	260,741	789	15.8	196,965.96	457,707
Slab	541.5	684	22.97	360,892.70	361,434.2
Σ		1,473			819,141

Table 3: Properties of composite section

From Table 3:

- Total area of composite section = 1,473 in.^{2}
- Moment of inertia of composite section = 819,141 in.^{4}

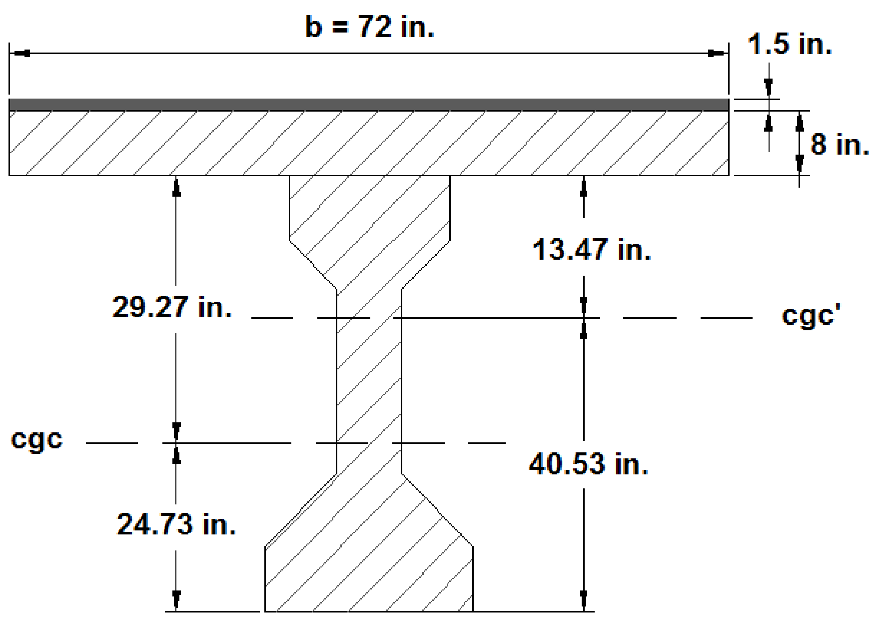

Figure 12: Composite section

2.4. Shear Forces and Bending Moments due to Dead Loads

2.4.1. Dead Loads

According to AASHTO LRFD Bridge Design Specifications, the weight of the beam and the weight of the slab act together on the non-composite simple span structure. Whereas, the weight of the barriers, asphalt wearing, and live load act together on the composite simple span structure.

DC = Dead load of structural components

Dead loads acting on the non-composite structure:

- Self-Weight of the ASSHTO Type IV Beam = 0.821 kip/ft.
- Weight of slab on each beam (8") = (0.150 pcf) x (6ft) x (8 in.) / 12 in/ft. = 0.600 kip/ft.

Total DC = 0.821 + 0.600 = 1.421 kips/ft./beam

DW = Super Imposed Dead Load on the composite structure:

According to LRFD Specifications (AASHTO, 2012), permanent dead loads including concrete barriers and wearing surface, may be distributed uniformly among all girders, provided all of the following conditions are met: [LRFD Specifications, Art. 4.6.2.2.1]

1. There are at least four beams supporting the concrete deck.
   (6 AASHTO TYPE IV), (OK)
2. Beams are parallel and have approximately the same stiffness (EI). (OK)
3. The roadway part of the overhang, de (distance between the inside face of the curb and the centerline of the exterior web of the exterior girder), does not exceed 3.0 ft.
   (2.5 -1.417 = 1.083ft.), (OK)
4. Curvature in plan is less than the limit specified in LRFD Art. 4.6.12. (0^0), (OK)

Since all the criteria are satisfied, the barrier and wearing surface loads are equally distributed among the 6 beams.

- Weight of T501 Rails of Barriers on each beam
  = 2 x [(326plf / 1000) / 6 beams] = 0.11kips/ft./beam
- Weight of 1.5” Wearing surface = (0.140 kcf) x [1.5 in. / (12in./ft)] = 0.0175 kips/ft^2.
- Weight of wearing surface on each beam = 0.0175 ksf x w_l / 6 beams,

Where, (w_l) is the clear roadway width between the curbs. Since w_l = 384in. = 32ft. (Figure 1) therefore, 0.0175 ksf x 32ft / 6 beams = 0.093 kips/ft./beam

Total Super Imposed Dead Load = 0.110 + 0.093 = 0.203 kip/ft./beam

Total Dead Load = 1.421 + 0.203 = 1.624 kips/ft./beam

2.4.2. Unfactored Shear Forces and Bending Moments

The bending moment (M) and shear force (V) that occur due to uniformly distributed dead load at any distance (x) are calculated by using the following formulas:

$$M = 0.5wx(L - x)$$, *[Mmax at mid-span, x=39.3 ft.]*

$$V = w(0.5L - x)$$, *[Vmax at support, x=0]*

(w =1.624 kips/ft./beam, L = span length = 78.6 ft.)

Unfactored shear forces and bending moments which were calculated for the support and mid-span are shown in Table 4.

Distance (x)	Dead Loads				Super Imposed				Total	
	Beam		Slab		Barrier		Wearing			
	V	M	V	M	V	M	V	M	V	M
ft	kips	k-ft	kips	k-ft	kips	k-ft	kips	k-ft	kips	k-ft
0	32.26	0	23.58	0	4.32	0	3.65	0	63.81	0
19.65	16.13	475.5	11.79	347.51	2.16	63.71	1.83	53.86	31.91	940.58
39.3	0	634	0	463.35	0	84.95	0	71.82	0	1254.12

Table 4: Unfactored shear forces and bending moments due to DC and DW

2.5. Shear Forces and Bending Moments due to Live Loads

2.5.1. Vehicular Live Loads

Vehicular live load consists of two types of groups - design vehicular live load (HL-93) and permit vehicles in other word, P loads.

2.5.1.1. HL-93 Design Live Load

The AASHTO HL-93 (Highway Loading adopted in 1993) load includes variations and combinations of truck, tandem, and lane loading:

- Design truck or design tandem with dynamic allowance (AASHTO Art. 3.6.1.2.1)
- Design lane load of 0.64 kip/feet without dynamic load allowance (IM) (AASHTO Art. 3.6.1.2.4)

1- Design Truck:

The design truck is the same as HS20 design truck specified by the Standard Specifications as shown below. (Figure 13)

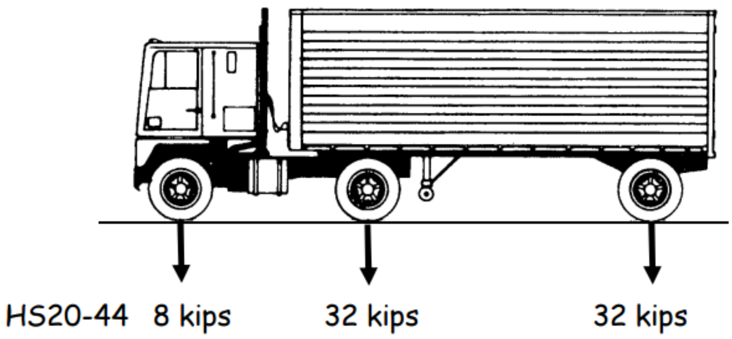

Figure 13: HS20 Design truck

AASHTO has a series of specifications for trucks. For two-axial trucks AASHTO designates these vehicles as "H series trucks". For example, a H15-44 is a 15–ton truck as report in the 1944 specifications. Trucks that pull trailers are designated as HS, for example HS 20-44 (a 20–ton semi-trailer truck).

In general, a truck loading depends on the type of bridge, its location, and the type of traffic anticipated.

2- Design Tandem: The design tandem consists of a pair of 25.0-kip axles spaced at 4.0 feet apart with the transverse spacing of the wheels equal to 6.0 feet. The design tandem generally controls for short spans (L < 40 feet), which does not apply to this bridge design. (L=80 feet)

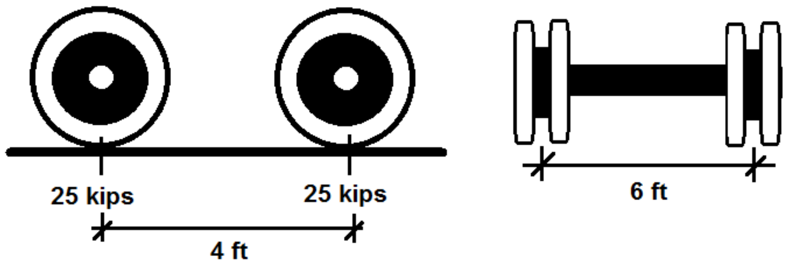

Figure 14: Design tandem

3- Design Lane Load:

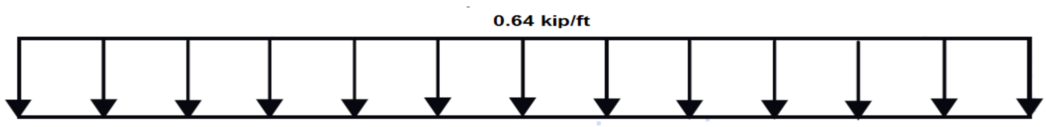

Figure 15: Design lane load (uniformly distributed load)

When loading the superstructure with HL-93 loads, only one vehicle per lane is allowed on the bridge at any one time, except for Cases 3 and 4 (Figure 18 and Figure 19). Trucks should be placed transversely in as many lanes as practical.

The following 4 cases represent, in general, the requirements for HL-93 loads as shown below. Cases 1 and 2 are for positive moments, and Cases 3 and 4 are for negative moments and bent reactions only.

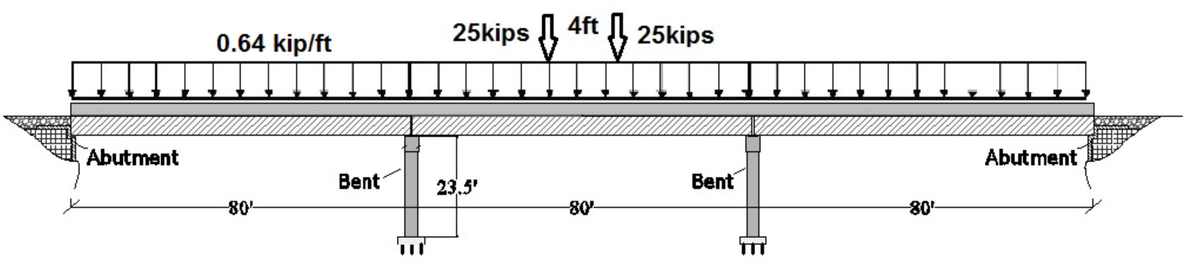

Figure 16: Case 1: tandem + lane

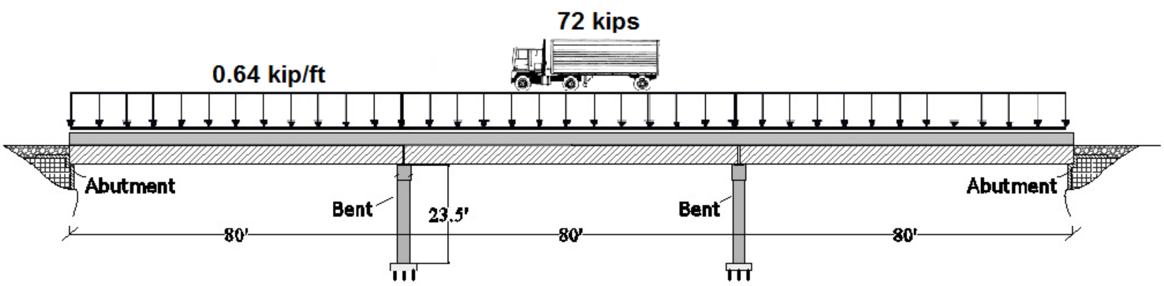

Figure 17: Case 2: truck + lane

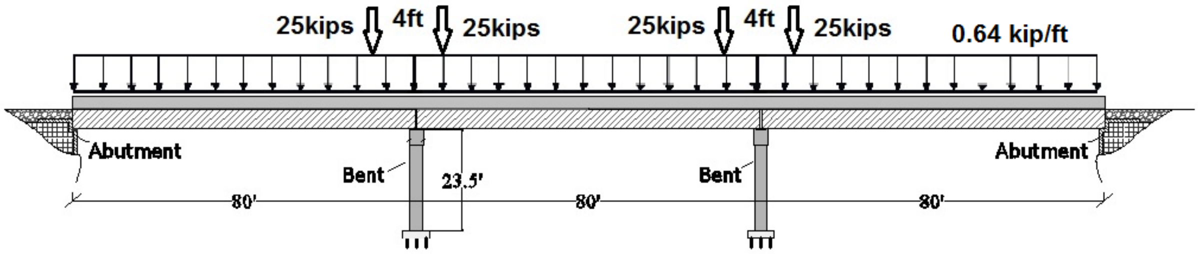

Figure 18: Case 3: two tandem + lane

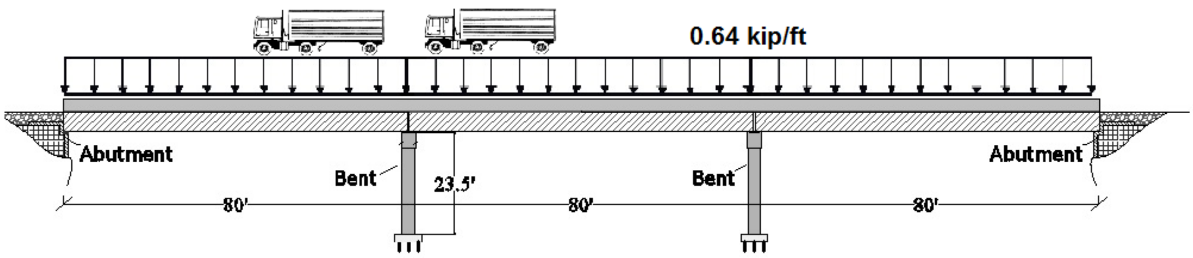

Figure 19: Case 4: two truck + lane

2.5.2. Pedestrian Live Load, PL

Since there is no sidewalk on the roadway, pedestrian live load will not be considered in the model.

Also the bridge is not a structure with tall single column bents over 30 feet, therefore wind load on structure (WS) and wind on live load (WL) will not be considered as well. [Bridge Design Practice – Caltrans, Section 3.4)

2.5.3. Distribution Factor for Bending Moment

According to LRFD Specifications, the bending moments and shear forces due to live load are determined using simplified distribution factor formulas, if the following conditions are met: (LRFD Art. 4.6.2.2)

1. The bridge deck has constant width. (OK)
2. There are at least four beams supporting the concrete deck.

(6 AASHTO TYPE IV – OK)

3. Beams are parallel and have approximately the same stiffness (EI). (OK)
4. The roadway part of the overhang, de (distance between the inside face of the curb and the centerline of the exterior web of the exterior girder), does not exceed 3.0 ft. (2.5-1.417 = 1.083 ft.) (OK)
5. Curvature in plan is less than the limit specified in LRFD Art. 4.6.12. (0^0) (OK)
6. (For precast concrete I-beams with cast-in-place concrete, the bridge type is chosen as ‘’k’’.)

Since the required conditions are met, for all limit states (except fatigue) the distribution factor for bending moment is calculated by using LRFD Table 4.6.2.2.2b-1 (Table 5)

Type of Beams	Applicable Cross-Section from Table 4.6.2.2.1-1	Distribution Factors	Range of Applicability
Concrete Deck, Filled Grid, Partially Filled Grid, or Unfilled Grid Deck Composite with Reinforced Concrete Slab on Stee or Concrete Beams; Concrete T-Beams, T- and Double T-Sections	a, e, k and also i, j if sufficiently connected to act as a unit	One Design Lane Loaded: $0.06+\left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12.0Lt_s^3}\right)^{0.1}$ Two or More Design Lanes Loaded: $0.075+\left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12.0\,Lt_s^3}\right)^{0.1}$	$3.5 \le S \le 16.0$ $20 \le L \le 240$ $4.5 \le t_s \le 12.0$ $N_b \ge 4$ $10{,}000 \le K_g \le 7{,}000{,}000$
		use lesser of the values obtained from the equation above with $N_b = 3$ or the lever rule	$N_b = 3$

Table 5: Distribution of live load per lane for moment - interior beams

- *For two or more lanes loaded:*

DFM = $0.075 + (\frac{S}{9.5})^{0.6} (\frac{S}{L})^{0.2} (\frac{Kg}{12\ Lts^3})^{0.1}$ [LRFD Table 4.6.2.2.2b-1]

Provided that:	3.5 ≤ S ≤ 16	S = 6.0 ft. (O.K.)
	4.5≤ ts ≤12	ts = 8.0 in (O.K.)
	20≤ L≤240	L = 78.6 ft. (O.K.)
	Nb ≥ 4	Nb = 6 (O.K.)
	10,000 < Kg < 7,000,000	(O.K.) – as shown below

Where,

DFM = distribution factor for moment for interior beam

S = spacing of beams = 6.0 ft.

L = beam span = 78.6 ft.

ts = depth of concrete deck = 8.0 in.

Kg = longitudinal stiffness parameter, in.4

Kg = n (I + Aeg^2) [LRFD Art. 3.6.1.1.1]

Where,

n= $\frac{Ec\ (for\ slab)}{Ec\ (for\ beam)} = 1$

The modular ratio is assumed to be 1, and needs to be updated once the optimum fc' value is established. The distribution factor based on new modular ratio will be compared to the present distribution factor and updated if needed.

A = cross-sectional area of the beam (non-composite section)

A = 789 in.2

I = moment of inertia of beam (non-composite section) = 260,741 in.4

eg = distance between centers of gravity of the beam and slab, in.

= (ts/2 + ct) = (8/2 + 29.27) = 33.27 in.

Therefore, Kg = 1[260,741 + 789(33.27)^2] = 1,134,079.5 in.4

DFM = $0.075 + \left(\frac{6}{9.5}\right)^{0.6} \left(\frac{6}{78.6}\right)^{0.2} \left(\frac{1134079.5}{12\,(78.6)8^{3}}\right)^{0.1}$ = 0.569 lanes/beam

- *For one design lane loaded:*

DFM = $0.06 + \left(\frac{S}{14}\right)^{0.4} \left(\frac{S}{L}\right)^{0.3} \left(\frac{Kg}{12\,Lts^{3}}\right)^{0.1}$

DFM = $0.06 + \left(\frac{6}{14}\right)^{0.4} \left(\frac{6}{78.6}\right)^{0.3} \left(\frac{1134079.5}{12\,(78.6)8^{3}}\right)^{0.1} =$ 0.419 lanes/beam

Thus, the case of “two or more lanes loaded” is applied.

DFM = 0.569 lanes/beam

For fatigue limit state:

[LRFD Specifications Art. 3.4.1] states that for fatigue limit state, a single design truck should be used. However, live load distribution factors given in [LRFD Article 4.6.2.2] take into consideration the multiple presence factor (m).

[LRFD Article 3.6.1.1.2] states that the multiple presence factor (m), for one design lane loaded is 1.2. Therefore, the distribution factor for one design lane loaded with the multiple presence factor removed should be used.

The distribution factor for the fatigue limit state is: 0.419/1.2 = 0.349 lanes / beam

2.5.4. Distribution Factor for Shear Forces

Using [LRFD Table 4.6.2.2.3a-1] – Table 6, the distribution factors for shear is calculated as shown below.

Type of Superstructure	Applicable Cross-Section from Table 4.6.2.2.1-1	One Design Lane Loaded	Two or More Design Lanes Loaded	Range of Applicability
Concrete Deck, Filled Grid, Partially Filled Grid, or Unfilled Grid Deck Composite with Reinforced Concrete Slab on Steel or Concrete Beams; Concrete T-Beams, T-and Double T-Sections	a, e, k and also i, j if sufficiently connected to act as a unit	$0.36+\frac{S}{25.0}$	$0.2+\frac{S}{12}-\left(\frac{S}{35}\right)^{2.0}$	$3.5 \le S \le 16.0$ $20 \le L \le 240$ $4.5 \le t_s \le 12.0$ $N_b \ge 4$
		Lever Rule	Lever Rule	$N_b = 3$

Table 6: Distribution of live load per lane for shear - interior beams

- <u>For two or more lanes loaded:</u>

DFV = $0.2 + (\frac{S}{12}) - (\frac{S}{35})^2$ [LRFD Table 4.6.2.2.3a-1]

Provided that:	$3.5 \le S \le 16$	S = 6.0 ft (O.K.)
	$4.5 \le ts \le 12$	ts = 8.0 in (O.K.)
	$20 \le L \le 240$	L = 78.6 ft. (O.K)
	$Nb \ge 4$	Nb = 6 (O.K.)

Where,

DFV = Distribution factor for shear for interior beam

S = Beam spacing = 6 ft.

Therefore, the distribution factor for shear force is:

DFV = $0.2 + (\frac{S}{12}) - (\frac{S}{35})^2$

$= 0.2 + (\frac{6}{12}) - (\frac{6}{35})^2$

= 0.671 lanes/beam

- <u>For one design lane loaded:</u>

DFV = $0.36 + \left(\frac{S}{25.0}\right)$ [LRFD Table 4.6.2.2.3a-1]

= $0.36 + \left(\frac{6}{25.0}\right) = 0.600$ lanes/beam

Thus, the case of two or more lanes loaded controls.

DFV = 0.671 lanes/beam

2.5.5. Dynamic Load Allowance

IM = dynamic load allowance (applied to truck only)

- IM for fatigue = 15% [PCI Bridge Design Manual, Chapter 7]
- IM for all other limit states = 33%

2.5.6. Unfactored Shear Forces and Bending Moments

2.5.6.1. Due to Truck Load, V_{LT} and M_{LT}

Live loads must be placed on the span so as to create the maximum force effects. The following formulas (or with the help of a structural analysis program) calculate the maximum bending moment and shear force per lane at any point on a span due to the design truck loading (the design truck load controls over the design tandem load for spans greater than 40 feet.). [PCI Bridge Design Manual, 2003, Section 8.11]

- <u>For Truck Load with Impact:</u>

Shear Force, Vx = $\frac{72\,[(L-x)-9.33\,]}{L}$ (for 0 < x/L < 0.5)

Moment, Mx = $\frac{72(x)\,[(L-x)-9.33\,]}{L}$ (for 0 < x/L < 0.333)

Moment, Mx = $\frac{72(x)\,[(L-x)-4.67\,]}{L} - 112$ (for 0.333 < x/L < 0.5, x > 14)

- For Fatigue Truck with Impact:

Moment, Mx = $\frac{72(x)\,[(L-x)-18.22\,]}{L}$ (for 0 < x/L < 0.241)

Moment, Mx = $\frac{72(x)\,[(L-x)-11.78\,]}{L} - 112$ (for 0.241 < x/L < 0.5, x > 14)

- For all limit states except fatigue limit state:

Truck load shear forces and bending moments per beam are:

(DFV = 0.671 Section 3.4.3, DFM = 0.757 - Section 3.4.4, and IM = %33 - Section 3.4.5)

V_{LT} = (shear force per lane)(DFV)(1 + IM)

= (shear force per lane)(0.671)(1 + 0.33)

= (shear force per lane)(0.892) kips

M_{LT} = (bending moment per lane)(DFM)(1 + IM)

= (bending moment per lane)(0.569)(1 + 0.33)

= (bending moment per lane)(0.757) ft.-kips

- For fatigue limit state:

The fatigue load is a single design truck which has the same axle weight used in all other limit states, but with a constant spacing of 30.0 ft. between the 32.0-kip axles.

Therefore, the bending moment of the fatigue truck load (Mf) is:

(IM = %15 = 0.15, and DFM = 0.349)

Mf = (bending moment per lane)(DFM)(1 + IM)

= (bending moment per lane)(0.349)(1 + 0.15)

= (bending moment per lane)(0.401) ft.-kips

2.5.6.2. Due to Design Lane Load, V_{LL} and M_{LL}

In order to obtain the maximum shear force at a section located at a distance (x) from the left support - under a uniformly distributed load (0.64 kip/feet), the beam should be loaded by starting from the right support to the point located at x distance as shown below. (Figure 20)

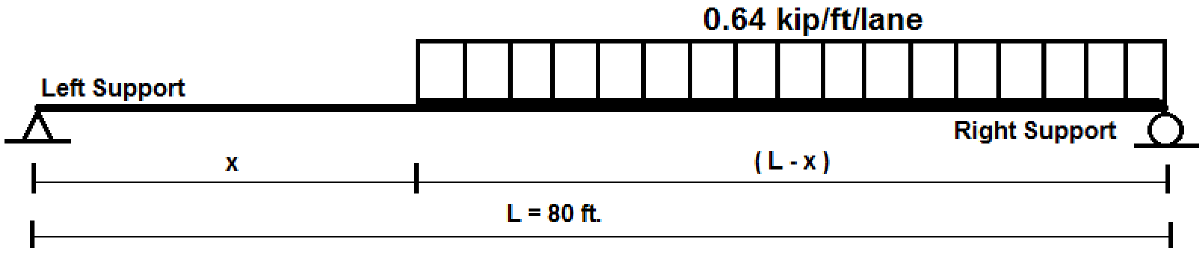

Figure 20: Loaded beam

Therefore, the maximum shear force per lane can be calculated by using the equation below:

$$\textit{Shear Force, } V_x = \frac{w\,(L-x)^2}{2L} \qquad (\text{for } \; x \le 0.5L)$$

Where,

- V_x is in kip/lane,
- L and x are in ft., and
- w=0.64 kip/feet (Design Lane Load – Uniformly Distributed)

Bending moment at any section:

$$\textit{Moment, } M_x = \frac{wx\,(L-x)}{2} \qquad (\text{for } \; x \le 0.5L)$$

Where,

- V_x is in kip/lane,
- L and x are in ft., and
- w=0.64 kip/f

Lane load shear force and bending moment per interior beam are determined as below:

(The dynamic allowance is not applied to the design lane loading)

(DFV = 0.671, DFM = 0.757)

- <u>For all limit states except fatigue limit state</u>

V_{LL} = (lane load shear force)(DFV)

= (lane load shear force)(0.671) kips

M_{LL} = (lane load bending moment)(DFM)

= (lane load bending moment)(0.757) ft.-kips

Unfactored shear forces (V_{LL}, V_{LT}) and bending moments (M_{LL}, M_{LT}) that are calculated at different points along the span are presented in Table 7.

Where,

V_{LL} = Unfactored Shear Force due to Lane Load (max value at support, x = 0)

M_{LL} = Unfactored Moment due to Lane Load (max value at mid-span, x = 39.3 ft.)

V_{LT} = Unfactored Shear Force due to Truck (max value at support, x = 0)

M_{LT} = Unfactored Moment due to Truck (max value at mid-span, x = 39.3 ft.)

Distance (x) (from the support)	x/L	Truck Load with Impact		Lane Load		Fatigue Truck with Impact
		Shear V_{LT}	Moment M_{LT}	Shear V_{LL}	Moment M_{LL}	Moment M_F
ft.		kips	kip-ft.	kips	kip-ft.	kip-ft.
0	0	56.6	0	16.88	0	0
7.86	0.1	50.178	442.15	13.67	134.68	151.63
15.72	0.2	43.755	771.12	10.80	239.44	257.88
23.58	0.3	37.333	986.90	8.27	314.27	262.52
31.44	0.4	30.91	1111.71	6.08	359.17	296.60
39.3	0.5	24.488	1134.68	4.22	374.13	285.25

Table 7: Unfactored shear forces and bending moments due to live loads

2.6. Load Combinations

Total factored load shall be taken as: [LRFD Eq. 3.4.1-1]

$Q = \eta \Sigma \gamma_i q_i$

Where,

- η = a factor relating to ductility, redundancy and operational importance
- (η = 1 in present case) [LRFD Art. 1.3.2]
- γ_i = load factors [LRFD Table 3.4.1-1]
- qi = specified loads

The following limit states provided in [LRFD Table 3.4.1-1] are applicable:

- *Service I:* check compressive stresses in prestressed concrete components:

Q = 1.00(DC + DW) + 1.00(LL+IM) [LRFD Table 3.4.1-1]

This load combination is the general combination for service limit state stress checks and applies to all conditions other than Service III.

- *Service III:* check tensile stresses in prestressed concrete components:

Q = 1.00(DC + DW) + 0.80(LL+IM) [LRFD Table 3.4.1-1]

This load combination is a special combination for service limit state stress checks that applies only to tension in prestressed concrete structures to control cracks.

- *Strength I:* check ultimate strength: [LRFD Table 3.4.1-1 and 2]

Maximum Q = 1.25(DC) + 1.50(DW) + 1.75(LL + IM)
Minimum Q = 0.90(DC) + 0.65(DW) + 1.75(LL + IM)

This load combination is the general load combination for strength limit state design. For simple span bridges, the maximum load factors produce maximum effects. However, minimum load factors are used for dead load (DC), and wearing surface load (DW) when dead load and wearing surface stresses are opposite to those of live load.

- Fatigue: check stress range in strands: [LRFD Table 3.4.1-1]

$Q = 0.75(LL + IM)$

This load combination is a special load combination to check the tensile stress

2.7. Estimating Required Prestress

According to PCI Bridge Design Manuel, the required number of strands is usually governed by concrete tensile stresses at the bottom fiber for Service III at the section of maximum moment. For estimating the number of strands, only the stresses at mid-span are considered.

2.7.1. Service Load Stresses at Mid-span

Bottom tensile stress due to applied dead and live loads using load combination Service III is:

$$f_b = \frac{Mg + Ms}{Sb} + \frac{MSD + (0.8)(MLT + MLL)}{Sbc}$$

Where,

- f_b = Concrete stress at the bottom fiber of the beam
- Mg = Unfactored bending moment due to beam self-weight
- MS = Unfactored bending moment due to slab weight
- MSDL = Unfactored bending moment due to super imposed dead load
- MLT = Bending moment due to truck load plus impact
- MLL = Bending moment due to lane load

Substituting the bending moments (Table 4, and Table 7), and section modulus values (section 2.3), bottom fiber stresses at mid-span is:

$$f_b = \frac{(634 + 463.35)12}{10{,}544} + \frac{(156.77)12 + (0.8)(1134.68 + 374.13)12}{20{,}210.73} =$$

fb = 1.25 + 0.81 = 2.06 ksi

2.7.2. Allowable Stress Limit

At service load conditions, allowable tensile stress is:

- Fb = 0.19 $\sqrt{fc'}$ [LRFD Art. 5.9.4.2b]

Where, fc' = beam concrete strength at service = 5 ksi (Section 1.1)

Fb = 0.19 $\sqrt{5}\ =\ -0.425$ ksi

2.7.3. Required Number of Strands

Required precompressive stress in the bottom fiber after losses is determined as below:

Bottom tensile stress – Allowable tensile stress at final = f_b – F_b

- f_{pb} = 2.06 – 0.425 = 1.635 ksi

Assuming the distance from the center of gravity of strands to the bottom fiber of the beam is equal to y_{bs} = 2 in.

Strand Eccentricity at mid-span:

e_c = y_b – y_{bs} = 24.73 – 2 = 22.73 in.

Bottom fiber stress due to prestress after losses:

$$f_b = \frac{P_{pe}}{A} + \frac{P_{pe}\ e_c}{Sb}$$

Where,

Ppe = effective prestressing force after all losses (will be calculated)

A = Are of the Beam = 789 in.^{2}

S_b = Bottom-Section Modules = 10,544 in^3

$$2.06 = \frac{P_{pe}}{789} + \frac{P_{pe}\,22.73}{10{,}544}$$

Solving for Ppe, Ppe = 601.78 kips

Assuming final losses = 20% of fpi (fpi = 202.5 ksi - Section 1.1)
Assumed final losses = 0.2(202.5) = 40.5 ksi

The prestress force per strand after losses:

= (cross sectional area of one strand) [fpi – losses]
= 0.153(202.5 – 40.5) = 24.78 Kips

Number of Strands Required = 601.78/24.78 = 24.28

Try 30 – ½ in. diameter – seven wire, 270 ksi strands as an initial trial

- Check for 30 – ½ in. diameter, seven wire strands

Strand eccentricity at mid-span after strand arrangement:

$e_c = 24.78 - \frac{12(2+4)+6(8)}{30} = 24.78 - 4 = 20.78$ in.

Ppe = 30(24.78) = 743.4 Kips

$f_b = \frac{P_{pe}}{A} + \frac{P_{pe}\ e_c}{Sb}$

$f_b = \frac{743.4}{789} + \frac{743.4(\ 20.78)}{10{,}544}$

f_b = 0.94 + 1.47 = 2.41 ksi > fpb reqired = 1.635 ksi (OK)

Therefore, use 30 – ½ in. diameter, seven wire, 270 ksi strands (Figure 21)

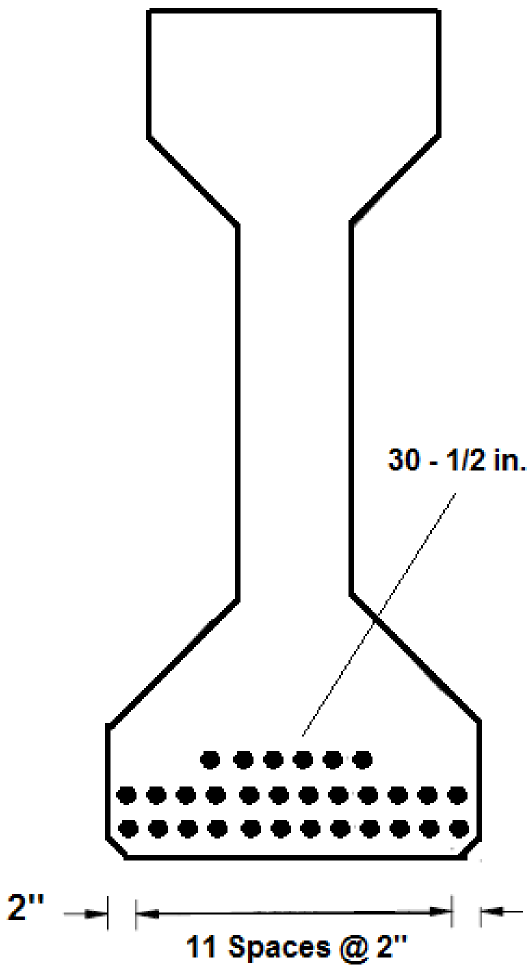

Figure 21: ASSHTO Type IV beam section and strands

2.8. Section 2 Design Values

Below is a summary of design values derived in entire Section 2; some of which will be used in the next section during modeling.

- Total Length of The Bridge: 240 feet
- Span Length: 80 feet (3 spans)
- Total Width of The Bridge: 35 feet (two 12-foot lanes and two 4-foot shoulder)
- Girder Type: ASSHTO Type IV (5 spaces @ 6’-0’’ c/c)
- Bridge Deck: 8 in. Concrete Slab + 1.5 in. Asphalt Wearing
- Total Height of The Bridge (including the column height): 29 feet
- Total Width of The Roadway: 384 in. (32 feet)
- Overhang Length: 2.5 feet

- T501 type barrier width: 1.5 feet
- Design Span, Lc = 80' – 2(8") = 78'-8" = 78.6' (c/c of bearing)
- Effective Flange Width (b) = 72 in. (6 feet)
- Total area of composite section = 1,473 in.^{2}
- Moment of inertia of composite section = 819,141 in.^{4}
- Total DC = 0.821 + 0.600 = 1.421 kips/ft./beam
- Total Super Imposed Dead Load = 0.110 + 0.093 = 0.203 kip/ft./beam
- Total Dead Load = 1.421 + 0.203 = 1.624 kips/ft./beam
- Design Live Load = HL-93 Combinations (Truck-Tandem-Lane)
- DFM = 0.569 lanes/beam
- DFM = 0.349 lanes/beam (for fatigue)
- DFV = 0.671 lanes/beam
- IM for fatigue = 15% [PCI Bridge Design Manual, Chapter 7]
- IM for all other limit states = 33%
- Required precompressive stress in the bottom fiber after losses: 1.635 ksi
- Strand Eccentricity at mid-span: 22.73 in
- Ppi or Fpi= 930 kips
- Ppe or Fpe (After Losses) = 743.4 Kips
- Required Number of Strands = 30 – ½ in. diameter, 270 ksi strands

Composite Section Properties					
	I	Area in.^{2}	y in.	A x y^2 in.^{3}	I + Ay^2
Beam	260,741	789	15.8	196,965.96	457,707
Slab	541.5	684	22.97	360,892.70	361,434.2
Composite Section	Area = 1,473 in^2		Moment of inertia = 819,141 in^4		

Table 8: Section 2 design values (table 1)

Unfactored Bending Moments and Shear Forces Due to Dead Loads										
Distance (x)	Dead Loads				Super Imposed				Total	
	Beam		Slab		Barrier		Wearing			
	V	M	V	M	V	M	V	M	V	M
ft	kips	k-ft	kips	k-ft	kips	k-ft	kips	k-ft	kips	k-ft
0	32.26	0	23.58	0	4.32	0	3.65	0	63.81	0
19.65	16.13	475.5	11.79	347.51	2.16	63.71	1.83	53.86	31.91	940.58
39.3	0	634	0	463.35	0	84.95	0	71.82	0	1254.12

Unfactored Bending Moments and Shear Forces Due to Live Loads						
Distance (x)	x/L	Truck Load with Impact		Lane Load		Fatigue Truck with Impact
		Shear V_{LT}	Moment M_{LT}	Shear V_{LL}	Moment M_{LL}	Moment M_F
ft.		kips	kip-ft.	kips	kip-ft.	kip-ft.
0	0	56.6	0	16.88	0	0
7.86	0.1	50.178	442.15	13.67	134.68	151.63
15.72	0.2	43.755	771.12	10.80	239.44	257.88
23.58	0.3	37.333	986.90	8.27	314.27	262.52
31.44	0.4	30.91	1111.71	6.08	359.17	296.60
39.3	0.5	24.488	1134.68	4.22	374.13	285.25

Table 9: Section 2 design values (table 2)

3. 3D MODELING FOR SEISMIC DESIGN/ANALYSİS

In this section, the proposed bridge will be modeled in three dimension utilizing CSi Bridge software, step by step.

3.1. The Software – CSi Bridge

Overview:

Modeling, analysis and design of bridge structures have been integrated into CSi Bridge to create the ultimate in computerized engineering tools. The ease with which all of these tasks can be accomplished makes CSi Bridge the most versatile and productive software program available on the market today.

Using CSi Bridge, engineers can easily define complex bridge geometries, boundary conditions and load cases. The bridge models are defined parametrically, using terms that are familiar to bridge engineers such as layout lines, spans, bearings, abutments, bents, hinges and post-tensioning. The software creates spine, shell or solid object models that update automatically as the bridge definition parameters are changed.

CSi Bridge design allows for quick and easy design and retrofitting of steel and concrete bridges. The parametric modeler allows the user to build simple or complex bridge models and to make changes efficiently while maintaining total control over the design process. Lanes and vehicles can be defined quickly and include width effects. Simple and practical Gantt charts are available to simulate modeling of construction sequences and scheduling.

CSi Bridge includes an easy to follow wizard that outlines the steps necessary to create a bridge model.

Completely integrated within the CSi Bridge design package is the power of the SAPFire® analysis engine, including staged construction, creep and shrinkage analysis, cable tensioning to target forces, camber and shape finding, geometric nonlinearity (P-delta and large displacements), material nonlinearity (superstructure, bearings, substructure and soil supports), buckling and static and dynamic analysis. All of these apply to a single comprehensive model. In addition, AASHTO LRFD design is included with automated load combinations, superstructure design and the latest seismic design. [www.csiamerica.com]

3.2. Seismic Design in CSi Bridge

The CSi Bridge Program provides automatic seismic design of bridges following the procedures described in ASSHTO Seismic Guide Specifications, and can calculate Demand/Capacity ratios including those for Seismic Design Category "D". Additionally, it can produce a report containing all design results.

3.3. Seismic Design Steps

The seismic design of bridges using CSi Bridge consists of eight basic steps. A quick overview of these eight steps is as follows:

Step 1 – Crete The Bridge Model:

In step 1, construct the model using bridge objects. The focus on the seismic design is on the bents. These bends will be isolated by the program when doing pushover analysis.

Step 2 – Obtain Seismic Design Curve:

In step 2, determine the ground motion hazard. This can be done by simply giving the bridge coordinates using latitude and longitude or by zip codes.

Step 3 – Calculate Cracked Section Properties (Program Automated)

In step 3, the program applies the dead load to the entire bridge and calculates the cracked section properties of the columns.

Step 4 – Calculate Modes & Perform RS Analysis (Program Automated)

In step 4, the program calculates the mods and performs response spectra (RS) analysis which gives demand displacements values.

Step 5 – Isolate the Bents with Equivalent Loads

If the bridge is a seismic design category "D" structure, the bents and foundation of the model are isolated with equivalent load in step 5.

Step 6 – Calculate Displacement Capacity

In step 6, the program calculates the displacement capacity. Code equations are used for seismic design categories “B” and “C” and the pushover analysis is performed for category “D”

Step 7 – Calculate Demand/Capacity Ratios

The program calculates the Demand/Capacity ratios.

Step 8 – Produce Seismic Design Report

3.4. Modeling

3.4.1. Units

Before modeling, users are required to choose the units that will be used throughout the process. In this design, Kip/in./F will be the adopted unit.

3.4.2. Layout Line

The first step of modeling in CSi Bridge is to define layout line. The layout line defines the control geometry for the bridge. The proposed bridge will be straight with no skew. Therefore, this layout line will be represented by two stations (initial-end) in layout prompt.

Since the model has a total length of 240 feet, the end station is supposed to be 2880 inches; the initial value is usually zero.

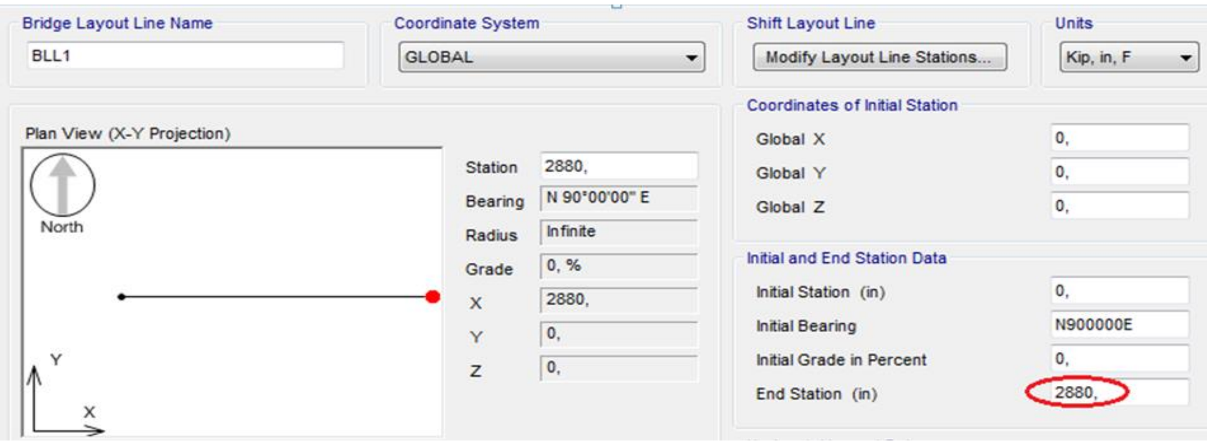

Figure 22: Layout line

3.4.3. Loads Distribution Factors for CSi Bridge

For Precast Girder Bridges, CSi Bridge provides four different options for calculating the load distribution factors. The load distributions factors effects how the shear and moment are distributed to the girders.

Option 1- a) User Defined LDF

b) Spine or Area Objects

c) Single Line

In this method, distribution factors for interior and exterior girders can be specified. The model may either be a spine model in which a single frame object represents the entire cross section or modeled with area objects. In either case, there should be only a single lane that is loaded.

Option 2- a) ASSHTO Defined LDF

b) Spine or Area Objects

c) Single Line

In the second option, CSi Bridge determines the distribution factors from the ASSHTO LRFD Code. The model may use the spine or area objects, but should have only a single lane centered on the model.

Option 3- a) Live Load Demands from Analysis

b) Area Objects Only

c) All Lines

In option 3, CSi Bridge determines the live load demands directly from the analysis in which case the model must use area objects and all three lanes should be defined.

Option 4- a) Loads Distributed Uniformly

b) Spine or Area Objects

c) All Lines

In the last option, the program distributes the loads equally to all girders and the model may use either span or area objects. All lanes should be defined.

For this model, the second option will be used during modeling. Therefore, the load distribution factors will be determined from the code by the program.

Since Option 2 is selected, a single lane is defined, which will be 12-foot or 144 inch wide and runs along the layout center line.

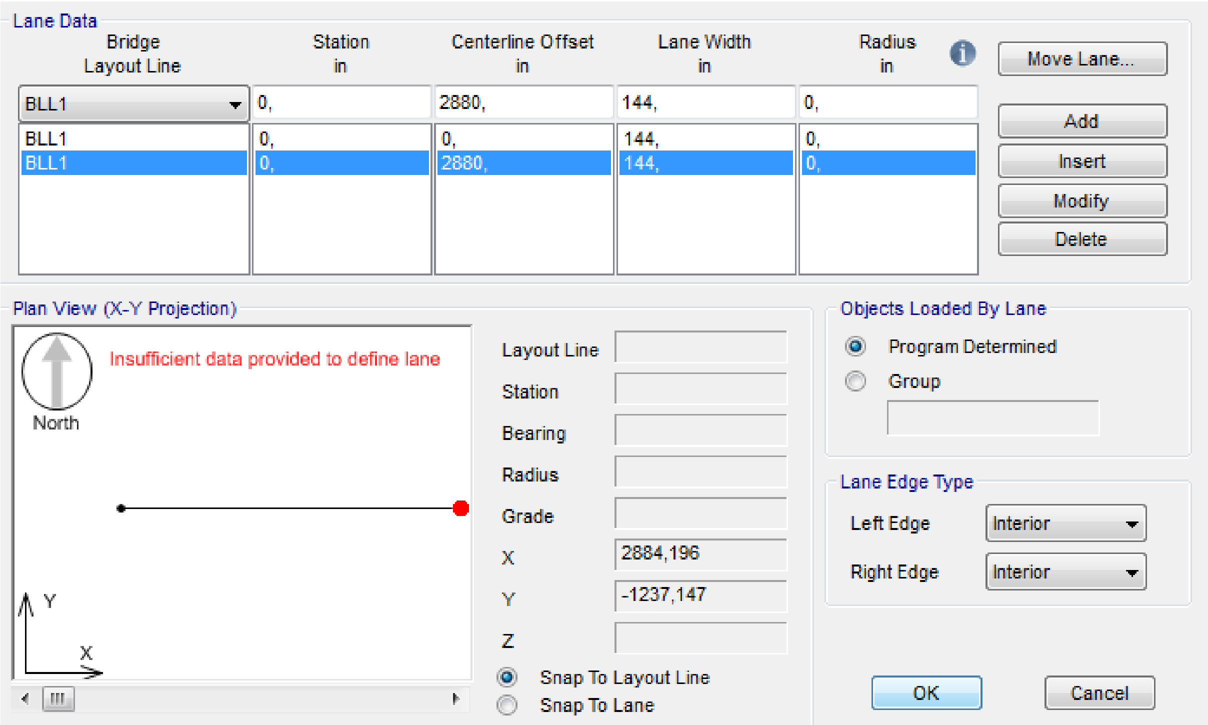

Figure 23: Traffic lane prompt

3.4.4. Defining Materials Properties

Each material used to construct the bridge model such as deck concrete, girder concrete, prestressing steel and reinforcing steel will be defined step by step in the following section.

3.4.4.1. Concrete

<u>1- Deck Concrete</u>

- Concrete Strength at 28-days, fc’ = 4,000 psi (4 ksi)
- Unit weight of concrete = 150 pcf
- Modulus of Elasticity, Ec (ksi) = 3605 ksi

Using the components tab - properties panel, the property type is selected to be materials, and modify the sample concrete values to match the properties above - also depicted in Figure 24.

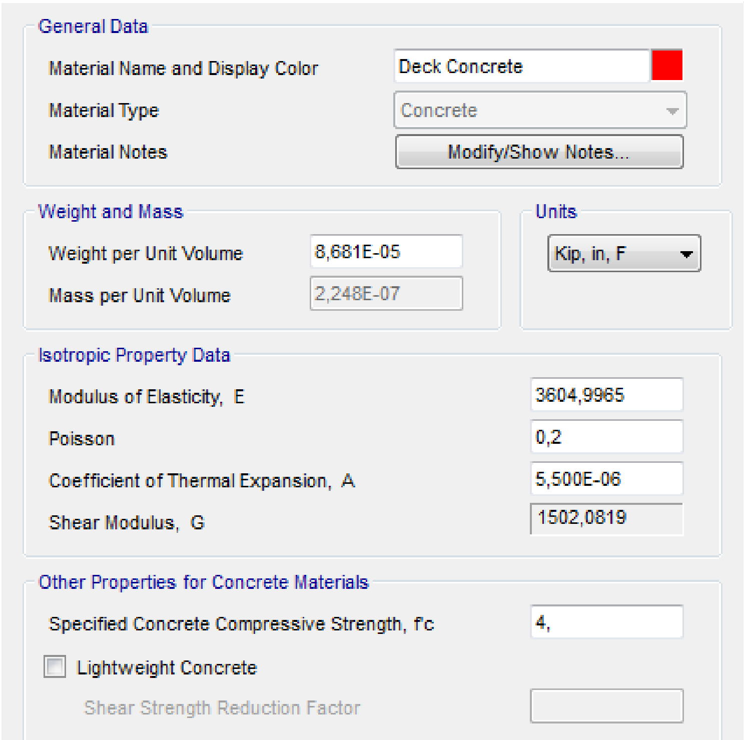

Figure 24: Properties panel – deck concrete

2- Girder Concrete

For the girders, a new concrete type is defined using the data below:

- Concrete Strength at 28 days, f'c = 5000 psi, High Strength Concrete - Normal Weight
- Concrete Strength at transfer, f'ci = 4000 psi (Assumed the 80 percent of the cylinder strength, f'ci = 0.8xf'c)
- Concrete unit weight = 150 pcf
- Modulus of Elasticity, Ec (ksi) = 57,000 x $\sqrt{5000}$ / 1000 = 4031 ksi
- Poisson's Ratio (U) = 0.2

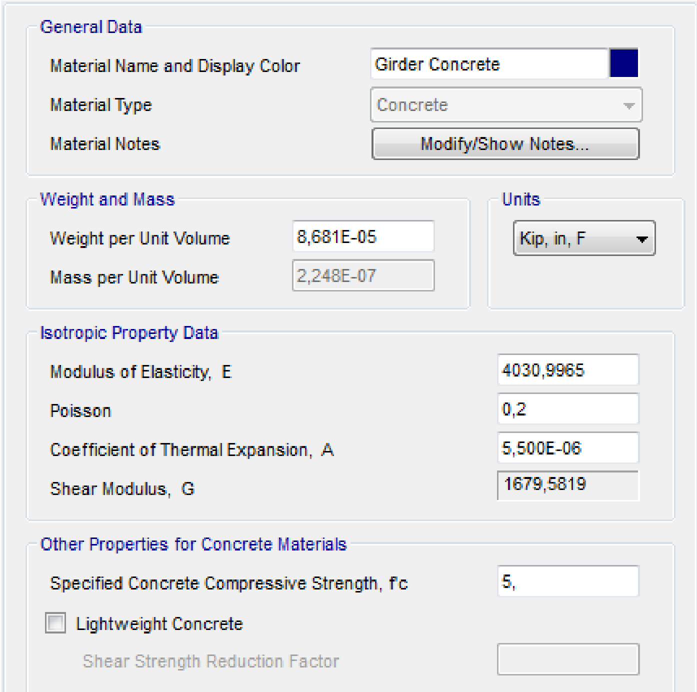

Figure 25: Properties panel – girder concrete

3- Bent Concrete

- Concrete Strength at 28 days, f'c = 6000 psi, High Strength Concrete - Normal Weight
- Modulus of Elasticity, Ec (ksi) = 4415 ksi
- Poisson's Ratio (U) = 0.2

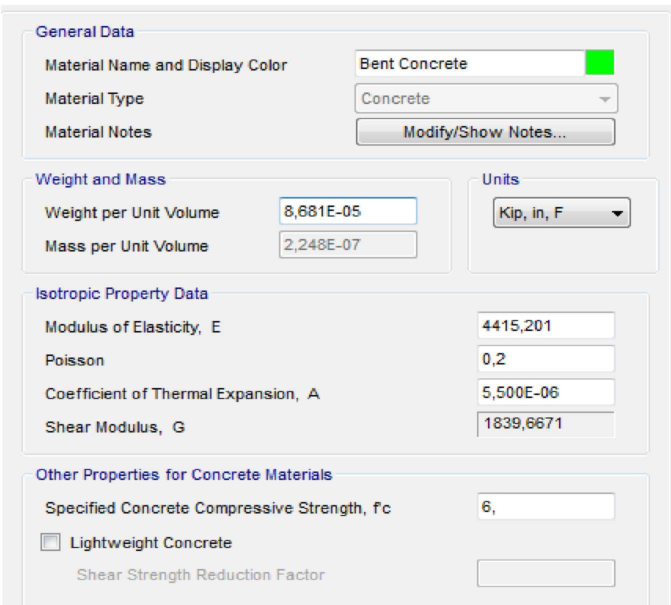

Figure 26: Properties panel – bent concrete

3.4.4.2. Prestressing Material

The material properties of ASTM A416 – Grade 270 prestressing steel are entered in to the material panel of the software as shown in Figure 27.

Prestressing Strands (½ in. diameter - seven wire low relaxation)

- Area of one strand = 0.153 in.2
- Ultimate Stress, fpu = 270,000 psi
- Yield Strength, fpy = 0.9 fpu = 245,000 psi (LRFD)

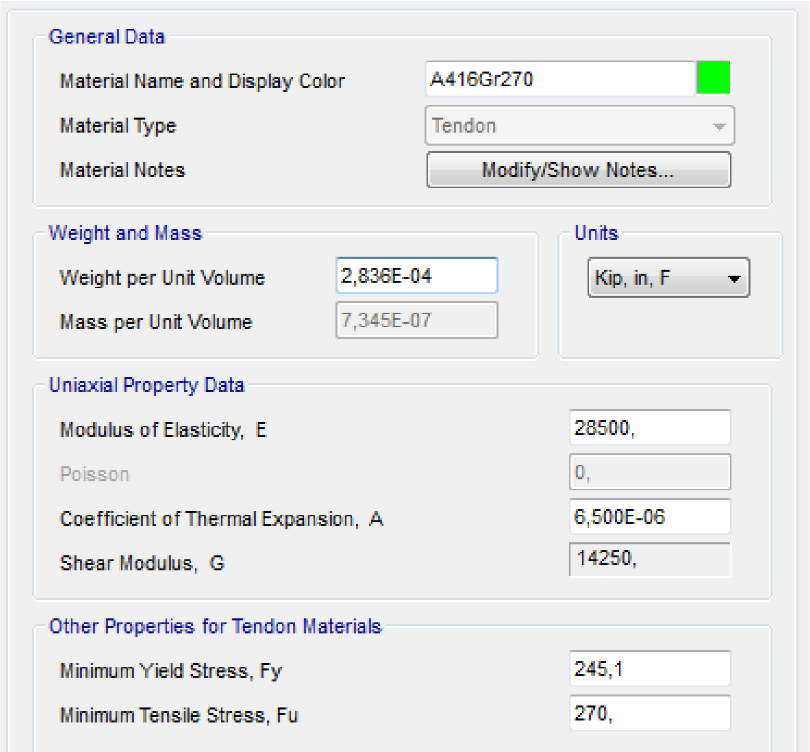

Figure 27: Properties of ASTM A416 – Grade 270

3.4.4.3. Reinforcing Steel

Reinforcing Steel (A615 - Grade 60)

- Minimum Yield Strength, fy = 60,000 psi
- Modulus of Elasticity, Es = 29,000 ksi (ASSHTO)
- Poisson : 0.3

Reinforcing steel is defined as shown in Figure 28.

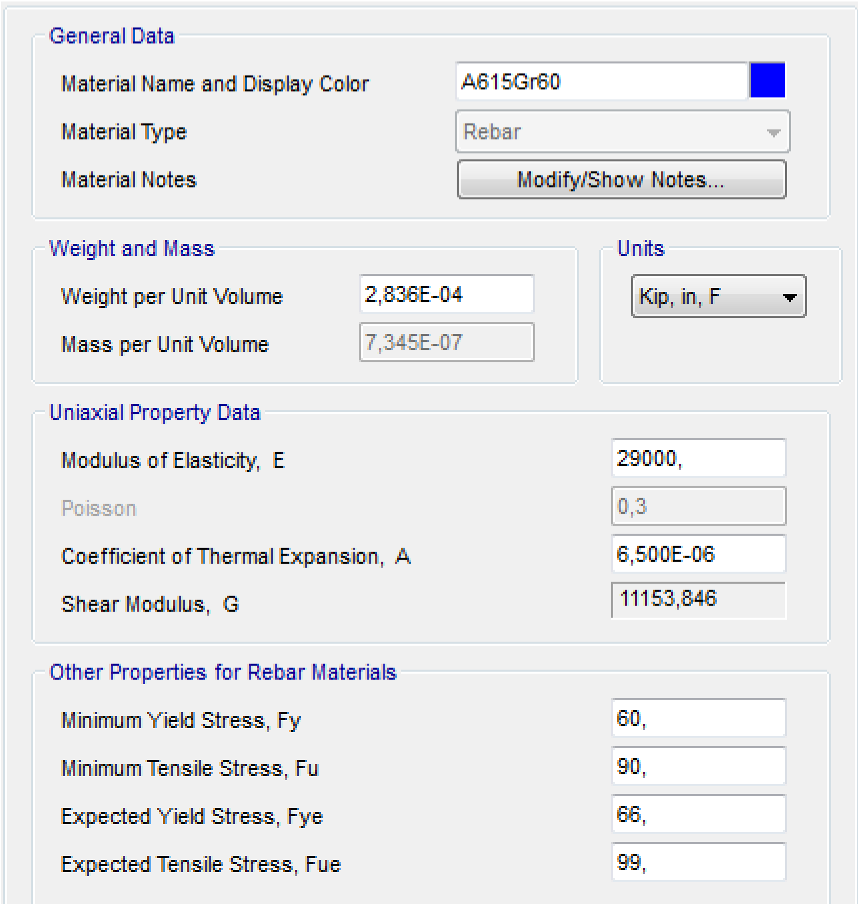

Figure 28: Reinforcing steel properties

3.4.5. Defining Section Properties

3.4.5.1. ASSHTO Type IV Precast Girders

Section Properties of precast girders are defined by using the frame properties panel as shown below.

First, the frame section property type is selected to be precast concrete; then, Type IV Girder is selected from the ASSHTO standards. Lastly, the concrete type must be changed to previously defined “girder concrete (5000psi)” (Figure 30).

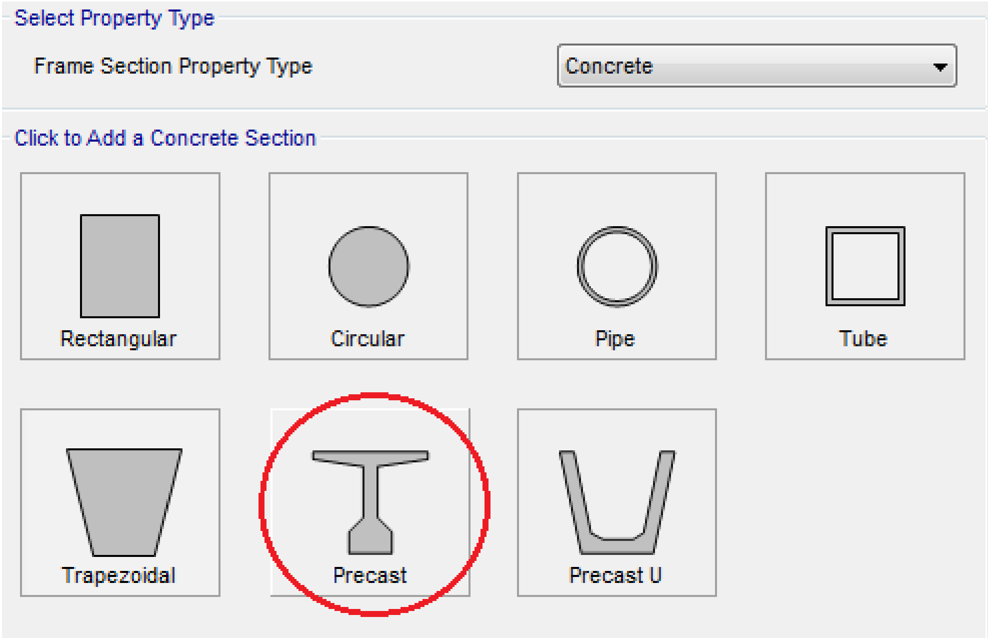

Figure 29: Precast concrete panel

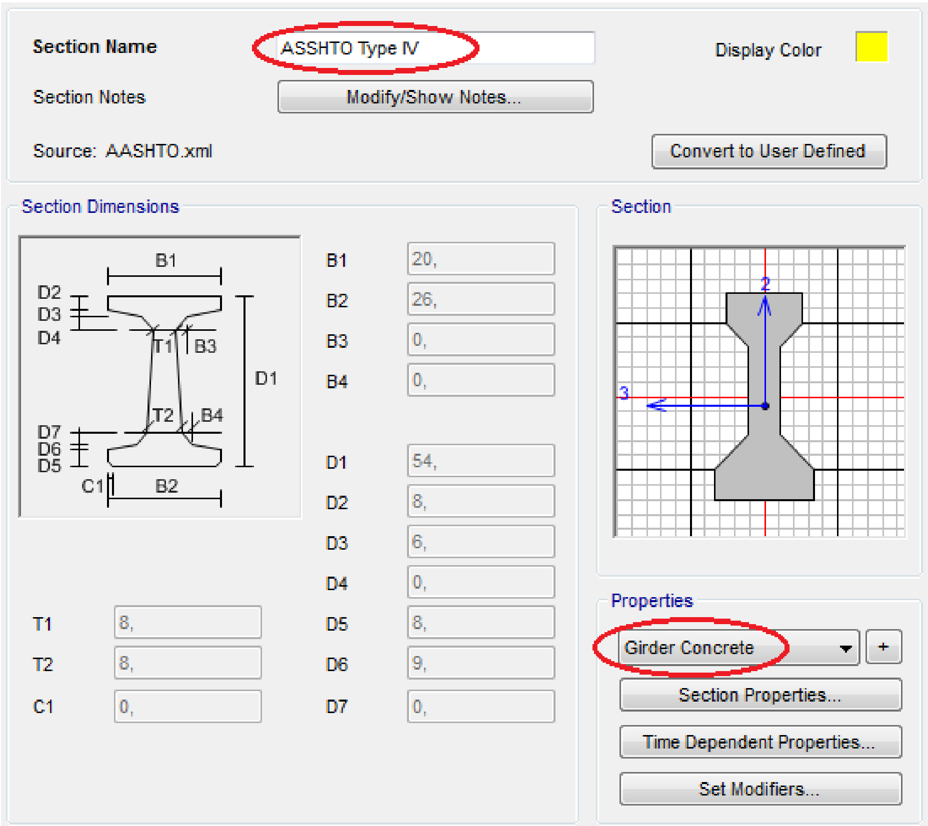

Figure 30: AASHTO Type IV girder section dimensions

3.4.5.2. Superstructure - Deck Section

Since the precast girders have been completed, now the deck section properties can be defined using the superstructure panel; the process is explained below:

1- Click on new deck button, then select the deck section type as "concrete deck on composite girders - Precast I Girder" (Figure 31)

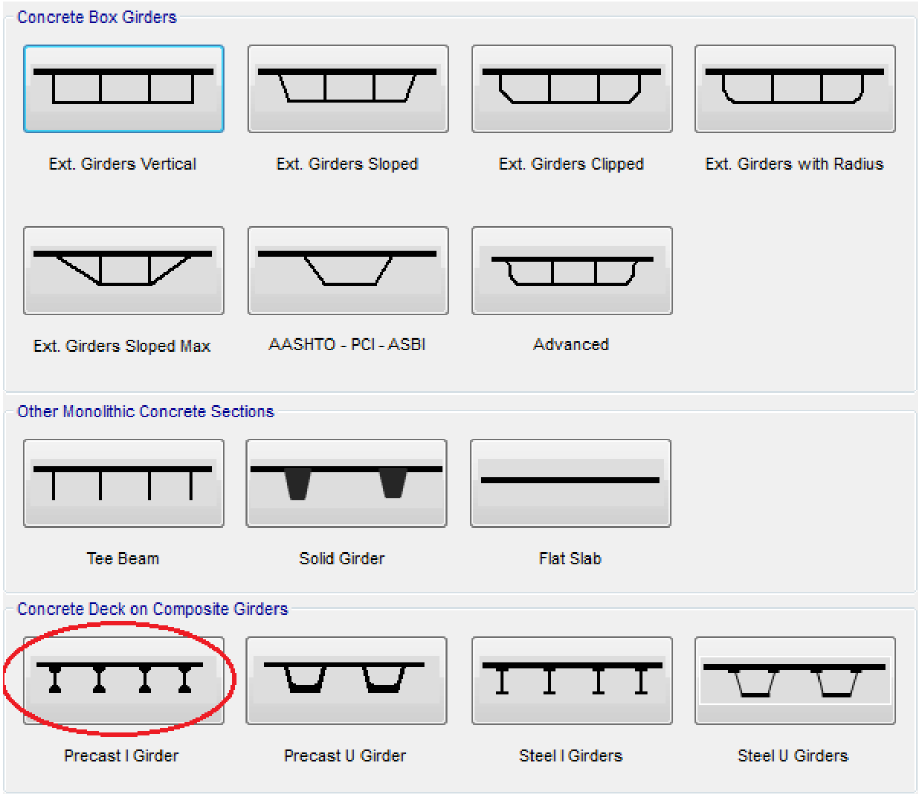

Figure 31: Superstructure panel - concrete deck on composite girders

Then,

2-Verify slab material matches the previously defined "Deck Concrete",

3-Change number of total interior girders to 4, since the model has 6 girders total.

4-Enter total width of bridge to be 35 feet or 420 in.

5-Consider that girders are laid out along the layout line.

6-The top slab thickness should be 8 in. and there should be no haunch.

7-The girder section should be ASSHTO Type IV which had been already defined.

8-The left and right overhang lengths should be 30 in. (2.5 feet)

9-The distance to the edges of the curb should be 66 in.

The detailed finished section of the bridge deck is shown below. (Figure 32 - Figure 33)

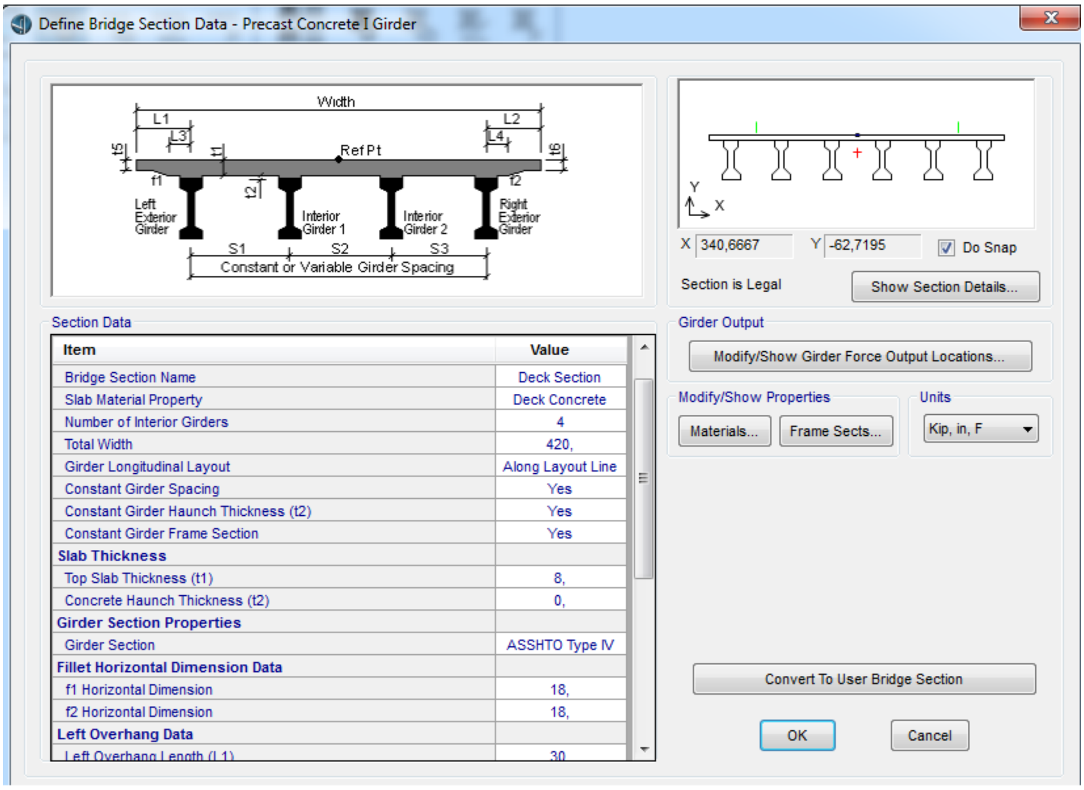

Figure 32: Section properties of bridge deck (on the left) and finished section (on the right)

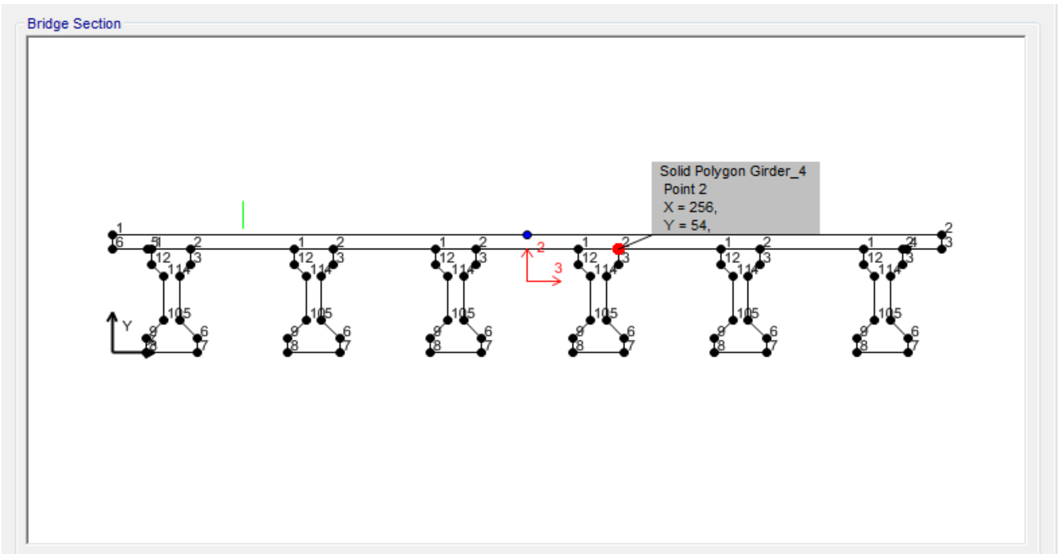

Figure 33: Detailed bridge section

3.4.5.3. Defining Bent Cap Section

Each interior bent of the model consists of a bent cap and three columns. Therefore, these sections (column section & bent cap section) are supposed to be defined separately in CSi Bridge.

The cross section of bent cap is shown in Figure 34. Using frame properties panel and selecting rectangular concrete section, the bent cap is defined as below: (Figure 35)

1- Material is selected as “Bent Concrete” (4000 psi concrete)

2- Depth (t3) is 42 inches and width (t2) is 39 in. (Figure 34)

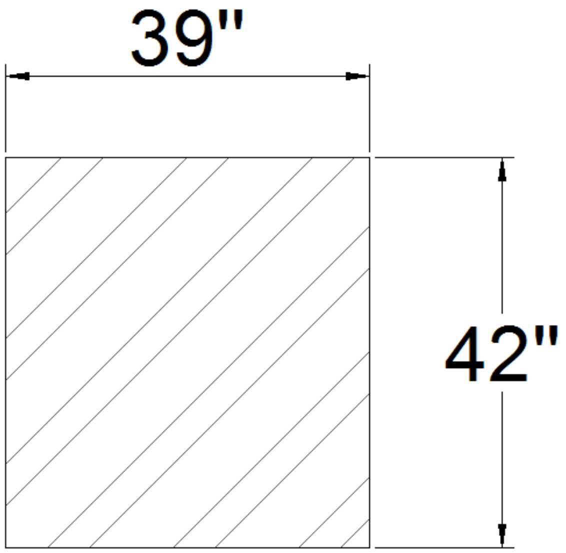

Figure 34: Cross section of bent cap

In addition to these two steps, concrete reinforcement data is modified with the following sub-steps.

1-Materiel is selected as “reinforcement steel” which had been already defined in Section 3.4.4.3.

2-Design type is selected to be “beam”, since a beam section is being defined. (Figure 36)

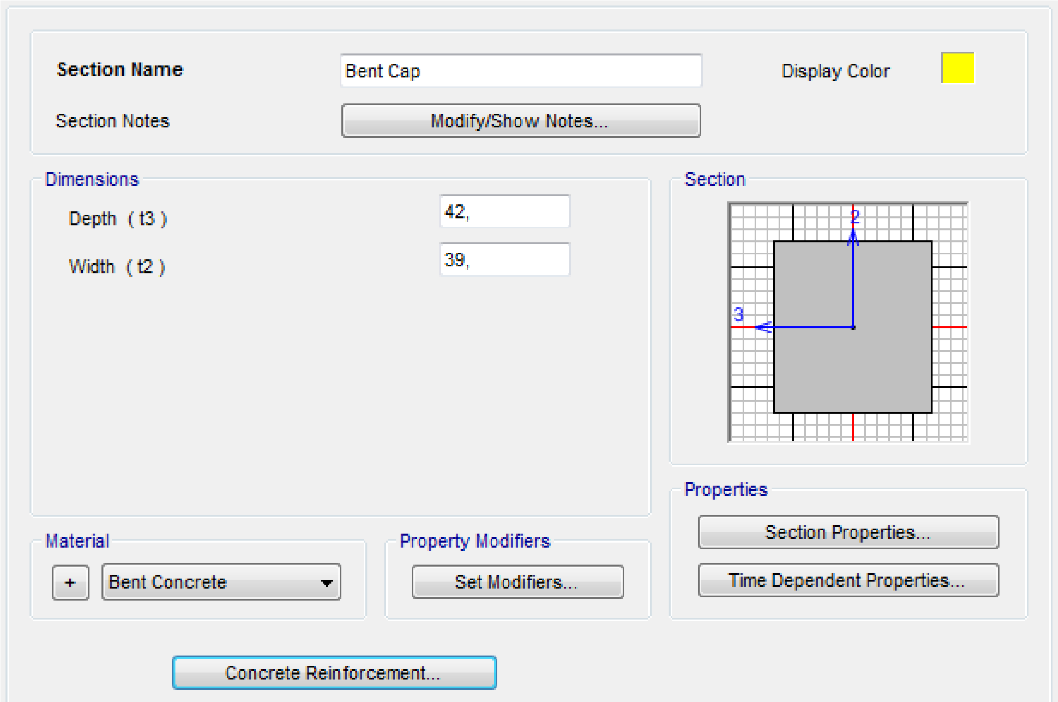

Figure 35: Details of bent cap section in CSi Bridge

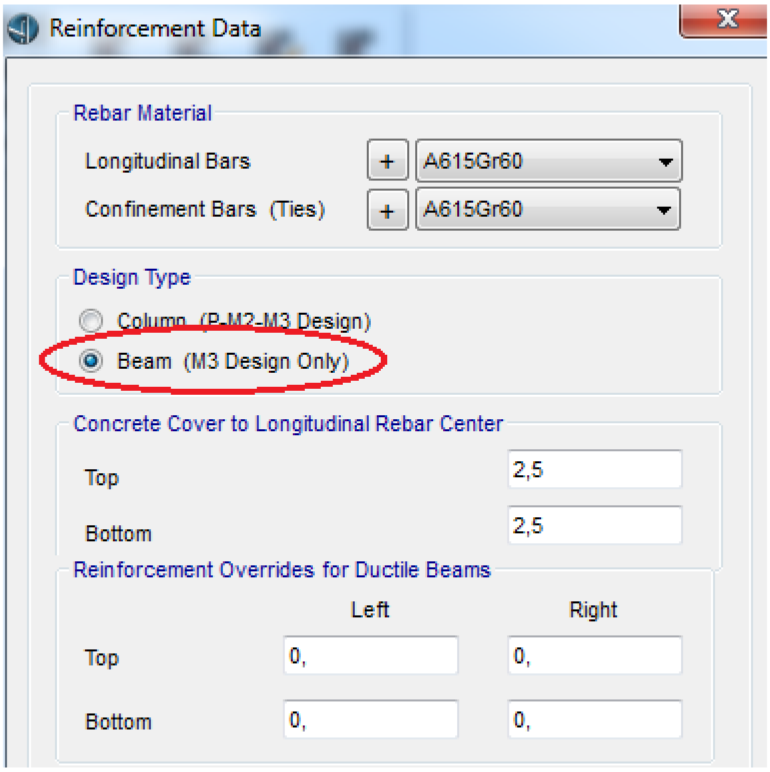

Figure 36: Reinforcement data for bent cap

3.4.5.4. Defining Bent Column Section

The second required process for interior bents is to define a column section. As mentioned in the previous section, necessary arrangements for concrete reinforcement are listed below. Also the column cross-section is shown in Figure 37 and Figure 39.

1-Design type is selected as “column”.
2-A615-GR50 is selected as the rebar material.
3-Number of longitudinal bars is 13, and the bar size is #8.
3-Reinforcement Configuration must be “circular”.
4-Confinement bars are selected to be “spiral”.
5-“Check/Design” option is selected as “reinforcement to be check” in order to see if the reinforcement is adequate. (Figure 38)

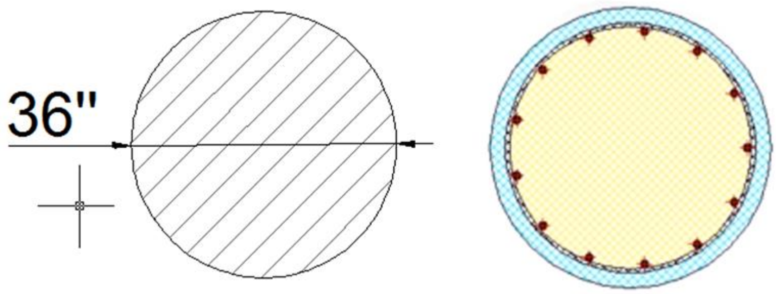

Figure 37: Column cross-section

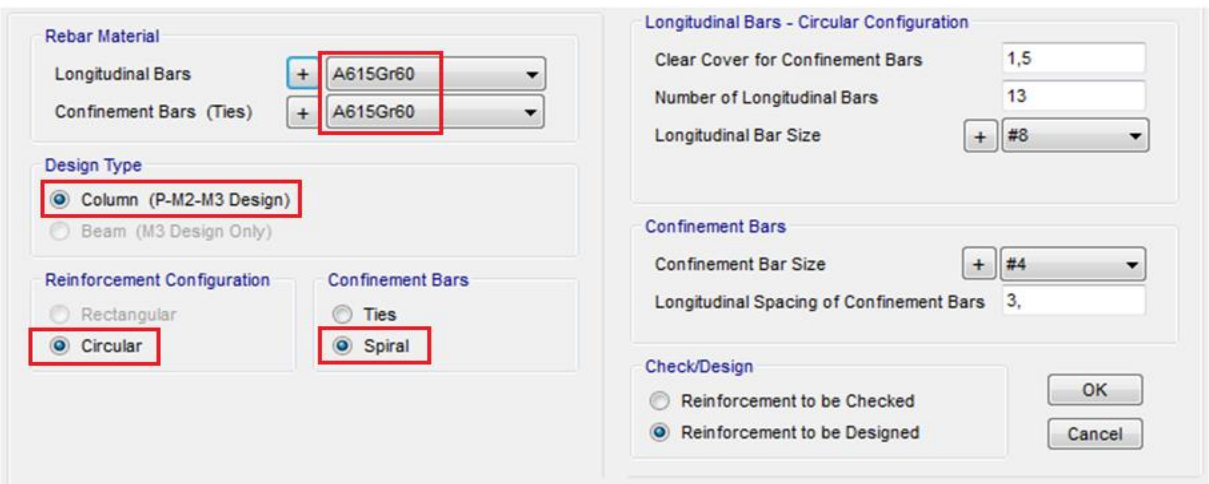

Figure 38: Reinforcement data for columns

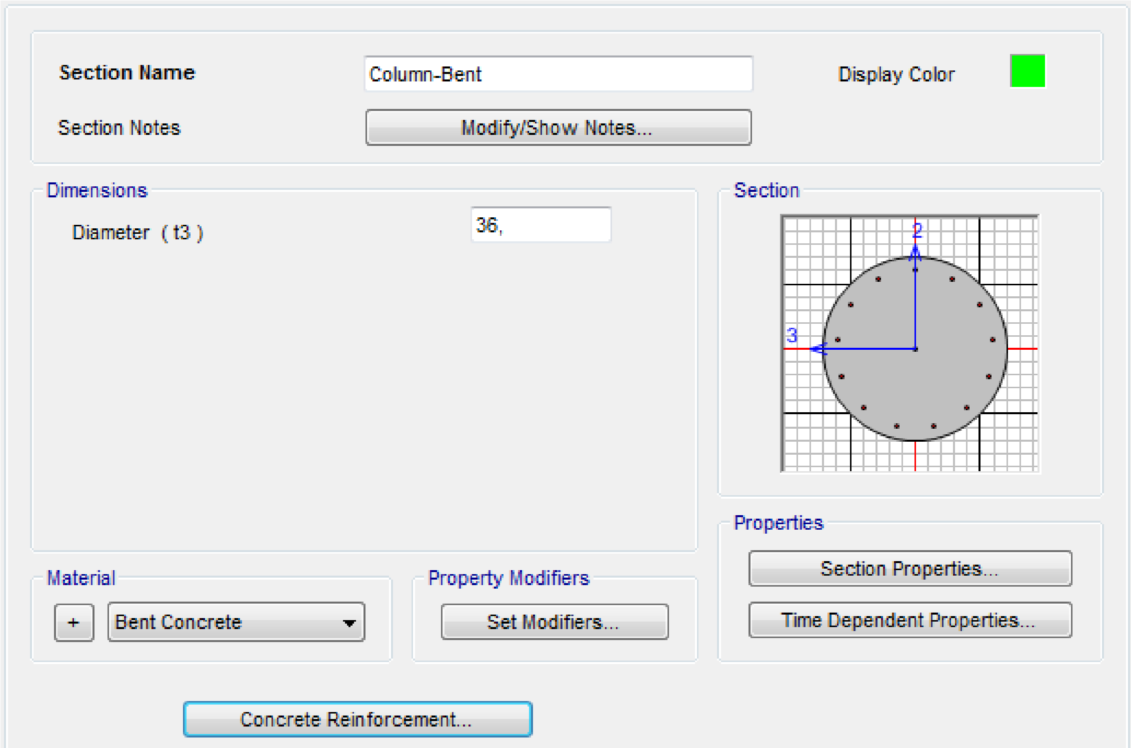

Figure 39: Details of column cross-section

3.4.6. Defining Substructure Components

Substructure of the model consists of two interior bents (bent cap and columns) and two abutments located at the each end point of the bridge. In addition to the bents and abutments, a total of twenty-four mechanical and seismic isolation bearings will be placed between the precast girders and the substructure elements (bents and abutments) in order to control movement or possible damage caused by seismic activity.

The substructure components mentioned above such as bents, abutments, bridge bearings and foundation springs will be defined separately in following sections.

3.4.6.1. Mechanical and Seismic Isolation Bearings

According to ASSHTO, a bridge bearing is a structural device that transmits loads while facilitating translation and/or rotation".

In general, bridge bearings are used to transfer forces or loads received from the decking on to the substructure, allowing the following types of movements of the superstructure:

- Translational movements
- Rotational movements

A bridge may have different types of bearings depending on various conditions or type of its structure. Therefore, there are several types of bearings used in construction of bridges. Most common types of these bearings are:

- Seismic Isolation Bearings such as Elastomeric Bridge Bearings (Figure 40)
- Pinned Bearings
- Rocker Bearing
- Roller Bearing
- Fixed Bearing

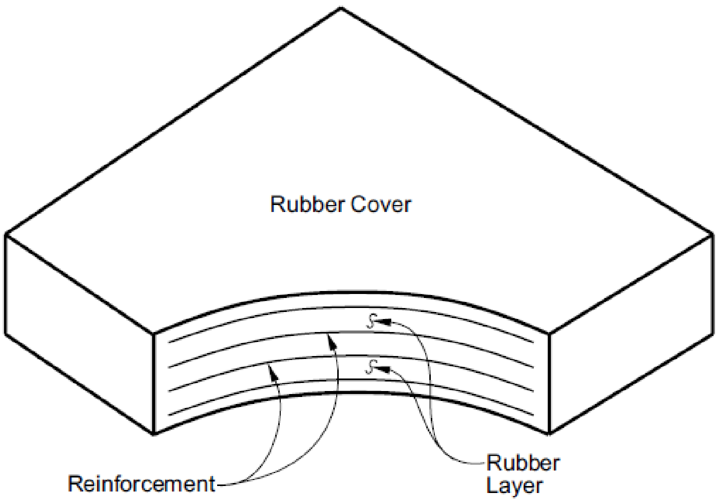

Figure 40: Elastomeric bearings

In proposed model, two different types of bearings will be used, namely, elastomeric bearings for abutments, and fixed bearings for interior bents. The fixed bearings taken into consideration for interior bents will allow rotation, but no other forms of movement. The elastomeric seismic isolation bearings which are ideal for protecting critical structures from ground movement in all seismic zones and soil types will be considered for the abutments.

1- Elastomeric Seismic Isolation Bearings:

The seismic isolation bearings adopted for the abutments restrain any motion in vertical direction and rotational motion around the layout line as shown in Figure 41.

2- Fixed Bearings:

Seismic Isolation Bearings required for the interior bents will allow rotation, but not other forms of movement as shown in Figure 42.

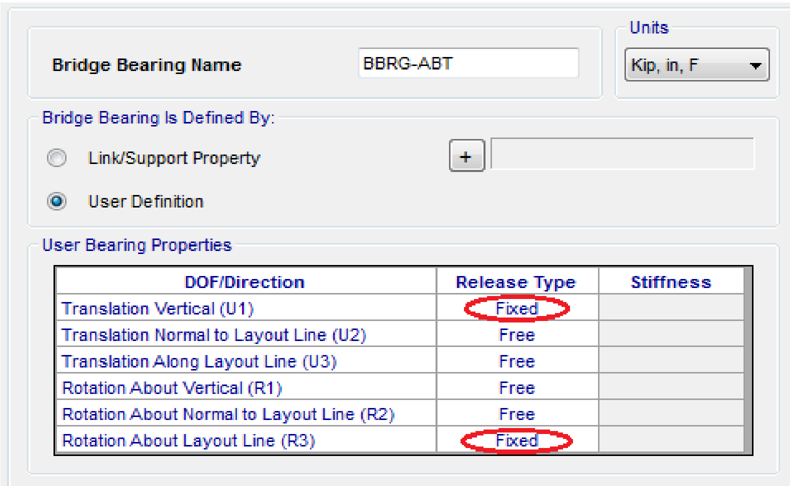

Figure 41: Properties of abutment bearings

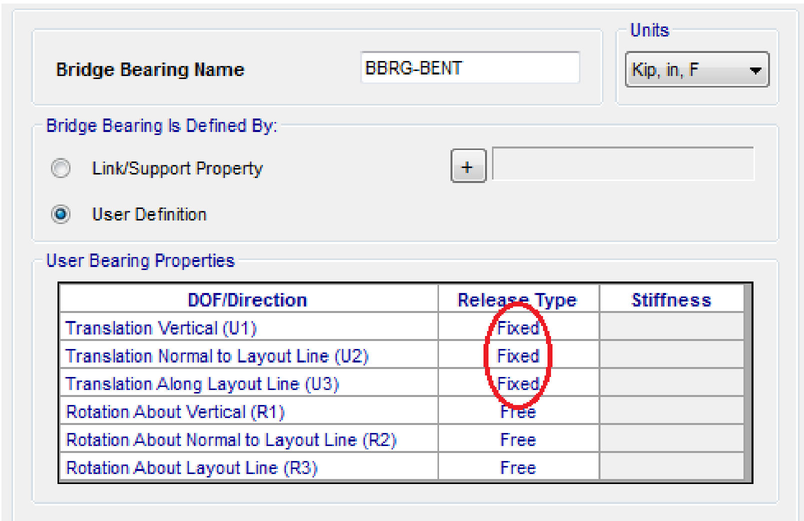

Figure 42: Properties of bent bearings

3.4.6.2. Foundation Springs

In this bridge model, the foundation springs will be defined as “fixed” in all degrees of freedom as shown in Figure 43.

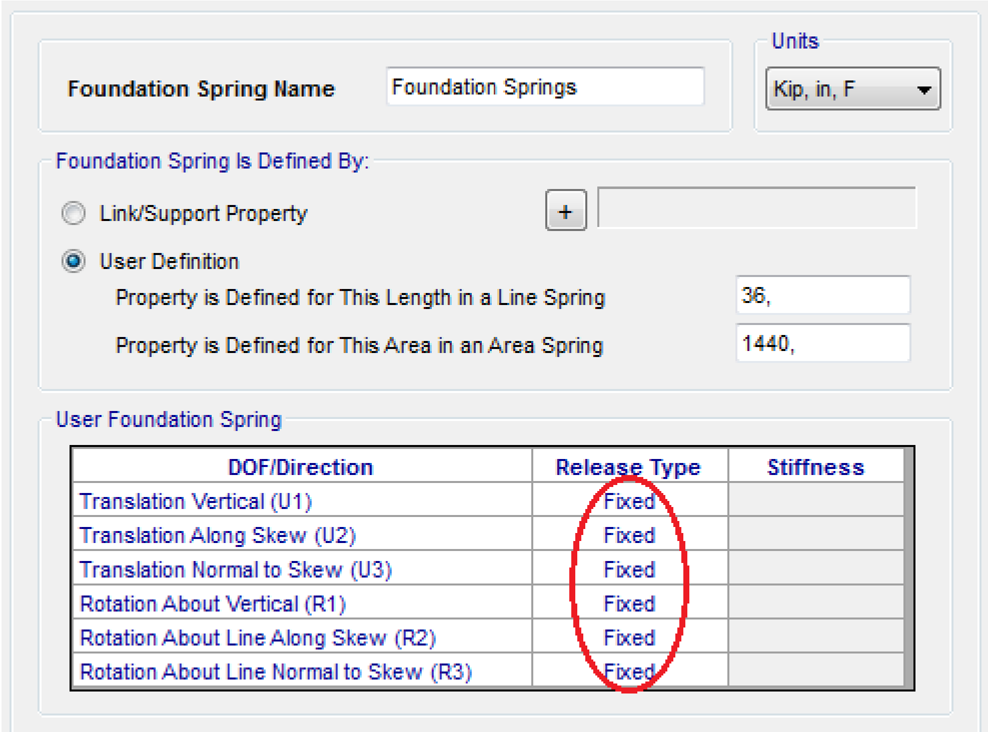

Figure 43: Properties of foundation springs

3.4.6.3. Abutments

The bridge model has two abutments located at each end point of the superstructure. By following the instructions and selecting values in abutment panel of CSi Bridge as mentioned below, the abutments are defined. (Figure 44)

1-Girder support condition is selected as “connect to girder bottom only”.

2-Substructure type is selected as “foundation spring”, since there is no section that was defined for the abutments.

3-Foundation spring property is selected as “fixed”

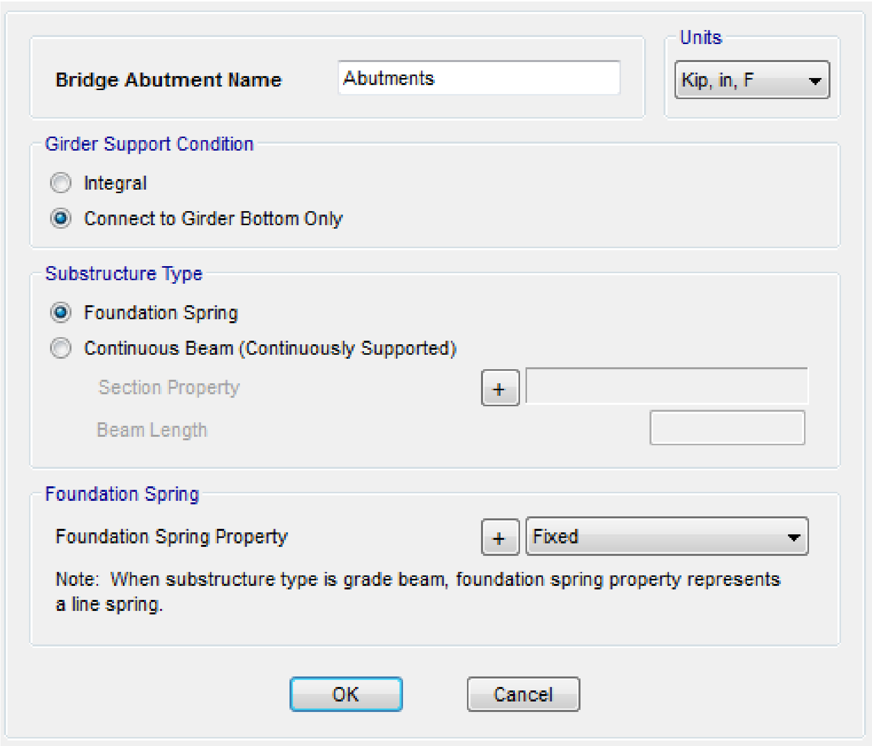

Figure 44: Abutments data prompt

3.4.6.4. Interior Bents

In addition to the abutments defined in the previous section, two interior bents spaced 80 feet are used to support the superstructure. These bents are defined considering the following steps: (see Figure 45)

1-Length of Cap beam should be 420 inches (same as the bridge width).

2-Number of column should be 3.

3-Bent type is selected as "single bearing line", since the bridge model has continuous superstructure (more than one span).

4-Girder support type option should be "connect to girder bottom only".

Using "modify column data" button, the columns are organized as below (see figure 46):

1-To provide 12.5 feet space (based on the bridge geometry) between each column, the following distance values should be entered: (5 feet – 17.5 feet – 30 feet) or (60 in. – 210 in. – 360 in.)

2-Height of column should be 20 feet or 240 inches and there is no angle.

3-Lastly, foundation springs should be fixed.

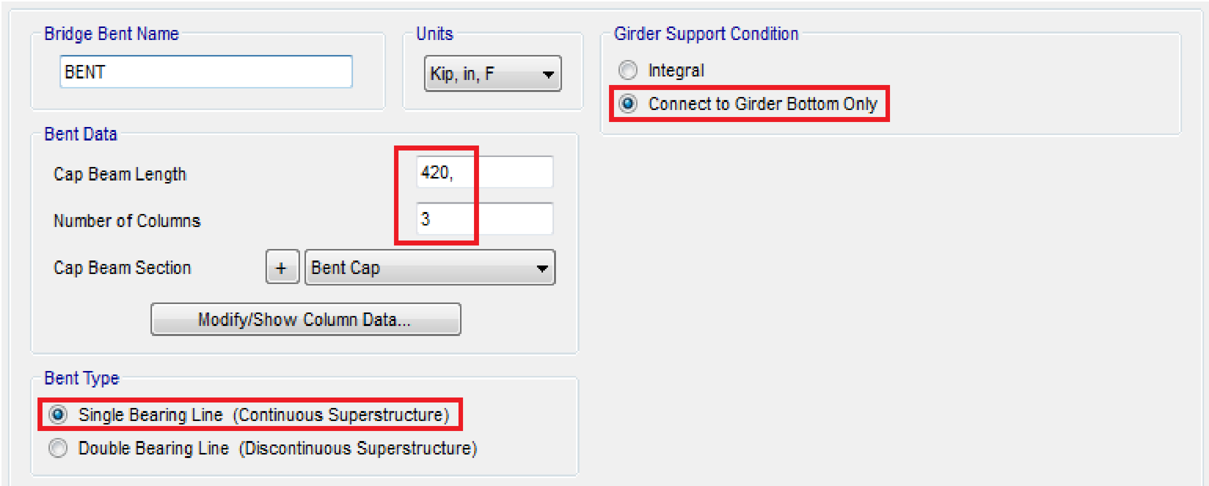

Figure 45: Bent date

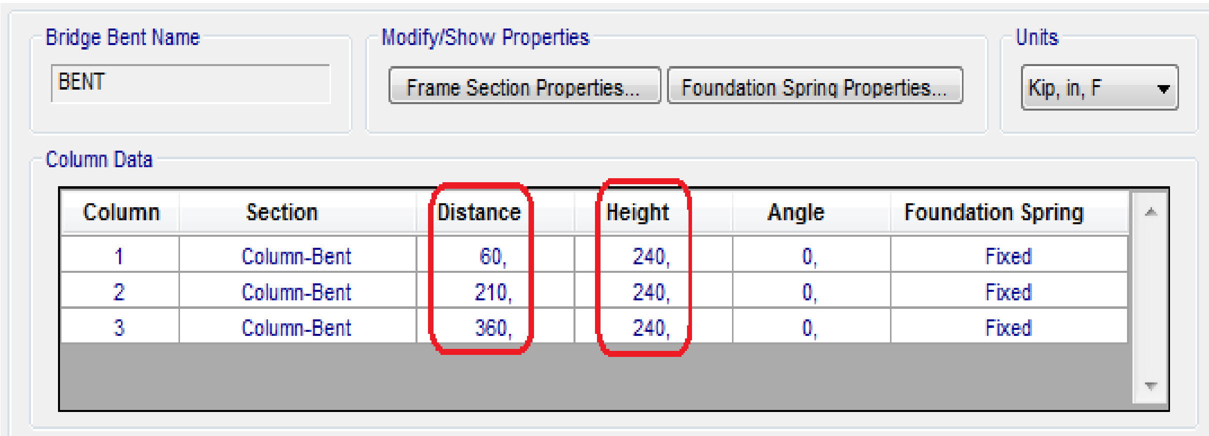

Figure 46: Bent date 2

3.4.7. The Effect of Live Loads in Seismic Design

Unlike earlier prestressed concrete girder design, in this section, the effect of live loads will not be considered for the seismic design. Most seismic design procedures for earthquake-resistant bridges do not include the effect of live load for two primary reasons. First, it is unclear, what fraction of the full design live load will be on the bridge during the design earthquake, and second, it is believed the seismic response of a bridge is dominated by the dead load of the bridge, and the self-weight and inertial effects of the live load are negligible in comparison. *[State Of California - Department Of Transportation, Report Number CA13-2349]*

3.4.8. Defining Prestress Load Pattern

In order to provide prestressing effects to the girders, first, a new load pattern namely, prestress is defined; second, enter the number of strands that had been calculated in Section 2 in to tendon panel of CSi Bridge (Figure 47- Figure 48).

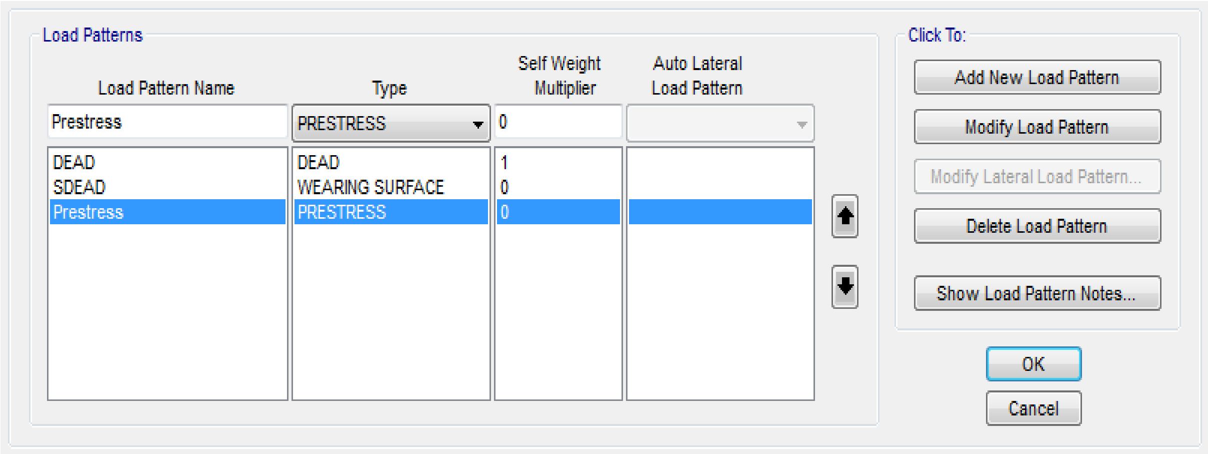

Figure 47: Prestress load pattern

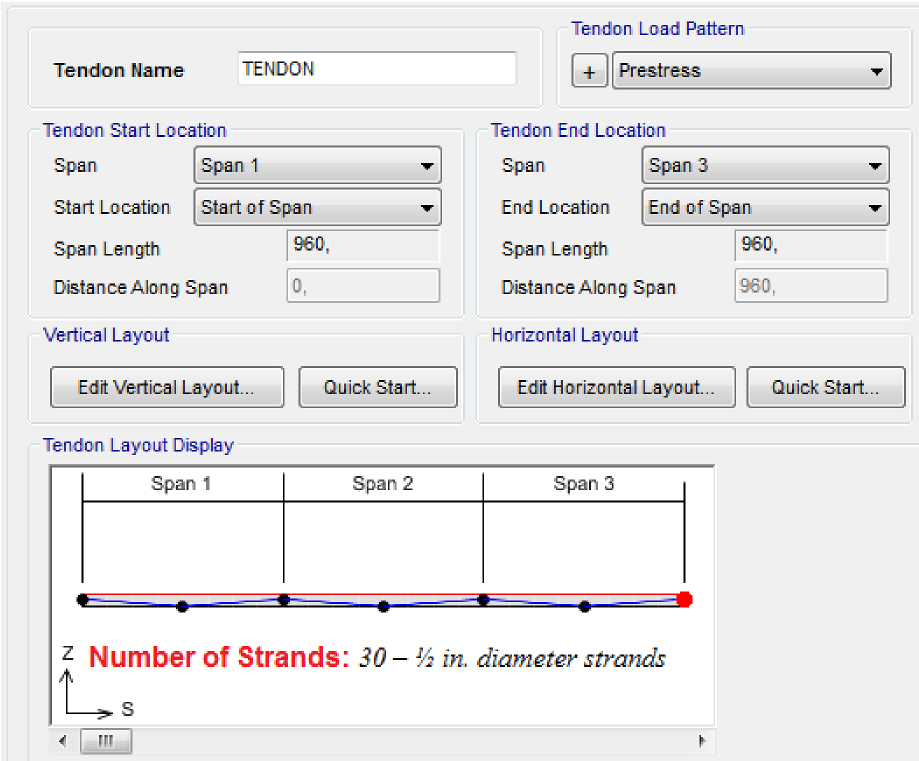

Figure 48: Tendon data in CSi Bridge

3.4.9. Ground Motion Hazard

As mentioned in Section 3.3 (Seismic Design Steps - Step 2), the ground motion hazard needs to be determined in order to consider the effect of a possible seismic activity on the bridge model. Using CSi Bridge, ground motion hazard is defined by following the steps below: (Figure 49)

1-Using the function panel of CSi Bridge, the function type is selected to be “Response Spectrum”, and the standard is selected to be “ASSHTO 2012”.

2-The software can determine function values by latitude, longitude, or zip code. Users however, have the ability to self-set function values manually. Latitude and longitude values will be entered for the model.

3-It is assumed that the bridge is located in San Francisco, CA. Therefore, the latitude and longitude values can be entered as below:

- Latitude: 37.7739
- Longitude: -122.431

4- The site class is assumed to be “C

5- Lastly, the function graph is plotted automatically by the program as shown in Figure 49.

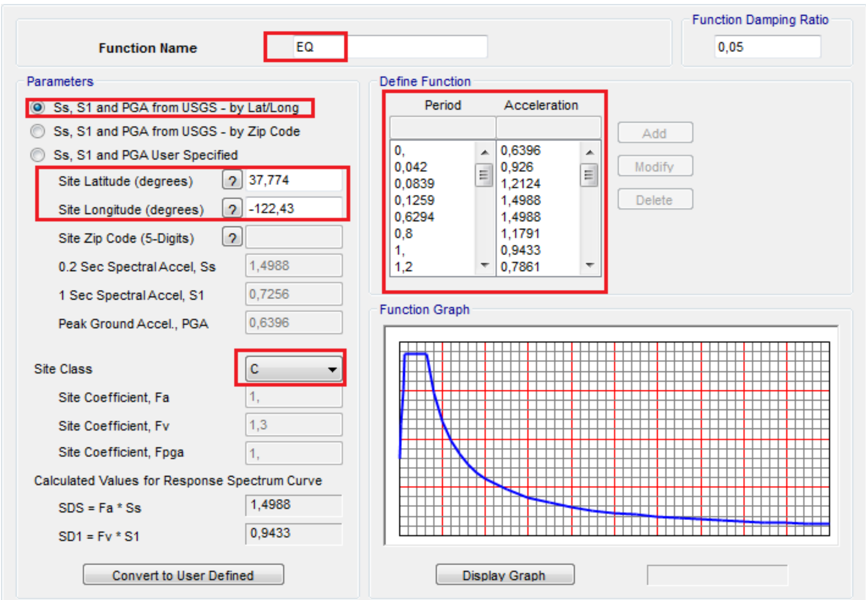

Figure 49: Response spectrum function graph

3.4.10. Integrating the Bridge

Since all the components of the bridge have been defined separately, they should be assembled into a single bridge. This process also called “bridge object assignments” allows CSi Bridge to isolate the loads and the bents when carrying out the seismic analysis. In order to assemble all of the components, the following steps are executed:

1- The reference line of the bridge defines the abutment and bent locations. Since the model has three spans, the reference line is set to have three spans by modifying the values of start and end stations considering 80-foot span length as shown in Figure 50.

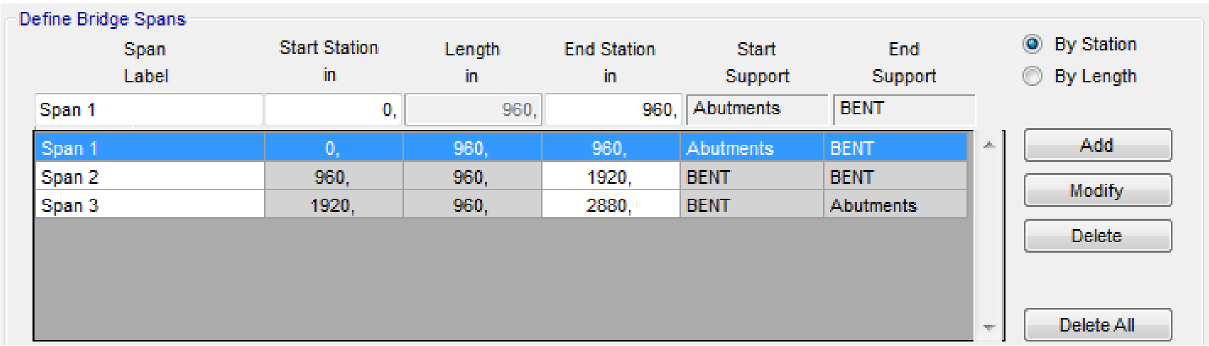

Figure 50: Start-end stations for bridge spans

2- For start and end abutments,

The substructure property is selected as “Abutment”; this data was made readily available in Section 3.4.6.3 to be entered here. The substructure elevation (Global Z) is entered as “- 64” by considering the deck thickness (8 in), girder depth (54 in.) and 2-inch thick bearing resting on a fixed substructure. The bearing property is selected to be “BBRG-ABT” (already defined for the abutment bearings in Section 3.4.6.1). Lastly, Global Z for the bearings is entered as -62 since the bearing thickness is 2 inches (see Figure 51).

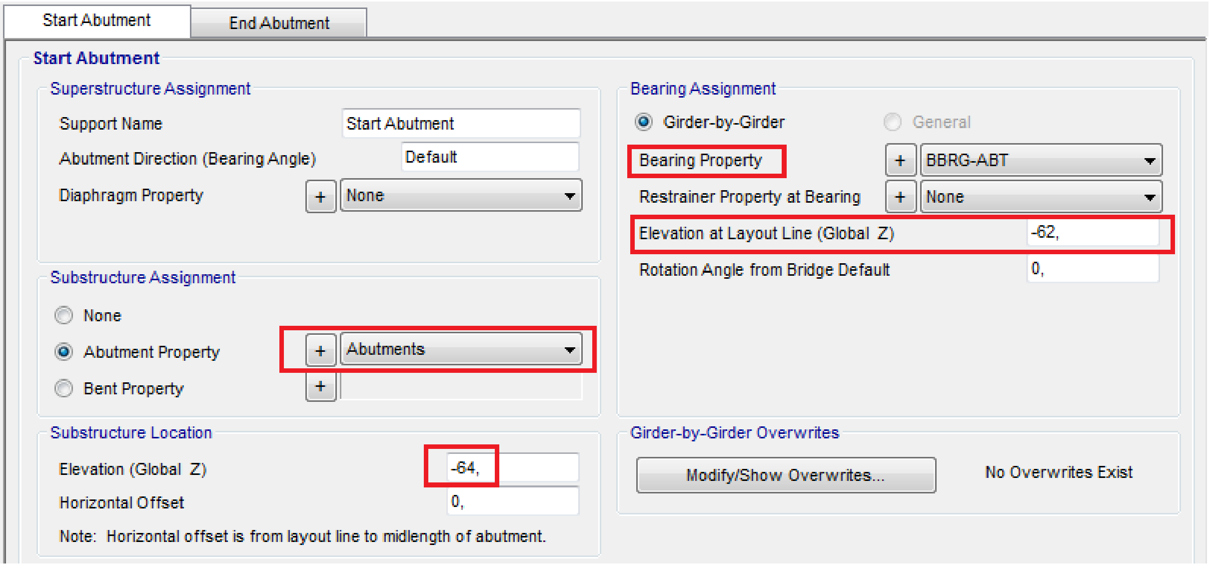

Figure 51: Start-End abutments panel

3- For interior bents,

The bent property is selected as “BENT”; this data was made readily available in Section 3.4.6.4 to be entered here. The substructure elevation (Global Z) is entered as “- 64” by considering the deck thickness (8 in), girder depth (54 in.) and 2-inch thick bearing resting on the bent structure. The bearing property is then selected to be “BBRG-BENT” (already defined for the bent bearings in Section 3.4.6.1). Lastly, Global Z for the bearings is entered as -62, since the bearing thickness is 2 inch (see Figure 52).

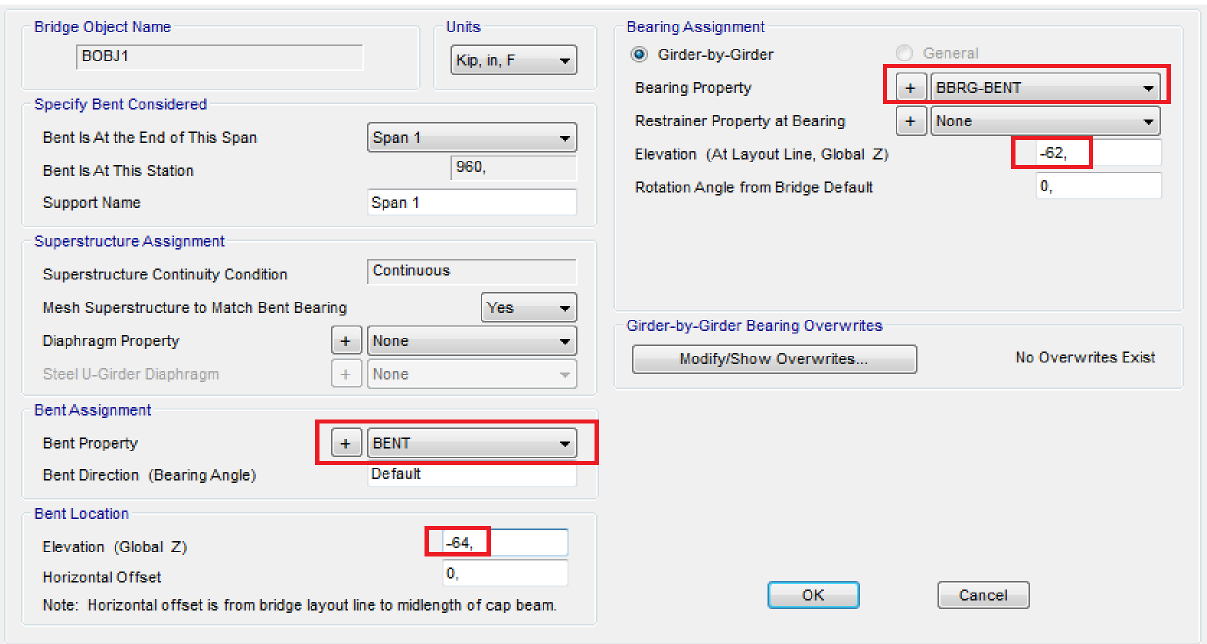

Figure 52: Interior bents panel

4- After completing all the steps above, a simplified bridge object plan is obtained as in Figure 53.

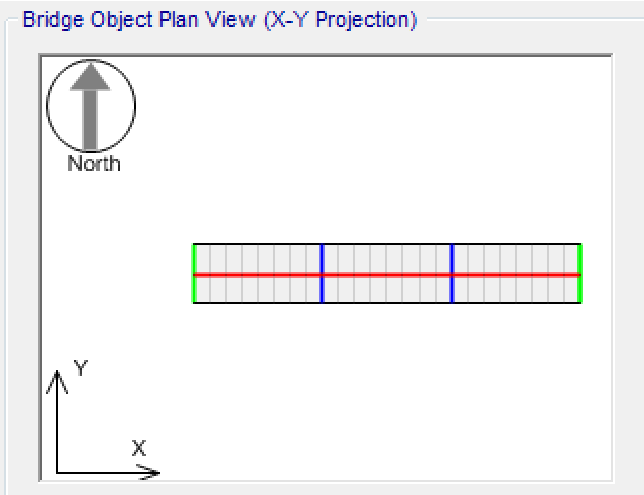

Figure 53: Bridge object plan view

3.4.11. 3D Bridge Model

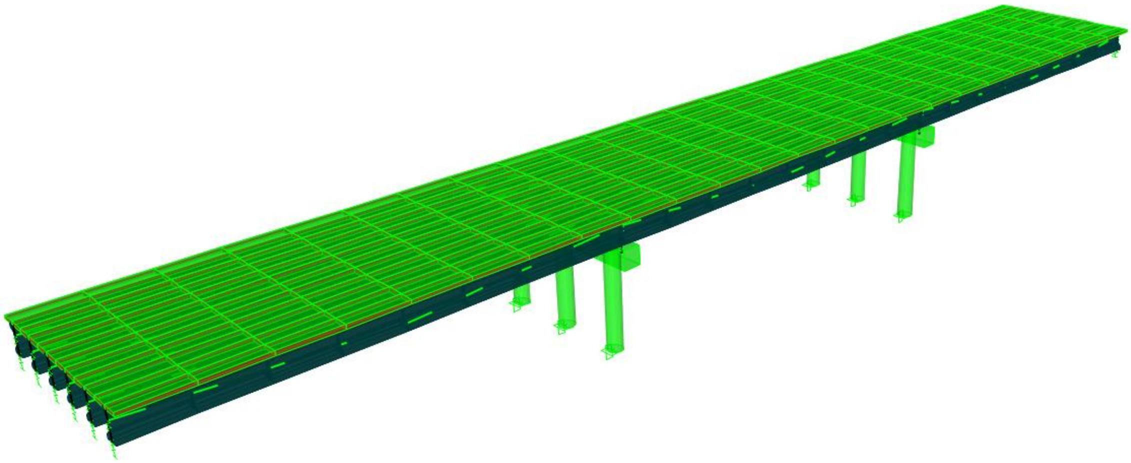

Figure 54: 3D Bridge model – CSi Bridge

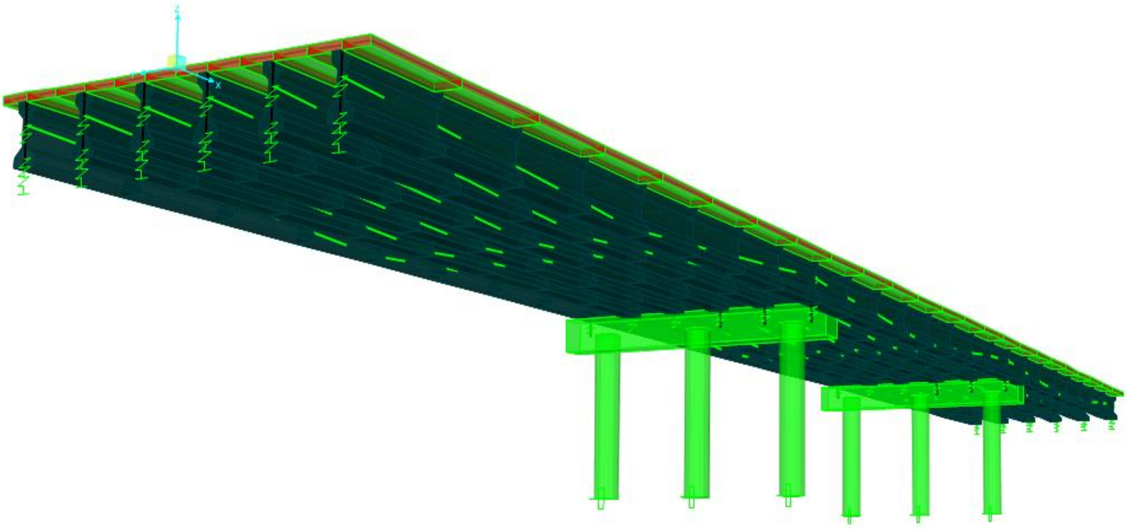

Figure 55: 3D Bridge model – CSi Bridge

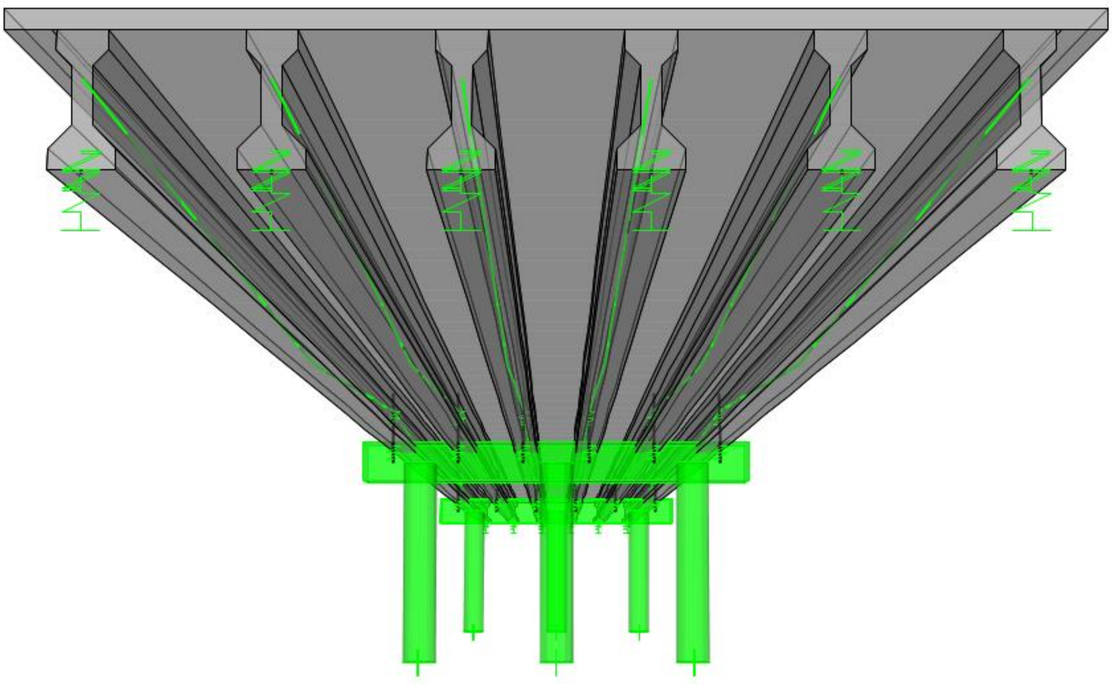

Figure 56: 3D Bridge model & bearings - CSi Bridge

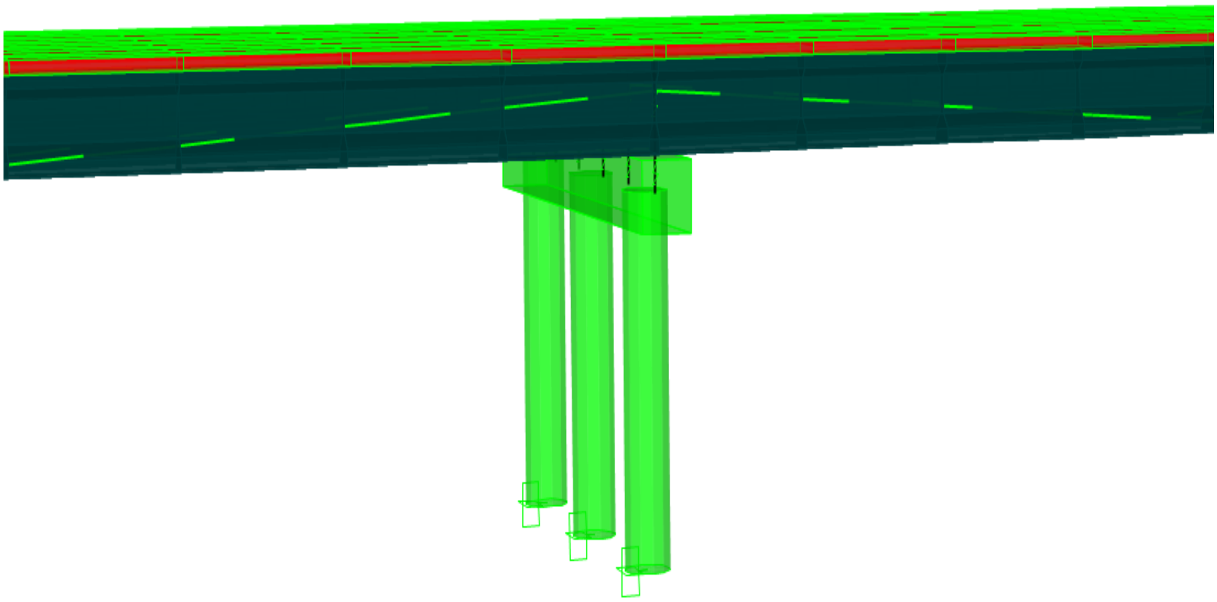

Figure 57: 3D Bridge model & bent – CSi Bridge

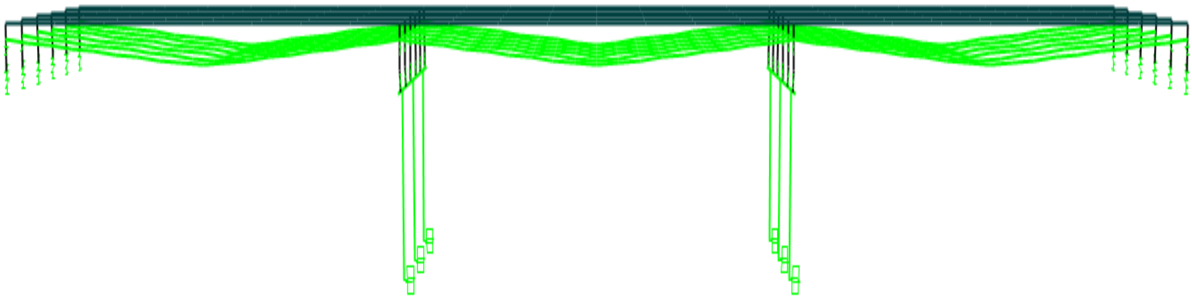

Figure 58: 3D Bridge model & tendons - CSi Bridge

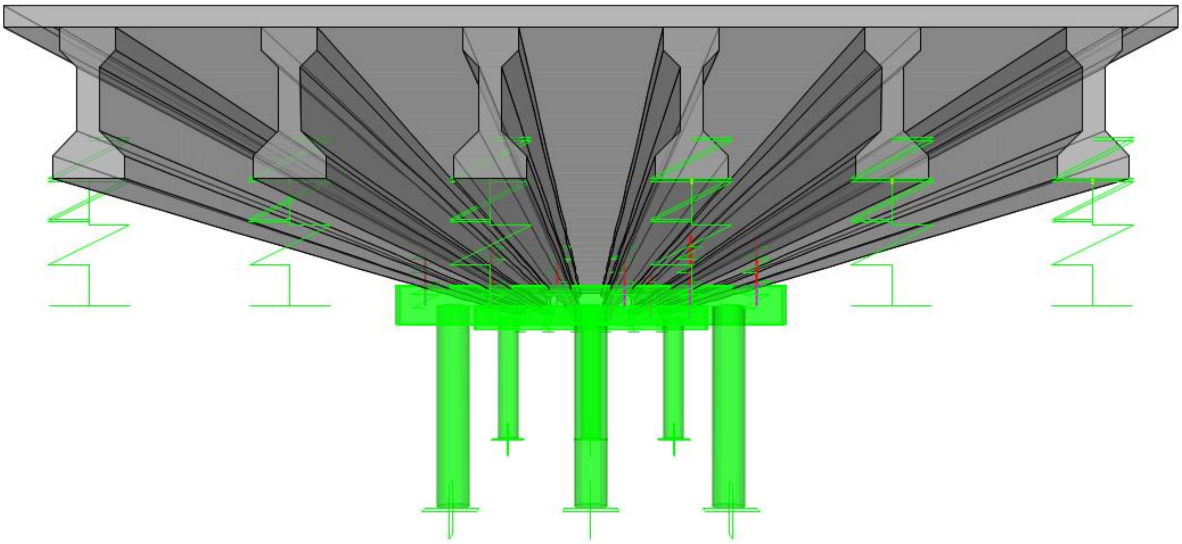

Figure 59: 3D Bridge model, superstructure & substructure - CSi Bridge

4. SEISMIC DESIGN/ANALYSIS – CSi BRIDGE

4.1. Setting Seismic Design Parameters

Prior to carrying out seismic analysis of the model, seismic design parameters are set by following the steps below: (see Figure 60)

1-Response spectrum function is selected to be “EQ”. EQ had been previously defined in Section 3.4.9.

2-Seismic design category is set to be “D”, so that the push-over analysis can be done by the software.

3-For the gravity load case, the entire structure is used.

4-The crack property and the modal load case options are selected as “program determined”.

5-Once the seismic design is completed; the software displays the load cases which are automatically created such as modal and pushover analysis.

	Item	Value
1	Seismic Design Category Option	Program Determined
2	Seismic Design Category	D
3	Bent Displacement Demand Factor	1,
4	Gravity Load-Case Option	Auto: Entire Structure
5	Gravity Load Case	
6	Additional Group	None
7	Include P-Delta	No
8	Cracked Property Option	Program Determined
9	Convergence Tolerance	1,000E-03
10	Maximum Number of Iterations	3
11	Accept Unconverged Results	Yes
12	Modal Load-Case Option	Program Determined
13	Modal Load Case	
14	Type of Modes	Eigen
15	Additional Number Of Modes	0
16	Response Spectrum Load-Case Option	Program Determined
17	Response-Spectrum Load Case	
18	Response-Spectrum Angle Option	Program Determined
19	Response-Spectrum Angle	0,
20	Directional Combination	Absolute

Figure 60: Seismic design parameters

4.2. Automatically Created Load Cases for Seismic Design

Load cases that have been automatically generated by the program are explained below. (Figure 61)

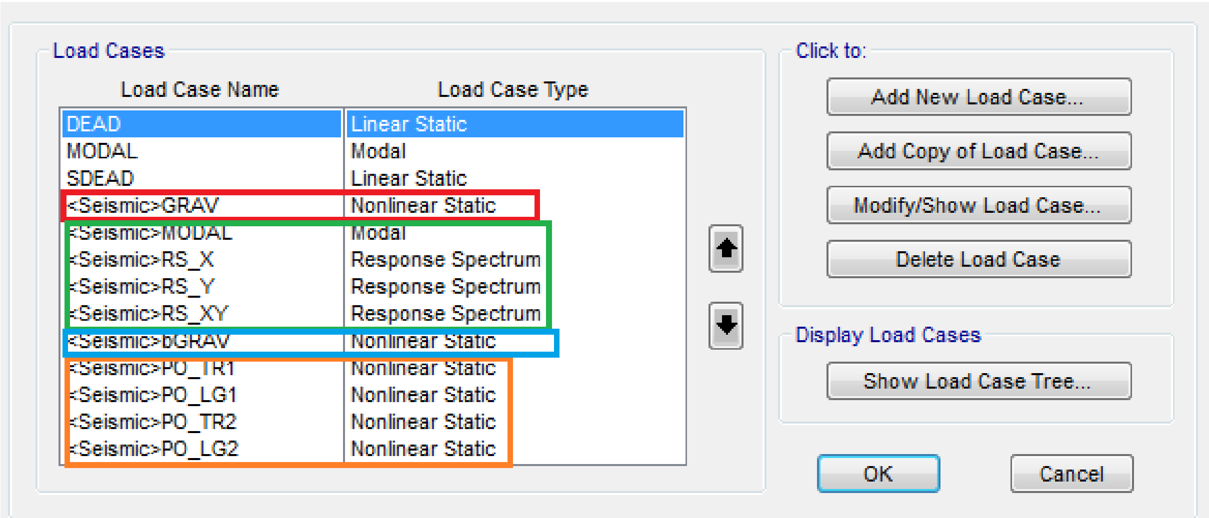

Figure 61: Automatically generated load cases

The loads cases which marked with red and blue (Figure 61) represent the dead load cases required in order to calculate cracked sections. Since the seismic design category of the bridge is “D”, the bents are loaded with equivalent loads from the bridge deck and pushover analysis is needed

The modal analysis and response spectrum analysis that are marked in green (Figure 61) are required in order to calculate the demand values. The response spectrum analysis is applied to the model in both x and y direction. (1.0L + 0.3T and 0.3T + 1.0L)

Lastly, the orange load cases in Figure 61 represent the pushover analysis in two directions on each of the two bent.

4.3. Modal Analysis Results

In a possible seismic activity, natural frequencies and periods of any structure are determined by running modal analysis. In order to obtain the seismic response of the proposed bridge, the first six modes (deformed shapes) and frequencies are shown below:

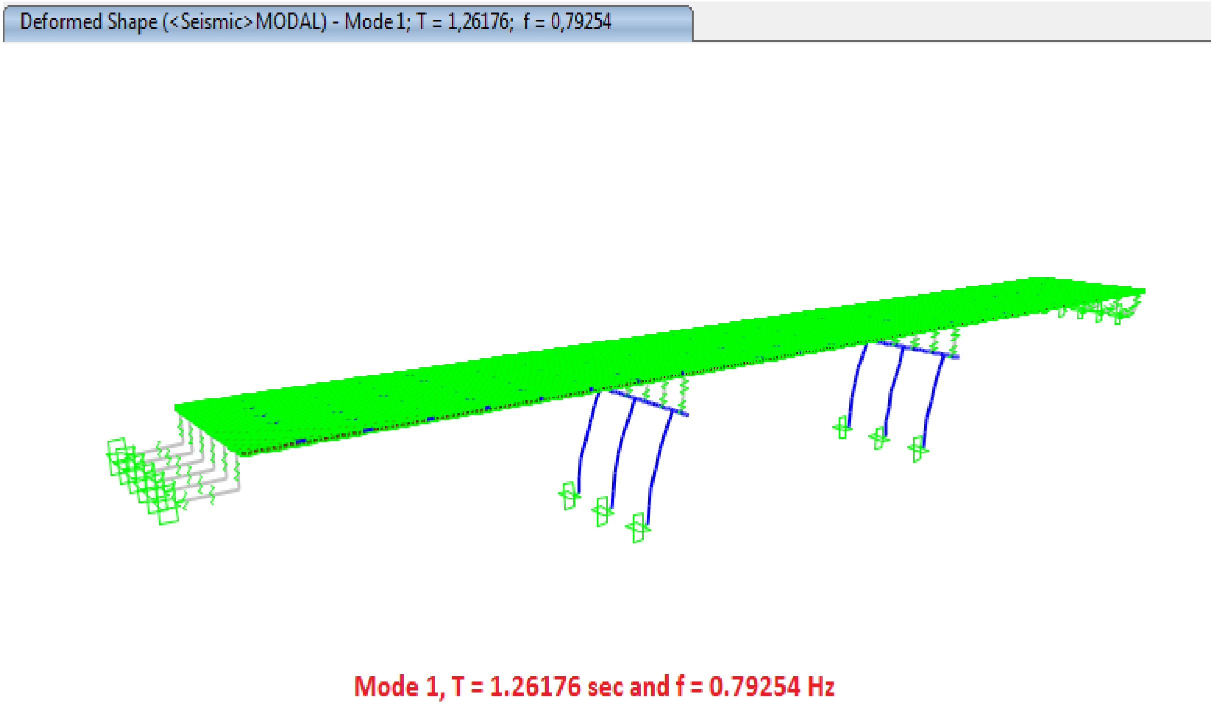

Figure 62: Mode 1

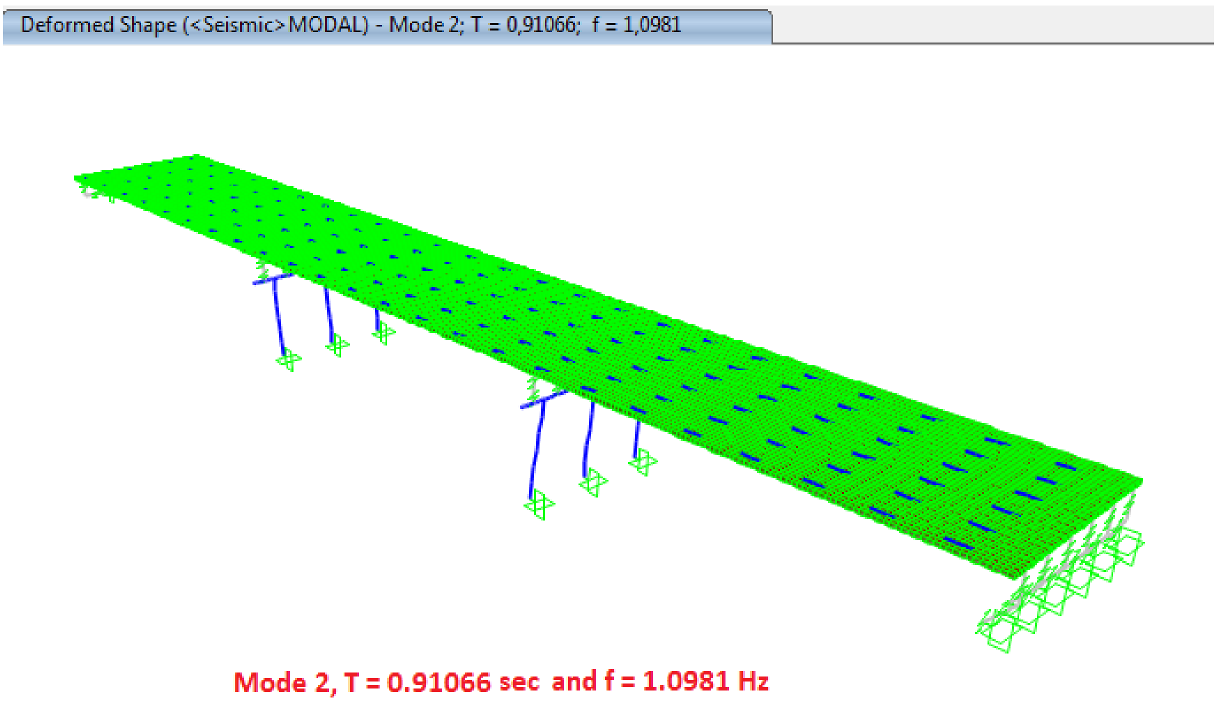

Figure 63: Mode 2

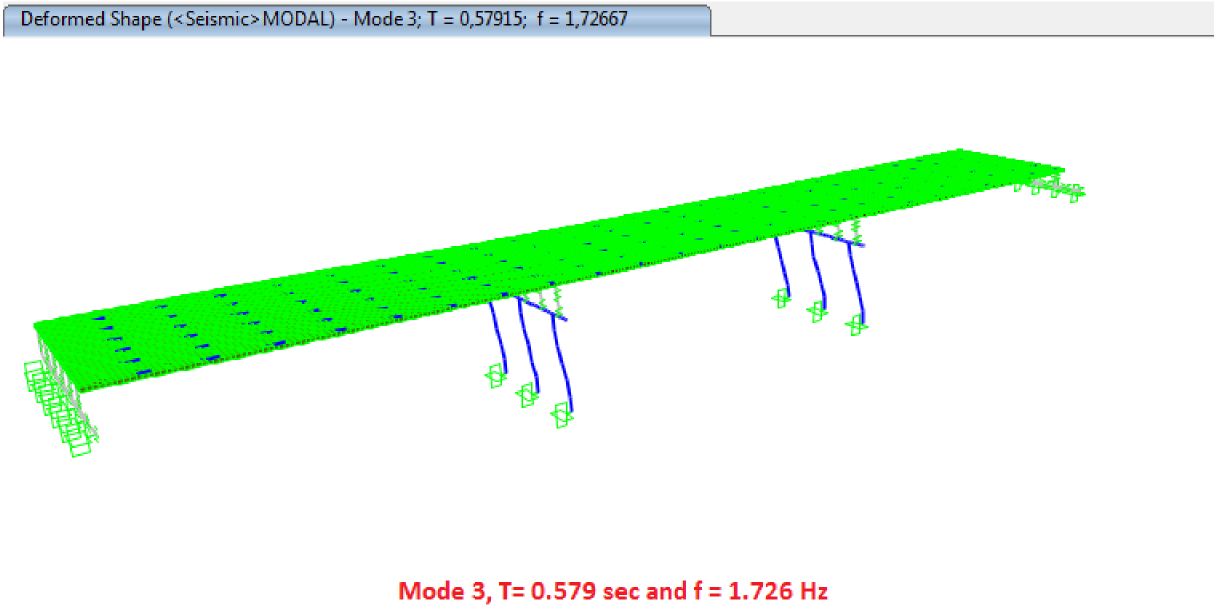

Figure 64: Mode 3

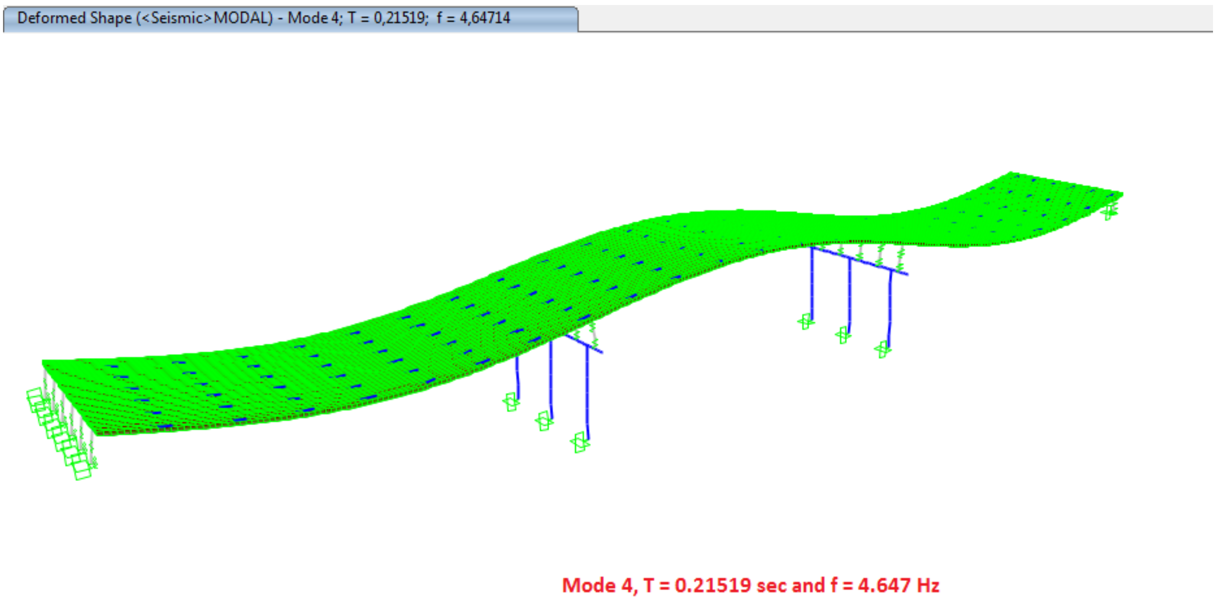

Figure 65: Mode 4

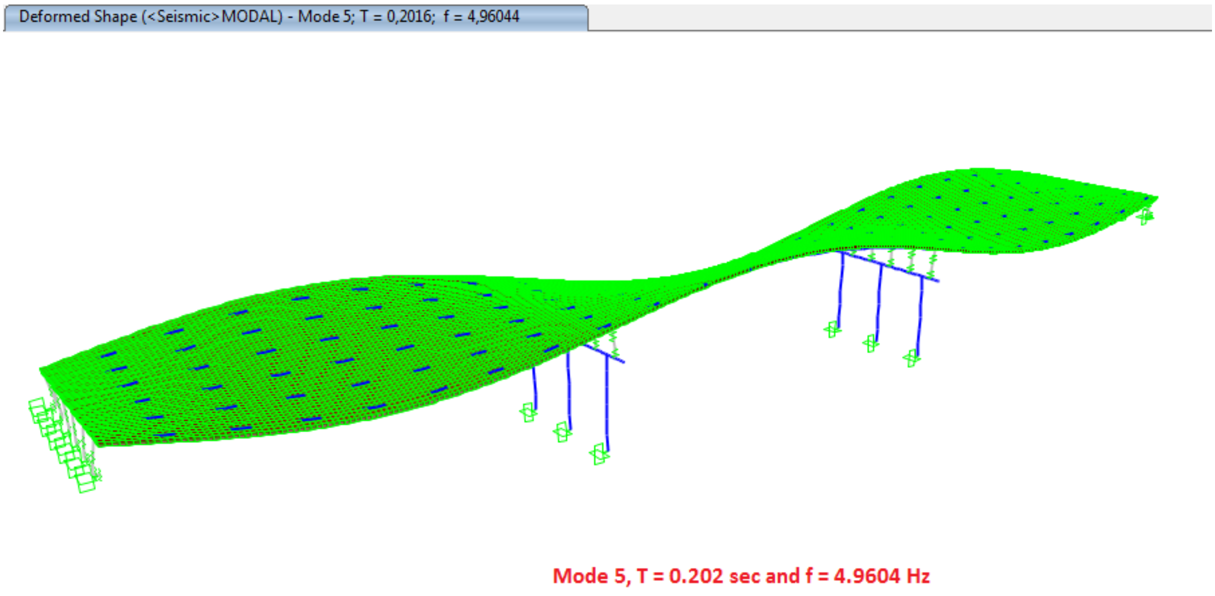

Figure 66: Mode 5

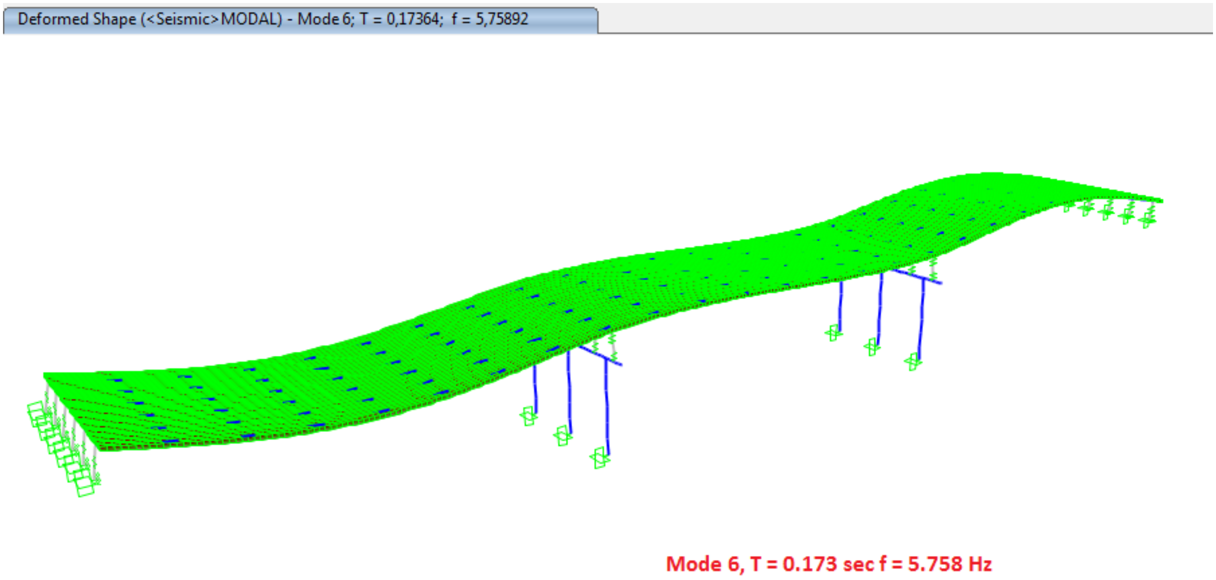

Figure 67: Mode 6

4.4. Pushover Analysis Results

Since the seismic design category is "D", CSi Bridge program calculates the displacement capacity by running the pushover analysis. Typically the displacement capacity is taken as the point on the curve that is located just after the peak value. The displacement capacities of the bents in both directions are shown in the following static pushover curves.

Static Pushover Curves:

- Load Case 1: PO-TR1 (for Bent 1 – in Transverse Direction)
  (Displacement capacity: 5.54 inches)

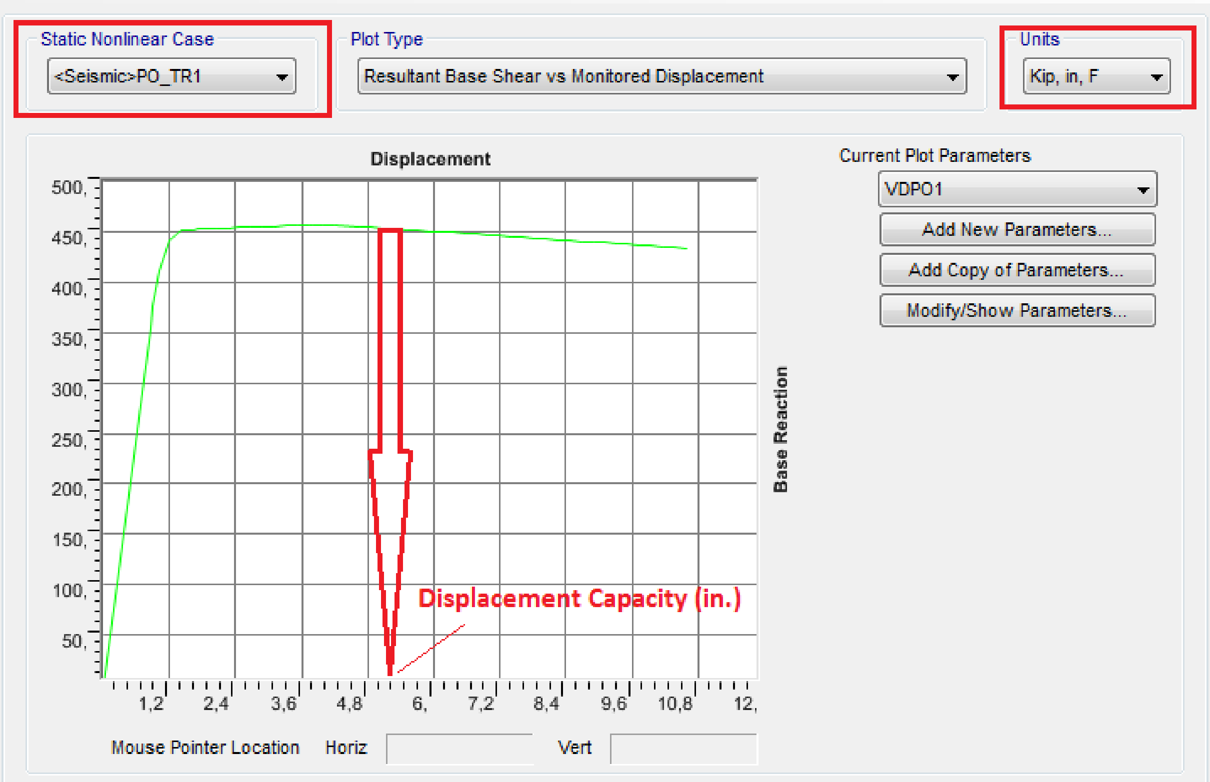

Figure 68: Static pushover curve - Load case 1

- Load Case 2: PO-LG1 (for Bent 1 – in Longitudinal Direction)

  (Displacement capacity: 7.53 inches)

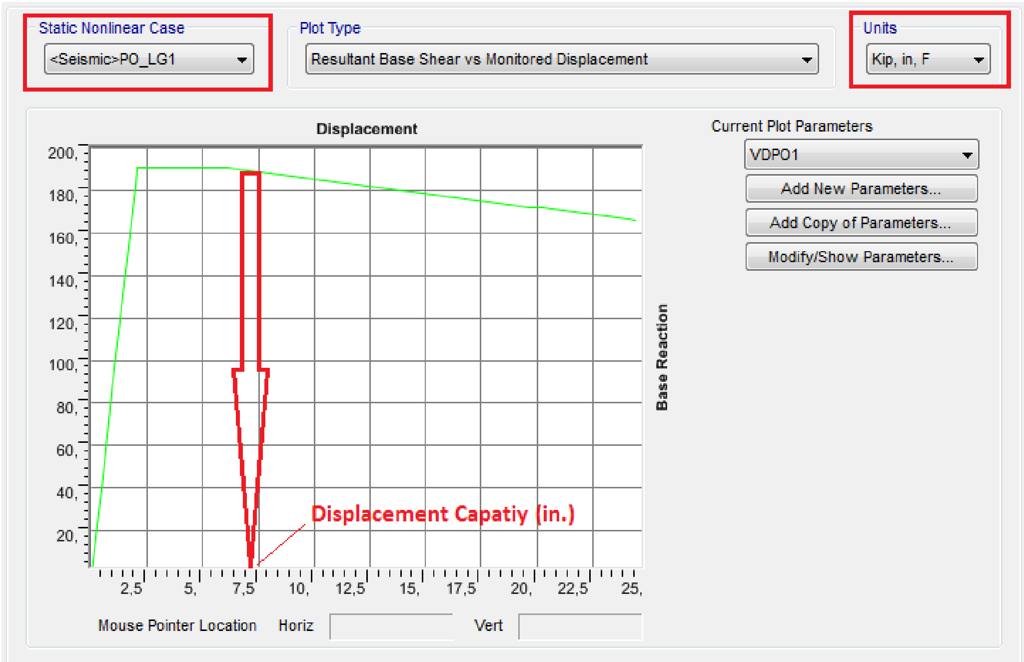

Figure 69: Static pushover curve - Load case 2

- Load Case 3: PO-TR 2 (for Bent 2 – in Transverse Direction)

  (Displacement capacity: 5.54 inches)

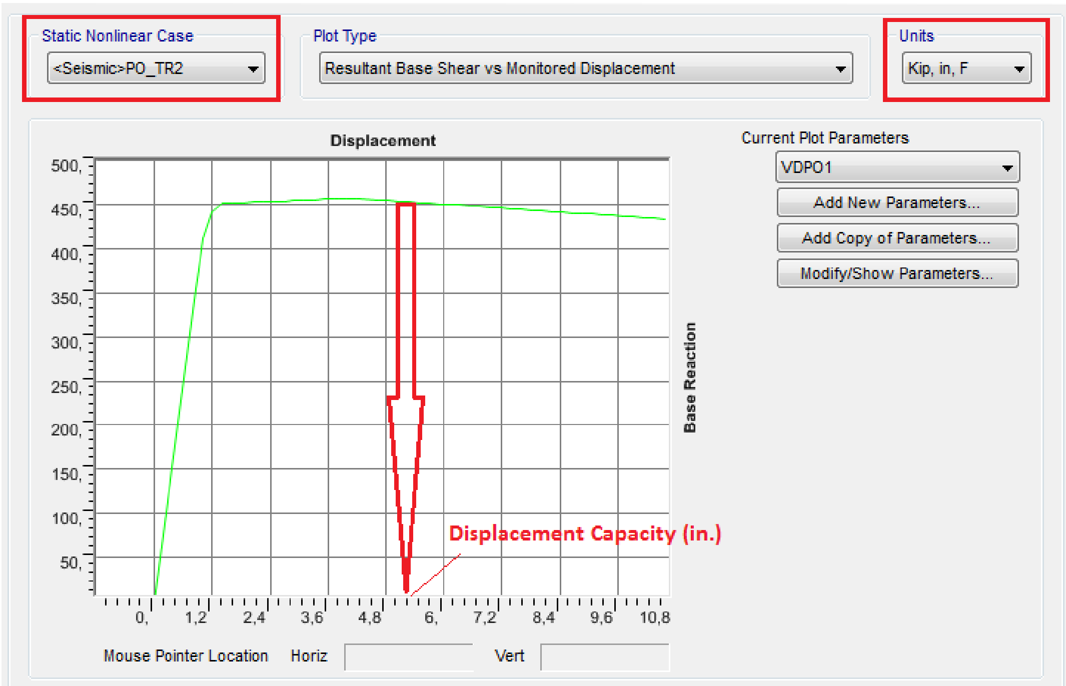

Figure 70: Static pushover curve - Load case 3

- Load Case 4: PO-LG 2 (for Bent 2 – in Longitudinal Direction)

  (Displacement capacity: 7.54 inches)

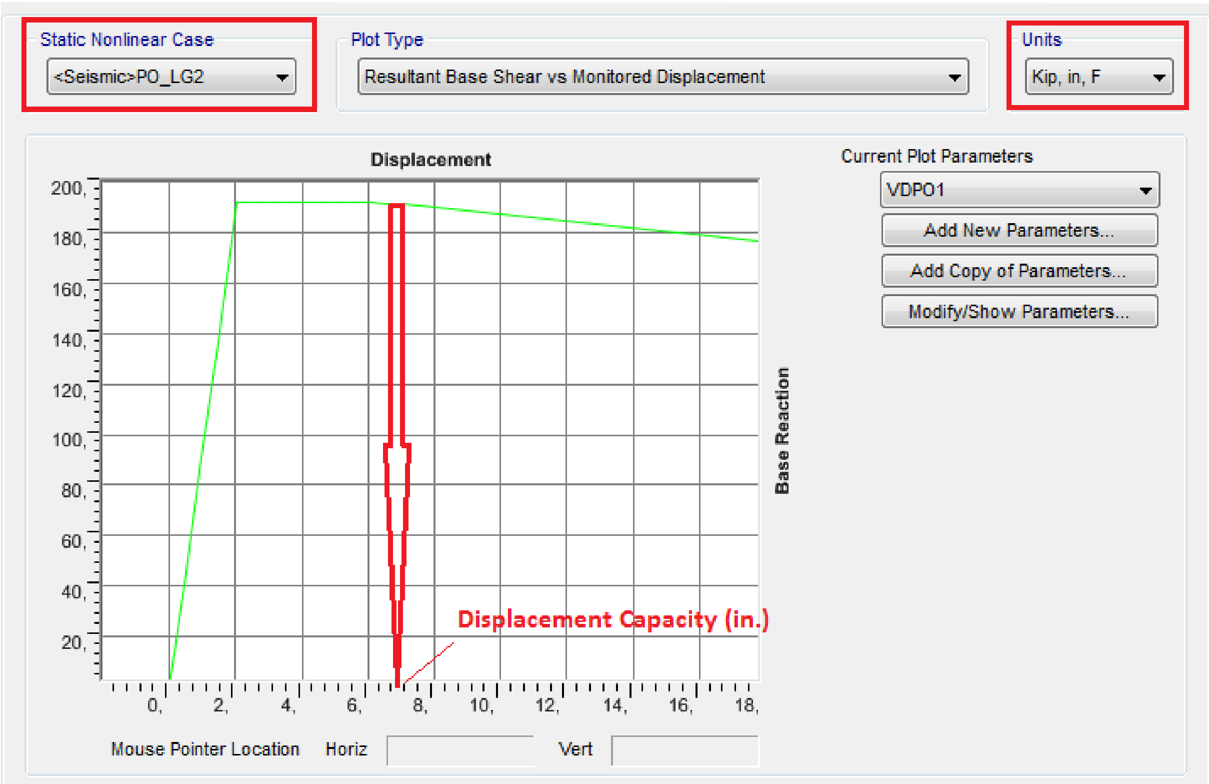

Figure 71: Static pushover curve - Load case 4

4.5. Demand/Capacity Ratios of Bents

D/C Capacities were found to be less than the critical threshold of 1; for both interior bents, translation D/C was 0.47 and longitudinal D/C was 0.81.

File View Format-Filter-Sort Select Options

Units: As Noted

Filter:

Bridge Seismic Design 01 - Bent D-C

DesReqName	BridgeObj Text	DCategor	SpanName Text	Station in	Direction Text	GenDispl Text	Demand in	Capacity in	DCRatio Unitless	Status Text
Seismic	BridgeModel	D	Span 1	960	TRANS	<Seismic>...	2,646686	5,54354	0,4774	Finished
Seismic	BridgeModel	D	Span 1	960	LONG	<Seismic>...	6,100918	7,531937	0,81	Finished
Seismic	BridgeModel	D	Span 2	1920	TRANS	<Seismic>...	2,646686	5,541371	0,4776	Finished
Seismic	BridgeModel	D	Span 2	1920	LONG	<Seismic>...	6,100918	7,545581	0,8085	Finished

Figure 72: Demand/Capacity ratios

5. CONCLUSION

The proposed bridge model has been examined in two parts - prestressing concrete girder design which produces some values to be inserted in to the second part - seismic design, where the model validation takes place.

During girder design, girder type, spacing and slap thickness were determined based on bridge geometry. Later, load distribution factors are derived in order to calculate bending moments and shear forces at critical sections. To wrap it up, required number of strands for girders has been obtained by taking bending moments and shear force in to consideration.

In seismic design, unlike girder design, seismic analysis of the proposed bridge was done using CSi Bridge software; in order to do so, the three-dimensional model in CSi Bridge had to be completed. Motion hazard is determined by using bridge coordinates; then, with the help of response spectrum function, seismic effects in both X and Y directions were simulated and applied. Later, displacement capacities of the bents were determined by examining static pushover curves. Lastly, modes were calculated and response spectrum analysis was performed in order to obtain demand/capacity ratios. Consequently, they were found to be less than the critical threshold of 1; specifically, for both bents, translation D/C was 0.47 and longitudinal D/C was 0.81. As a side note, comparing the result of modal analysis, frequency and period values were also determined to be expected and valid.

6. REFERENCES

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Vita

Alperen Ozel was born in Amasya, Turkey in 1991. He obtained his Bachelor's degree in Civil Engineering from Karadeniz Technical University in 2014. After his graduation, he moved to the United States and attended the University of New Orleans civil/structural engineering graduate program to pursue his Master's degree. During his graduate program, he focused on structural engineering.