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A Case Study of a Computer Aided Structural Design of a System Interchange Bridge

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***Abstract* -** In this paper, a case study of a computer aided design of a bridge that serves as an interchange is explored. The steps used in the design process and various challenges and AASHTO and ACI code requirements are addressed. Issues and challenges are explored and disseminated. The result is a clear and precise methodology of utilizing various computer based tools to accomplish the task at hand.

***Keywords*:** Bridge, CAD, AASHTO, ACI

1. Introduction

In this case study, the redesigning the system interchange located in Dbayyeh, Lebanon. This Interchange serves as a connector and distributor of all incoming flows between the freeways and inner city and the sea road. The design steps utilized by the Structural Engineer for the designing the whole bridge using Pre-stressed concrete are presented.

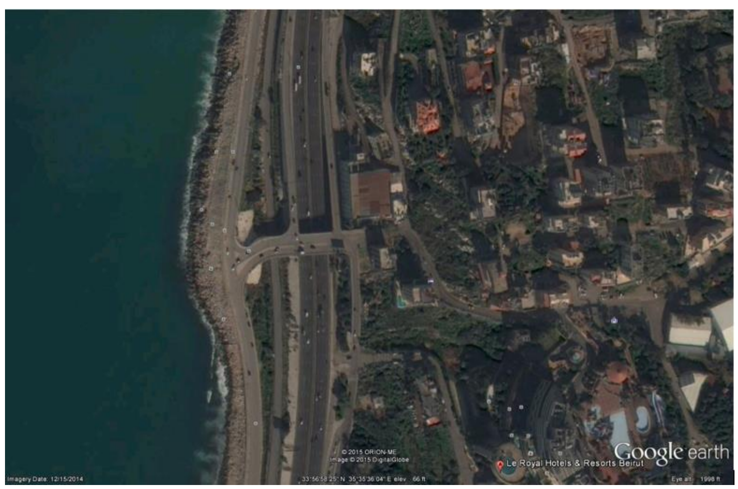


Fig. 1: Existing Interchange.

2. Bridge Dimension

The bridge (Figure 3) is 72m long and is divided into 3 spans of 24 m (79ft) each. The bridge constitutes of 2 piers and 2 abutment walls. The Piers and abutments are placed in a manner so that the bridge is divided into 3 equal spans. The Polygons colored in green in figure 3 represent the abutments of the bridge, and the polygons colored in blue represents the bridge piers. The bridge is divided into two directions. The directions are separated by a barrier. The 1st direction consists of 3 lanes of 3.6 m width each, and has a right and left shoulder of 1.2 m and 0.6 m respectively. The 2nd direction consists of 2 lanes of 3.6 m width each, and also has a right and left shoulder of 1.2 m and 0.6 m respectively. The total bridge width without the parapet is 22.2 m.

3. Bridge Design

According to Caltrans, California DOT Section 8.2.1:

The I-girder is most commonly used and has been in use in California for nearly 60 years. With bridge span lengths normally ranging from 50 ft. to 95 ft., the I-girder typically uses a depth-to-span ratio of approximately 0.05 to 0.055 for simple spans and approximately 0.045 to 0.05 for multi-span structures made continuous for live load. Having a multi-span structure with 3 equal spans of 24 m and knowing that the depth to span ratio is between 0.045 and 0.05 for multi-span structures. Thus select an AASHTO type III girder having a depth of 45” (1.14 m). Caltrans (California Department of Transportation) specifies that the maximum girder spacing can be taken to be 1.5 the depth of the superstructure, there use a girder spacing of 1.8m.Bridge Parapets that will be used having a width of 40 cm. According to MTD (Memo to Designers) 10-20, Attachment 1 (Caltrans, 2013), overhangs should be less than half the girder spacing (S/2) or 1.8 m maximum, therefore we can use AASHTO Type III Girders with a center-to-center spacing of 1.8m.

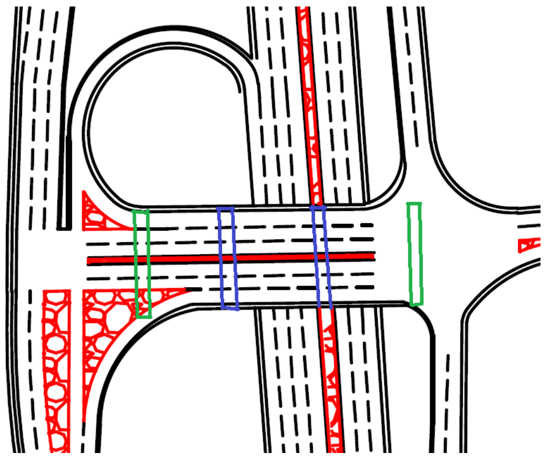


Fig. 2: Piers and Abutment Walls Placement.

4. Design Loads

Three traffic lanes loads were used with LRFD AASHTO Bridge Design Dynamic Load Allowance for bridge, IM, of 33 % and Multiple Presence Factor of 0.65. The loads acting on the different structural components of the bridge should be well defined and they include: a) Truck load: Design Lane Load (3.6.1.2.1), b) Barrier Load, c) Parapet Load, and d) Future Surface Wearing Load. According to AASHTO LRFD Bridge Design Specification section 3.6.1.2.5 Tire Contact Area, the tire contact area of a wheel is a rectangle whose width of 510 mm and length is 250 mm. The tire pressure is assumed to be uniformly distributed over the contact area. The Design Lane Load is 9.3 KN/m a uniformly distributed over a 3 meter width in the transversal direction. Force effects are not subject to Dynamic Load Allowance (3.6.1.2.4). Design Lane loads are spaced 0.6m apart from each other and extend over 3m in the transversal direction. The 1st Design lane load starts exactly from the face of the curb (uniform loading of 9.3 KN/m). The wheel load is placed as a point load 0.6 m apart from the edge of the design lane and the distance between the 2 consecutive trucks is 1.8m (3.6.1.2.3). The 2nd wheel load of the same truck is placed at a distance of 2 m from the 1st. The wearing surface load is 200N/m. According to AASHTO Specifications for Bridge Design the distance between the edge of the lane load and the truck point load should be 600mm however this is satisfied only in one end in our case, while on the other end it is only 400mm, and this is because the distance between the left and right set of wheels in the Mercedes Benz Actros 3344/45 Truck which is the most common truck in Lebanon is 2 m while the most common truck used in the USA has a distance of 1.8 m between the left and right set of wheels. For the seismic load, according to IBC and UBC Lebanon is classified as seismic zoning 3. Thus the corresponding acceleration shall be conservatively taken as 0.29 (Table 3.10.4-1). Since the substructure conditions are a combination of medium dense sands and sandy clay, the site coefficient can then be conservatively assumed to be 1.5 (S=1.5) (Table 3.10.5.1.1). The bridge categorized as a critical bridge, its corresponding importance factor I=1.5 (Table 3.10.7.1-1). AASHTO Section 3.6.4, specify that the braking force (BR) is taken as the greatest of 25% of the axle weights of the design truck or design tandem or 5% of the design truck plus lane load or 5% of the design tandem plus lane load.

5. Bridge Components Design

For the design of the deck slab the materials used have the following properties:

Concrete f'c = 42 Mpa = 6 ksi, Grade 60 Steel, fy = 420 Mpa = 60 ksi

The loads on the slab: Parapet load, barrier load, future wearing surface load and self-weight of the slab.

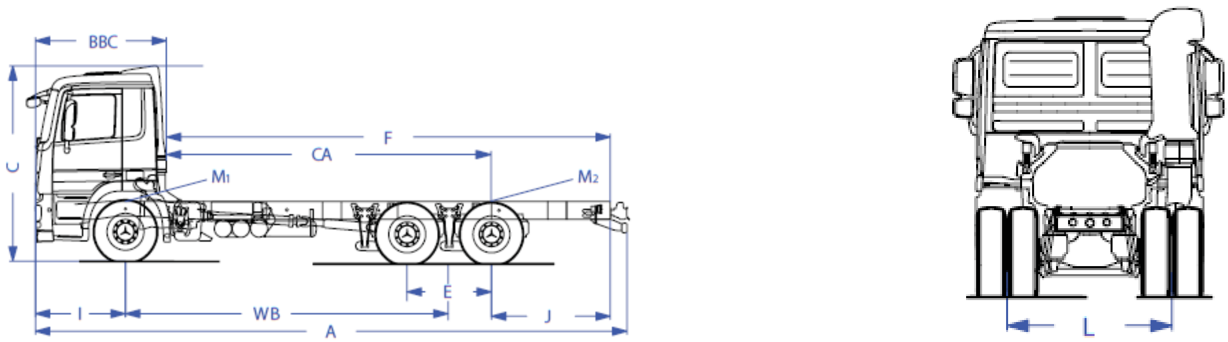
Parapet load = 8.25 KN; Barrier load = 16.5 KN

Future wearing surface load = 2 KN/m; Deck Slab Self Weight

Design Lane Load; Truck Point Load

Calculated before in Loads Section

$$W_{deck} = Deck\ Slab\ Thickness \times Slab\ Width \times Concrete\ \ Unit\ Weight = 6.25 KN/m$$



WB = 5.1 m E = 1.35 m L = 2 m

Fig. 3: Design Truck.

Table 1: Total Dead Load.

Dead Load Type	Span(m)	Area(m2)	Volume (m3)	Number of Units	Unit Weight (KN/m3)	Weight (KN)
Slab	72	8.05	579.6	1	25	14490
Girders	24	0.361	8.664	39	25	8447.4
Parapets	72	0.33	23.76	2	25	1188
Barrier	72	0.66	47.52	1	25	1188
Pier	NA	NA	15	2	25	750
					Total Dead Load (KN)	26063.4

In order to analyze the Truck Loads and Design Lane Loads on the deck slab two major cases are taken into consideration in the placing of the loads:

Case 1: represents the case where the loads are being placed as close as possible to the curb side and parapet in accordance with the articles present in AASHTO.

Case 2: resembles the case where the loads are being placed as close as possible to the middle barrier, also in accordance with the articles present in AASHTO.

The moments given by cases 1 and 2 will be compared and the greater value will control the design. Taking into consideration the limit states the Strength 1 limit state controls the Design. Thus Governing Shear Force (Strength 1) is 232.31 KN. Performing a quick calculation to obtain the shear strength of Concrete: we determine that we need to increase the slab thickness. After Readjusting the Deck Slab thickness to 35 cm so that the concrete will be able to take the shear forces without providing shear reinforcement. The maximum shear force obtained after increasing the self-weight of the slab, that is increasing the thickness of the slab is 232.87 KN. This value is obtained from Strength 1 Limit State which is the governing limit state that produces the highest values for shear and moments. Since the maximum positive and negative moments are close enough (67 KN-m compared to 62 KN-m) in terms of value, so there won't be a big difference in the steel provided for reinforcement. Thus we will take the greater value between the two moments which is $M_u^+ = 67.33$ KN-m and use it for design against both positive and negative flexure. Thus the same steel reinforcement is used in both layers the top one corresponding to the negative flexure and the bottom one corresponding for the positive flexure.

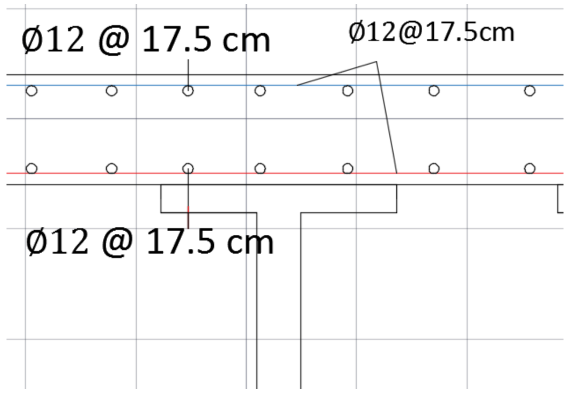


Fig. 2: Deck Slab Reinforcement.

6. Deck Joint Design

Deck Joint Gap is the gap used between adjacent spans in a bridge deck or between a bridge deck and abutment curtain wall. The joint gap width will vary where the joint is designed to accommodate thermal and other movements. First of all, and before starting the design, the range of temperature at Dbayyeh should be determined. It is mostly of common knowledge in the area that the maximum temperature they are exposed to is about 39°C. Also, the minimum temperature would be about 0°C.

In our case: $t_{high} = 39°C$; $t_{low} = 0°C$; $t_{average} = 19.5°C$; $\Delta t_{rise} = 19.5°C$; $\Delta t_{fall} = 19.5°C$

$\Delta x_{rise} = \Delta x_{fall} = \Delta t_{rise} \text{ x } L_{span} \text{ x } \alpha_T = 0.01516 \text{ m} = 15.16 \text{ mm}$

Deck expansion joint should also be considered to withstand shrinkage and creep.

However,

- Creep and Shrinkage prior to day 120 (casting of deck) is neglected for the expansion joint design.
- Creep [LRFD 5.4.2.3] is not considered at this time. After 120 days, all beams are assumed to creep towards their centers. The slab will offer some restraint to this movement of the beam. The beam and slab interaction, combined with forces not being applied to the center of gravity for the composite section, is likely to produce longitudinal movements and rotations. For most prestressed beams designed as simple spans for dead and live load, these joint movements due to creep are ignored.

For prestressed concrete structures, the movement is based on the greater of two cases:

- Movement from the combination of temperature fall, creep, and shrinkage.
- Movements of factored effects of temperature.

For case 1: $\Delta_1 = \Delta x_{fall} + \Delta sh = 15.16 + 36 = 51.16$ mm, only for contraction of joint.

For case 2: $\Delta_2 = 1.25 \text{ x } \Delta x_{fall} = 19$ mm for both expansion and contraction of joint.

Based on the longitudinal expansion (51.16 mm) the type of joint will be determined.

7. Girder Design

Since the bridge consists of 3 equal spans of 24 m each which is considered to be a relatively long span for normal reinforced concrete beams, pre-stressed girders will be used. Pre-stressed concrete girder are a type of girders that facilitates rapid construction of a bridge using girders that are fabricated off-site and then transported and erected into place at the job site. Because Pre-stressed girders require little to no false-work, they are a preferred solution for jobs where Accelerated Bridge Construction is sought, where speed of construction, minimal traffic disruption, and where temporary construction clearance is limited. Pre-stressed Concrete girders employ high performance concrete for strength, durability, and constructability and tend to be more economical and competitive when significant repeatability exists on a job (i.e., economy of scale). The longitudinal stiffness Parameter, k, is determine according to AASHTO to be in our case 0.47.

The final truck loading: $P1 = 37.5 \times 1.33 \times 0.47 = 23.45\,KN$; $P2 = 80 \times 1.33 \times 0.47 = 50.00\,KN$

Future Wearing Surface: $WFWS = 200 \frac{kg}{m2} \times Spacing\ of\ girders = 3.6\,KN/m$

Moment Due to Future Wearing Surface: $MFWS = 1/8\,W\text{FWS}\,x\,L2 = 259.2\,KN-m = 191.2\,K-ft$

Barrier Load on Girder: $W\text{Barrier} = Cross\ Sectional\ Area\ of\ Barrier \times Unit\ Weight\ of\ Concrete = 11\,KN/m$

Moment Due to Barrier: $M\text{Barrier} = 1/8\,W\text{Barrier}\,x\,L2 = 11/8\,x\,242 = 792\,KN-m = 584.1\,K-ft$

Deck Slab Self Weight:

$W\text{SLAB} = Thickness\ of\ Deck\ Slab \times Spacing\ of\ Girders \times Unit\ Weight\ of\ Concrete = 15.75KN/m$

Moment Due to Deck Slab: $M\text{SLAB} = 1/8\,W\text{SLAB} \times L2 = 1134\,KN-m = 836.3\,K-ft$

AASHTO states that when the span is greater than 40' a diaphragm should be added at mid-span to prevent the twisting of the girder.

Assume a diaphragm of 8" width and 45" depth

$P@mid-span = Diaphragm\ Weight: P = 1.83Kips\ Applied\ at\ midspan.$

$Md = PL/4 = 36.06\,K-ft$; M_{LL} = 1205.1 KN-m = 888.72 K-ft

<u>Basic assumptions prior to design:</u>

$f'c = 7.5ksi = 52\,MPa; f'ci = 5ksi = 35\,MPa$: $Strength\ of\ concrete\ at\ transfer$; Assumed Losses of 20%.

$fpu = 270ksi$: $ultimate\ strength\ of\ steel\ strands$; ½" $diameter\ seven\ wire\ strands$: $Asp = 0.153\,in2$

<u>Allowable stresses using AASHTO Stress limitations:</u>

@Transfer:

Maximum compressive stress: $\sigma_{ci} = -0.6f'ci = -3\,ksi$

Maximum tensile stress: $\sigma_{ti} = 3\sqrt{f'ci} = 0.212\,ksi$

@Service

Maximum compressive stress: $\sigma_c = -0.4f'c - (-0.4 \times 7.5) = -3\,ksi$

Maximum tensile stress: $\sigma_t = 6\sqrt{f'c} = 0.52\,ksi$

The Limit State that governs the design and is controlling over all the other combinations is Strength 1:

STRENGTH 1 = 1.25 DC + 1.5 DW + 1.75 LL

$M\text{LL} = 888.72 \times 1.75 = 1555.26\,K-ft; M\text{Deck} = 836.3 \times 1.25 = 1045.4\,K-ft$

$M\text{FWS} = 191.2 \times 1.5 = 286.8\,K-ft; M\text{g} = 454.82 \times 1.25 = 568.53\,K-ft$

$M\text{Barrier} = 584.1 \times 1.25 = 730.13\,K-ft; M\text{d} = 36.06 \times 1.25 = 45.1\,K-ft$

Number of strands: N=24

<u>Eccentricity:</u> At Mid-span: 17''; At Ends: 0''; Fi= 0.7f_{pu}xAsp x 28 = 694 Kips

<u>Check at mid-span:</u>

@Transfer: $Bot: \frac{-F_i}{A_g} - \frac{F_i\,e}{S_{bot}} + \frac{M_g}{S_{bot}} = \frac{-694}{560} - \frac{694 \times 17}{6186} + \frac{5458}{6186} = -2.667\,ksi > -3\,ksi\ OK$

$$Top: \frac{-F_i}{A_g} + \frac{F_i e}{S_{bot}} - \frac{M_g}{S_{bot}} = \frac{-694}{560} + \frac{694 \times 17}{5070} - \frac{5458}{5070} = -0.011\,ksi < 0.212ksi\ OK$$

@Service: (Un-shored System: Girders carrying the weight of the slab during erection)

$$Bot: -\gamma\frac{F_i}{A_g} - \gamma\frac{F_i \times e}{S_{bot}} + \frac{M_g + M_s + M_d}{S_{bot}} + \frac{M_{LL}}{S_{bot-c}} = 0.51 < 0.52ksi\ OK$$

$$Top: -\gamma\frac{F_i}{A_g} + \gamma\frac{F_i \times e}{S_{bot}} - \frac{M_g + M_s + M_d}{S_{bot}} - \frac{M_{LL}}{S_{bot-c}} = -2.44ksi > -3ksi\ OK$$

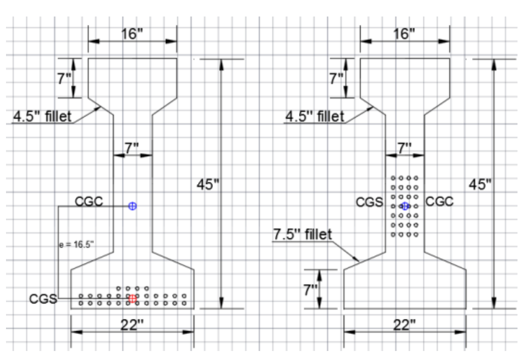


Fig. 5: Placement of Strands at mid-span and ends respectively.

8. Losses

For pre-tensioned concrete girder, the losses are calculated based on Elastic Shortening, Relaxation of Steel, Shrinkage, and Creep. The corresponding losses are calculated and added up to check if they meet the requirements of the assumed loss of 20% initially. The actual losses are calculated as:

$$\Delta f = \Delta fES + \Delta fpr + \Delta fps + \Delta fpc = -14.4 - 18.373 - 4.681 - 11.669 = -49.123$$

The value of calculated % losses is 25.989 which is clearly greater than the assumed loss of 20%. Thus, the redesign yields the requirement of using 28 strands with an eccentricity of 16.5" at mid-span and an eccentricity of 0" at the ends.

9. Diaphragm Design

The main function of the diaphragms (transverse girders) is to withstand the lateral forces acting upon the bridge superstructure, and to prevent the main pre-stressed girders from twisting due to the torsional forces. Lateral forces are basically composed of Earthquake Load and Wind Load which have been already determined as follows:

$$FCR = 78.8\, KN/m; \quad PD = 0.473 N/m2$$

The Diaphragm dimensions have been already taken as h = 45" (Same as the height of the girder), w=8''. The diaphragm is placed at mid-span and is designed as a compression member (Column). The largest tributary length for a diaphragm would then be: L_D = 24 m, in the longitudinal direction. Having said that, the earthquake and wind forces are calculated for each diaphragm based on the tributary length:

$$The\ earthquake\ force: EQ = FCR \times LD = 78.7 \times 24 = 1889\, KN$$

$$The\ wind\ force: WS = PD \times \left(\frac{45 \times 2.54 + 35}{100}\right) \times LD = 0.473 \times 1.493 \times 24 = 16.94\, N$$

According to AASHTO, the combinations that should be checked for the design are the following:

Extreme event 1: 1EQ and Strength 3: 1.4 WS

It is evident that extreme event 1 is the governing limit state with the following load. The figure below shows the reinforcement of the transverse girders.

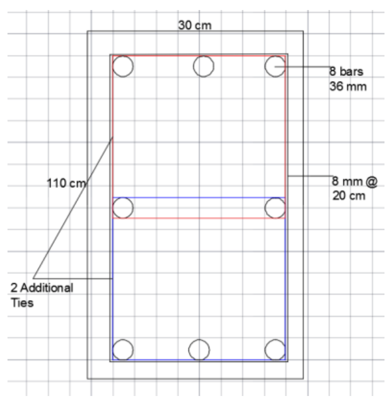


Fig. 6: Diaphragm Reinforcement.

10. Pier Cap Design

After determining the loads on the pre-stressed girders and the controlling limit state that governed the design of the girders, the reactions at the supports obtained from SAP2000 model of each girder will represent a point load on the pier cap which is to be designed as a deep beam. Note that the pier of the bridge is located right at the barrier-separator of the highway below the bridge, so in terms of dimensions the pier cannot have a width of more than 110 cm, and since the total width of the bridge is 23 m a set of 5 columns will be used with a center to center spacing of 4.6 m. The following figure represents the reactions obtained from SAP2000. The pier cap carries the weight of a whole girder (half from the left span and another half from the right one) in addition to whatever loads the girder is carrying so in order to place the girder loads on the pier cap the reaction (1120 KN) should be multiplied by 2. After analyzing the loads on the pier cap through SAP2000 the shear diagram is obtained.

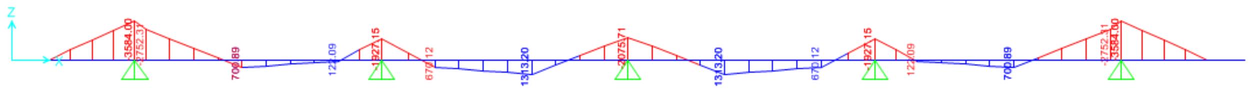


Fig. 3: Pier Cap Moment Diagram.

The maximum negative moment is $M_U^- = -3584$ KN-m; The maximum positive moment is $M_U^+ = 1313.2$ KN-m According to ACI 11.8 what really determines if a beam is considered to be a deep beam or not is the ratio of l_n over d where: l_n is the length from face of support to face of support; d is the depth of the center of gravity of the reinforcement bars; If $l_n/d < 5$ then the beam is considered a deep beam. The dimensions of the deep beam are assumed and then rechecked and h = 100 cm and b = 85 cm. After adding the self-weight of the pier cap to the loads coming from the pre-stressed girders the following new results were obtained: the Maximum Shear Force 4164 KN and the maximum negative moment is -3589 KN-m and maximum positive moment is 1315 KN-m. $Use: f'c = 6\ ksi\ =\ 42\ Mpa\ , fy = 60\ ksi\ =\ 420\ Mpa.$ $l_n = 4$ m

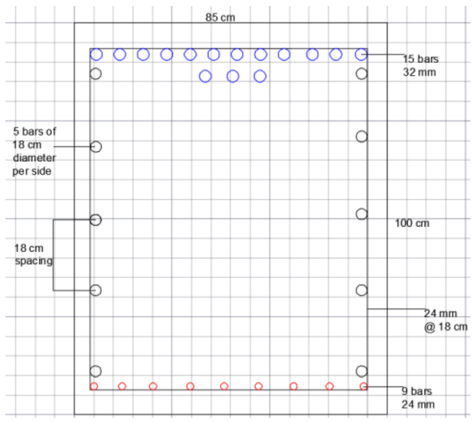


Fig. 8: Pier Cap Reinforcement.

11. Pier Design

The pier is the structural element of the bridge that transfers all the loads from the superstructure to the substructure. It is the main support of the bridge and must satisfy all live load cases, and sustain all other axial and transversal loads (Earthquake Load). The pier of the bridge is located right at the barrier-separator of the highway below the bridge. The bridge is supported on 2 piers each consisting of 5 circular columns. The pier loads and moments applied on every column must be obtained. The multi-column pier will be supporting the pier cap and all the types of loads that the pier cap is carrying, taking into consideration the different limit states. The following 3 limit states are the combinations that must be taken into consideration when designing a bridge pier. Due to previous analysis, it was evident that Strength 1 controls the design; hence this combination must be used as the controlling one. However, this combination does not take into consideration the earthquake loading. Thus “Extreme Event 1” limit state is taken into consideration, since it is the combination which recognizes the earthquake effect on the structure.

Strength 1: 1.25 DC + 1.5 DW + 1.75 LL + 1 WA

Strength 4: 1.0 DC + 1.0 DW + 1.0 LL + 1.0 WS + 1.0 WL + 1 WA

Extreme Event 1: 1.25 DC + 1.5 DW+ 1 LL + 1 WA+ 1 EQ

Since there is no water loads then WA = 0. The wind on live load is negligible in comparison with wind on the structure thus WL = 0. The values for both Earthquake loads and wind loads were already calculated, and it was found out that earthquake loads control over the wind loads. Thus“Strength 4” limit state can be eliminated without going through the analysis, because either “Strength 1” or “Extreme Event 1” will definitely control the design.

These loads consist of the following:

1. DC: Dead load of structural components and this includes:
   Deck Slab Self-weight, Parapets weight, Barrier Weight, Girders Weight, and Pier Cap Weight
2. DW: Dead Load of wearing surfaces
3. LL: LL = TL +LN

LL: Vehicular live load; LN: Design lane load; TL: Design Truck load or Design Tandem load

4. EQ: Earthquake Load. This load can be either in the longitudinal direction or in the transversal one. Each of these cases must be studied alone, and the one that produces higher values for moment and shear will be used in designing the circular columns. The Longitudinal Earthquake Force, F_{CL} = 11337.6 KN and the Transversal Earthquake Force, F_{CR} = 78.9 KN/m x 36 m = 2840.4 KN

The transverse earthquake load is applied at the top of the column (at 5m above the highway) beneath the bridge. Placing the transversal force on the left or right side will produce the same results of moments and shear. The only difference is that the column carrying the highest value of moment and shear forces will change, but this is not even an issue since all the columns will be designed based on the highest value of shear and moment. This load produces a maximum moment at the bottom equal to: M_{EQ} = 1670 KN-m as illustrated in the figures below:

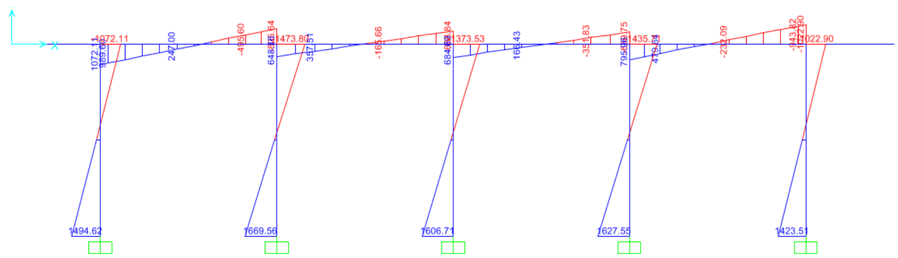


Fig. 4: Moment Diagram Due to Transversal Earthquake Loads.

The longitudinal load is applied at the top of the column (at 5m above the highway beneath the bridge). The maximum moment produced at the bottom equal to: M_{EQ} = 10323 KN-m as illustrated in the figures below:

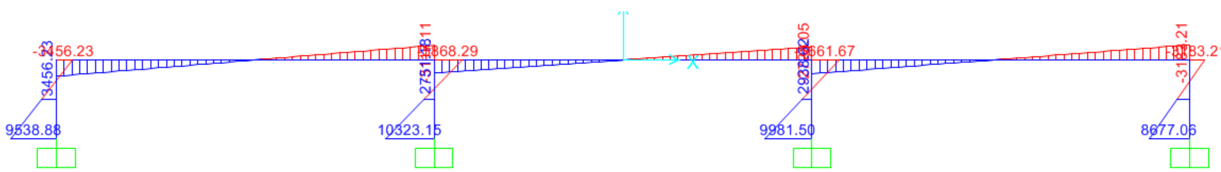


Fig. 5: Moment due to Longitudinal Earthquake Load.

This moment is distributed equally among 5 circular columns thus the maximum moment due to earthquake load is 2065 KN-m per column. After running the analysis, it is obvious that longitudinal earthquake load is more critical than the transversal one with a controlling moment of 2065 KN-m. The pier will be designed for both limit states and the combination which will result in the biggest amount of steel reinforcement for the same section of the pier will be the limit state that dictates the design. From the figure represented above the maximum axial load that the column has to withstand is 6211 KN, with Maximum Moment at bottom of 358 KN-m, and Maximum Moment at top of 745 KN-m. By applying these loads on Robot, the design obtained was a 65 cm diameter column with 20 φ 25 main reinforcement and φ 10 mm ties @ 39 cm.

12. Abutment Wall Design

The Structure upon which the ends of a Bridge rest is referred to as an Abutment. The most common type of Abutment Structure is a Retaining Wall. Although other types of Abutments are also possible and are used. A retaining wall is used to hold back an earth embankment or water and to maintain a sudden change in elevation. Abutment serves following functions: a) Distributes the loads from Bridge Ends to the ground, b) Withstands any loads that are directly imposed on it, and c) Provides vehicular and pedestrian access to the bridge.

In case of Retaining wall type Abutment bearing capacity and sliding resistance of the foundation materials and overturning stability must be checked. The abutment walls were designed as both a retaining wall, and a basement wall. The retaining wall has to withstand the earth lateral pressure of soil in addition to the live load surcharge due to the vehicle flow over the approach slab. As we previously mentioned, the bridge consists of 2 abutments one at each end, and since there is a difference in elevation between the roads that the bridge is connecting, then we have to design 2 different walls. The 1st abutment has a height of 6 m and the 2nd abutment has a height of 4 m. To determine the live load surcharge the wheel load of the Mercedes Benz Actros should be taken into consideration which has a rear axle load of 16 Tons; that is a wheel load of 8 Tons (80 KN). This load is distributed over an area of 0.25 m x 0.510 m. A ROBOT model was created and the different loads were applied to determine the required steel reinforcement. The following loads are calculated from "Extreme Event 1" limit state and act on the wall:

Girder Load: This is the AXIAL load coming from the superstructure due to the girders resting on the wall, it is a point load of 1020 KN along the 23 meter length of the wall.

Live Load Surcharge: This load is due to the truck load on the approach slab, and the weight of the slab itself, which adds up to 1606 KN/m^2.

Soil Load: load due to the backfilling material that the wall has to be filled with.

Applying these loads, the ROBOT model of a 50 cm wall, yielded special edge reinforcement that differs from the reinforcement applied to the entire wall. This is because the wall is treated also as a shear wall.

13. Pile Cap Design

A pile cap is a thick concrete structure that rests on concrete piles to provide a suitable stable foundation. It usually used when the soil conditions are poor, and when the axial loads coming from the structure are really heavy. From the Geotechnical Engineer there are 12 different piles having a diameter of 0.8 m connected by a pile cap that has a foot-print of is 25.6 m by 4.8 m. The Piles are distributed as shown in Figure 11. The Maximum Factored load given per pile is 2750 KN. The pile cap will be designed to resist the flexural bending, and shear failure, punching shear in particular. The controlling factor that governed the thickness of the pile cap is Punching Shear. The pile Cap is modelled on ROBOT, and the obtained thickness is 65 cm. Checks for punching shear were performed to find out that controlling regions where found out to be at the corners, and this makes sense because at the corners the value of b_0 (which is the punching perimeter) is smaller than that of the piles located at the edges.

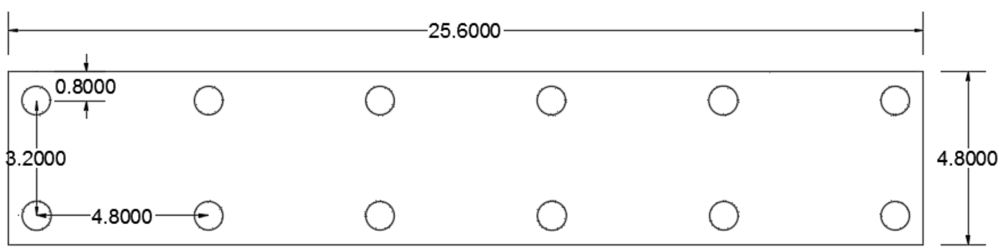


Fig. 6: Piles Distribution and Spacing.

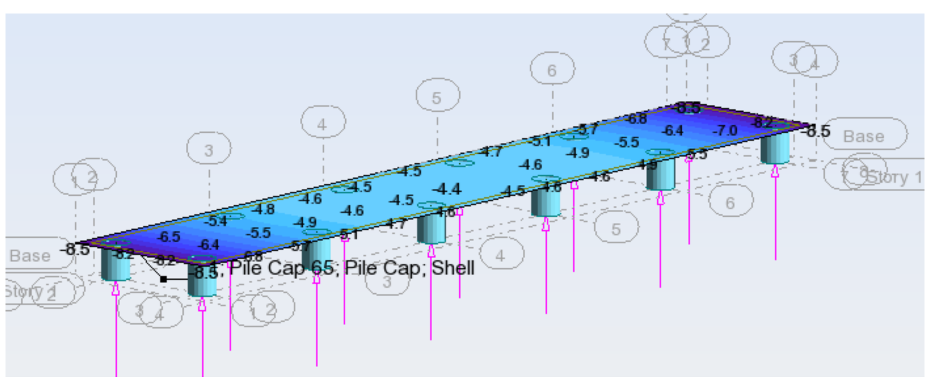


Fig. 7: ROBOT Model.

The red color represents high values of moments present, and thus the high amounts of steel reinforcement required, so the darker the color the higher the moment is and the greater the amount of steel provided would be.

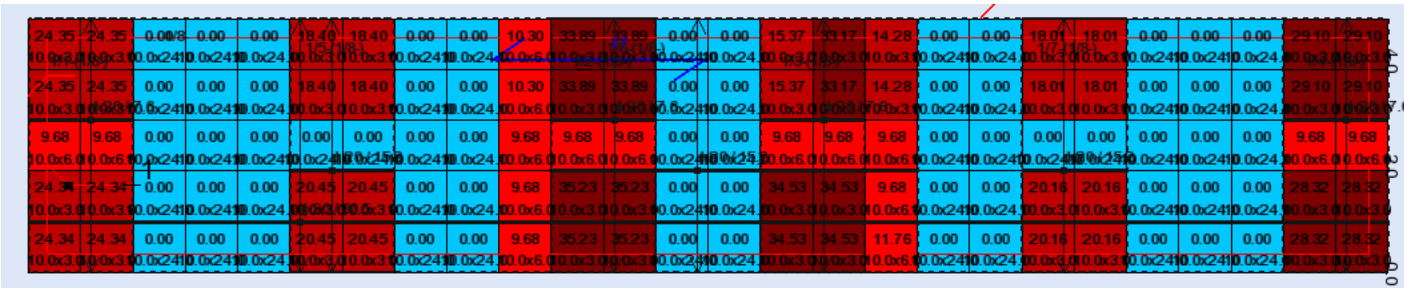


Fig. 13: Top Moment.

Conclusion

In this paper a case study was presented in which different Civil Engineering Computerized Design Programs were used to design a bridge while adhering to the latest Code requirements.

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